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Midland Plant, Units 1 and 2
Final Safety Evaluation Report Supplement
Geotechnical Engineering

Enclosure 1

Midland Plant, Units 1 and 2
Docket Numbers: 50-329/330
Final Safety Evaluation Report Supplement
Geotechnical Engineering
Prepared by: Joseph D. Kane, HGEB, NRR and
Includes Input from GES Consultants,
Hari Singh, U. S. Army Corps of Engineers and
Steve Poulos, Geotechnical Engineers, Inc.

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 1 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 June 18, 1982 along with the previously identified documents in Section 2.5.4 of the SER have been reviewed by the staff and its consultants and serve as the basis for the following sections which present the results of our safety evaluation.

In addition to identifying the applicable criteria (CFR, R.G., SRP, NUREGs) under which FSAR Section 2.5.4 review has been conducted, the SER also discussed the following topics related to the plant fill settlement problems:

a. Discovery of the plant fill deficiencies - SER Section 1.12
b. Affected safety related structures and utilities - SER Section 1.12 and Table 2.2.
c. NRC issuance of the Order Modifying Construction Permits and a related Licensing Board Order - SER Section 1.12.

2.5.4.1 Site Conditions

2.5.4.1.1 General

The proposed Midland nuclear 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 elevation 594 feet (National Geodetic Datum) along the Tittabawassee flood plain to elevation 630 feet in the southwest portion of the site area. In order to reach plant grade elevation 634 feet and to be above the floodplain, 30 to 35 feet 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 on FSAR Figure 2.5-46. The average original ground surface which existed prior to placement of the plant fill was slightly above elevation 600 and it is this surface below which future references in this SSER to natural soils is intended. Plant fill placement activities were conducted largely 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 elevation 575 to elevation 600 feet but has been located as deep as elevation 552 feet 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 to depths as deep as elevation 545 feet. Glacial till which 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 elevation 365 to 430 fee . Below the clay till and above the black shale bedrock of the Saginaw formation lie glacial outwash consisting of predominantly very dense fine sand layers (SP) with silt that are occasionally interlayered with very stiff clayey sands and very dense sand and gravels and very dense silts with gravel. The top of bedrock is encountered at approximately elevation 250 feet 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 percent of maximum dry density established in ASTM D1557 and 80 percent relative density, ASTM D2049, respectively) is considered to be the major cause of the plant fill settlement problem.

2.5.4.1.2 Site Foundation Description

Tables 2.1 and 2.2 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 notes on these tables also indicate the foundation remedial measures proposed for the various structures supported on the plant fill.

Table 2.1

Safety-Related Structures Founded on Natural Soils

Structure	Supporting Foundation Soil	Foundation Elevation	Foundation Type
Reactor Containment Buildings	Very Stiff to hard clay	572 to 582.5	9 ft to 13 ft thick reinforced concrete mat
Main Auxiliary Building	Very stiff to hard clay	562 to 579	5 ft to 6 ft thick reinforced concrete mat
Service Water Pump Structure (deeper portion)	Very Stiff to hard sandy clay	587	5 ft thick reinforced concrete mat

Table 2.2

Safety-Related Structures Founded on Plant Fill

Structure	Supporting Foundation Soil	Original Foundation Elevation	Original Foundation Type
Control tower	Plant fill	609(1)	5 ft thick(1) reinforced concrete mat
Electrical penetration areas	Plant fill	609(1)	5 ft thick(1) reinforced concrete mat
Feedwater isolation valve pits	Plant fill	615.5(2)	4 ft thick(2) reinforced concrete mat
Railroad bay	Plant fill	630.5	4 ft thick reinforced concrete mat
Service water pump structure	Plant fill	617(1)	3 ft thick reinforced(1) concrete mat
Diesel generator building	Plant fill	628	2.5 ft thick by 10 ft wide(3) continuous reinforced concrete wall footing
Diesel fuel oil tanks	Plant fill	612	3 ft thick concrete(4) pads
Borated water storage tanks	Plant fill	629	Continuous reinforced (4 and 5) concrete ring wall on 1.5 ft thick by 4 ft wide footings.

Notes:

(1) To be modified with permanent underpinning wall.

Table 2.2
(Continued)

(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 wall foundation to be constructed and resetting of Unit 1 tank is to be completed.

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 which 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 which 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 for 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 in order to establish static and dynamic engineering properties. The depth of borings varied widely and ranged from a minimum of four feet up to 370 feet where rock cores using an NX core barrel were obtained.

Based upon the information presented in the FSAR and technical reports, the staff and its consultants conclude that the site investigations completed by the applicant are acceptable and adequate in identifying the important subsurface features and foundation conditions and the investigations were completed in accordance with the guidelines recommended in R.G. 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 Subsection 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 previously identified in Section 2.5.4 are listed on Table 2.3.

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

Table 2.3

Engineering Properties of Natural and Fill Foundation Soils

Foundation Soil	LL	PL	Shear Strength Undrained C (ksf)	Drained Ø (degrees)	C (ksf)	SPT (blows/ foot)	Shear Wave Velocity (ft/sec)
Natural - Very stiff to hard clay (CL) (Reactor and Auxiliary Bldgs).	42	20	5.2 to 9.3 Median 7.6 Used 7 in design	23 Used in design	1.2	56 median	850-2300 Used in design
Natural - Very stiff to hard sandy clay (CL) (Service Water Pump Structure)	17	11	11.4 to 18.2 Median 15 Used 8 in design	36	0.7	75 median	850-2300 Used in design
Plant Fill - Silty clay (CL) (Diesel Generator Building after surcharging)	19-46	11-18	3.0 Used 2.7 in design	32 Used 29° and 0.1 in design	0.1	--	500-1000 used in desig
Bedrock - Black shale	--	--	Unconfined comp. 6000 to 7600 psi			--	5000 Used in design.

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 2.4. The staff will require submittal of the actual as-built foundation conditions for the auxiliary building and service water pump structure portions in a future amendment to the FSAR following completion of this underpinning construction work.

The staff and its consultants conclude that the soil profiles and sectional views are adequate and acceptable in appropriately representing the results of the subsurface investigations. The staff and its consultants find the engineering properties to be acceptable that have been used in design as shown on Table 2.3 for the various foundation layers depicted on the profiles and sections.

Table 2.4

Pertinent Soil Profiles and Sectional Views
Presenting Subsurface Investigation Results

Structure	Profile or Section	Document
Reactor Containments	Figures 2.5-20, 21 and 169	FSAR
Auxiliary Building	Figures 2.5-20, 21 and 169 AUX-38	FSAR Applicant's ASLB Testimony - November, 1981
Service Water Pump Structure	Figures 2.5-22 and 170 SWP	FSAR Applicant's ASLB Draft Testimony
Borated Water Storage Tanks	Figures 2.5-176, 182 and 183 Figures 4, 5, and 6	FSAR Applicant's ASLB Testimony November, 1981
Diesel Generator Building	Figure 2.5-177 (Prior to surcharging)	FSAR
Underground Piping	Figures 2.5-100 and 101	FSAR
Diesel Fuel Oil Storage Tanks	Figure 2.5-191	FSAR

2.5.4.4 Foundation Treatment

The following sections provide the geotechnical engineering staff and its consultants evaluation of the techniques proposed by the applicant to treat the deficiencies in the plant fill and to assure long term foundation stability.

2.5.4.4.1 Underpinning

In this section the cause of the need for underpinning is described and the design of the underpinning systems is evaluated. Since underpinning work may cause movement and stressing of the underpinned structures, and since this stressing is dependent chiefly on the construction procedures used in the excavation drifts and pits to remove the presently supporting soils beneath the existing structures, the construction and construction control procedures also have been evaluated.

The main auxiliary building is founded on the very stiff to hard natural clay soil, with foundation elevations ranging between 562 to 579 feet. The control tower (CT) and electrical penetration areas (EPA's), which are structurally connected to the southerly end of the main auxiliary building, presently are founded at elevation 609 feet on inadequately compacted plant fill varying up to 30 feet 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. At the extremeties of the EPA's, the feedwater isolation valve pits (FIVP's) are located and are founded on inadequately-compacted plant fill at elevation 615.5 feet. The FIVP's, 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 blowcounts in the plant fill at the auxiliary building area obtained during the late 1978 subsurface investigations caused concern for future differential settlements. Since the CT and EPA's were not designed to cantilever from the main auxiliary building, the differential settlements could cause unacceptable stresses. A one-foot deep void also was discovered in one of the borings beneath the mud mat under the control tower during the late 1978 investigations. Evidence of cracking at several locations on the auxiliary building were additional reasons for concerns.

To assure long-term foundation stability, the applicant has proposed to underpin the control tower and EPA's with a new permanent underpinning wall which will extend through the plant fill to the competent hard clay natural soil on which the main auxiliary building is also founded. 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.

Foundation treatment for the inadequate plant fill beneath the FIVP's consists of excavating the fill and a portion of the hard clay and replacing it with approximately 30 feet of compacted granular fill and 4 feet of concrete fill. The granular fill is to be compacted to 95% of maximum dry density as determined by ASTM test D1557 or ASTM test D2049, whichever test 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 procedures for controlling compaction which is acceptable to the staff and its consultants. 85%?

The granular fill and the concrete beneath the FIVP's 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 is completed. Subsequent settlements are anticipated to be minimal. Presently the FIVP's are temporarily supported by an overhead steel structure which is bolted to the existing concrete structure. The overhead structure transfers the load to the adjacent turbine building and buttress access shafts.

Underpinning details and foundation treatment of the FIVP are presented on Figs. 2.___ through 2.___ of this supplement (Source: Figures 2-1, 2-2, 2-3, and 2-5 of the Applicant's June 7, 1982 submittal).

An underpinning construction sequence and load transfer procedure was developed and reviewed which 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 control settlements and strain during underpinning, as described in subparagraph 2.5.4.6. In addition, a series of contingency plans have been developed (Specification C-200) which 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 underpinning design and the construction procedures, as well as the instrumentation to control underpinning, are 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 safety classification of these structuress.

Based on the documents submitted by the Applicant for modifying the foundations of the control tower, EPA's and FIVP's, the staff and its consultants conclude 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, if properly executed, will provide a stable and safe foundation.

Conditions at the northerly portion of the service water pump structure (SWPS) are similar to the conditions beneath the control tower and EPA's in that this portion is founded on the clay and sand plant fill and is structurally connected to the southerly part of SWPS which is founded on the deeper, more competent, very dense sandy clay till. The concerns for differential settlement between the shallower, northerly portion which overlies the plant fill and the southerly portion founded on till, along with unacceptable stresses has prompted the applicant to require a new permanent underpinning wall to assure 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 on Figure 2.__ of this supplement (Source: Figure SWP-26 in the Applicant's submittal dated December 31, 1981).

The proposed new permanent underpinning wall beneath the north portion of the SWPS will extend through the fill to at least elevation 587 feet which is the same bearing level as the existing deeper portion. Views of underpinning details are presented on Figures 2.__ and 2.__ of this supplement (Source: Figures SWP-14 and 15 of the Applicant's December 31, 1981 report).

An instrumentation system as described in subparagraph 2.5.4.6 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 (Specification C-200) to be implemented to limit further movements.

The sequence of construction and the procedure 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 and found to be suitable.

Based on the documents provided by the Applicant for underpinning the SWPS, the NRC staff and its consultants conclude that the underpinning fix is a conservative solution for eliminating the fill settlement problem and, if 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 elevation 628. The footings rest on approximately 25 feet of plant fill and were poured in October 1977. The structure is further described in Section 3.8 of this supplement. In July 1978, with the generator pedestals and approximately 60 percent 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 inches which already exceeded the building's initial 40 year settlement prediction of 2.8 inches.

The applicant temporarily halted construction of the DGB and completed a subsurface exploration program 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 its consultants, elected to surcharge the partially completed structure with 20 feet of sand placed above plant grade elevation 634. The sand fill was placed to approximately elevation 654 in each of the four interior bays of the DGB and extended horizontally at elevation 654 for a 20 foot distance around the east, south and west perimeter of the DGB. Along the north wall, where the DGB is adjacent to the turbine building, the 20 feet depth of sand extended for approximately 19 feet, 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 feet surcharge height in April 1979 when approximately 94 percent of the DGB structure was completed. The purpose of surcharging was to accelerate the settlement of the cohesive fill soils under a load that would produce vertical stresses at all depths in the fill in excess of those which would result during plant operation.

The applicant's consultants recommended removal of the sand surcharge in mid-August 1979 following their 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 inches, which resulted in a total settlement of 7.45 inches 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 due to 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 the 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 which did result, with the accompanying concerns for structural degradation (warping and cracking of the reinforced concrete - see Section 3.8 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 which will occur in the future beneath the DGB. This basic settlement data is essential for use in a structural analysis that evaluates the effects of these settlement stresses, in conjunction with other required load combinations in order to reach an engineering conclusion on the safe performance of the DGB.

Several piezometer and settlement readings recorded in the field during the time of surcharging raised reasonable doubts before the staff and its consultant 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 and its consultants requested additional explorations in the surcharged plant fill in order 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 and its consultant following our evaluation of the laboratory results is that the future settlements (time frame of 12/31/81 to 12/31/2025, FSAR Figure 2.5-127) identified by the applicant for use in their structural analysis of the DGB is sufficiently conservative. The future settlements identified cover the settlements which have been calculated for the more compressible zones of cohesive fill soils that were recovered in the NRC requested borings where attainment of 100 percent primary consolidation was shown not to have been achieved.

The staff and its consultant did not agree with the selection of settlement values, obtained prior to November 24, 1978, which were used by the applicant's consultants nor with the applicant's indicated status of construction which affected the flexibility of the integral footing and walls built prior to this time. These differences resulted from our 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 made additional analysis 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 as follows:

Time Frames	Case Study	Type of Study	Status of Construction
3/28/78 to 8/15/78 (Before surcharging)	1A	Hand Calculation	Top El 654 (East Wall) Top El 656.5 (South
8/15/78 to 1/5/79 (Before surcharging)	1B	Finite element - computer	Top El. 662
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 complete

We have 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 at the July, 1982 Design Audit. Our conclusions on 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 we are 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. We are in agreement with the identified settlements tabulated for Cases 1B, 2A and 2B. However, we do not agree that the straight line plot assumed to replace the actual measured settlements on each wall and used in the finite element analyses is appropriate or conservative. The staff plans to further evaluate the error bands of the surveyed settlements which are indicated by the applicant.

3. In our opinion the best information available is the measured settlements; the assumed straight line plot and the results of the finite element studies do not reasonably match these measured values. The stress levels 'n the DGB caused by the settlements and other required load combinations are discussed in Section 3.8 of this SSER, along with the plans of the staff and its consultants to assess the effects of the measured and future differential settlements.

2.5.4.4.3 Surcharging of the Borated Storage Tank Foundations

As discussed in SER Section 1.12.8, the foundations of the two borated water storage tanks (BWST) were constructed in July 1978 and in January 1979. The erection of the tanks were 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. Following the applicant's investigation, which indicated cracks in the ring beam of Unit 1 tank as wide as .063 inch and .035 inch for Unit 2 tank, the applicant concluded that the observed differential settlements had occurred because there were larger foundation areas beneath the valve pits which resulted in lower foundation pressures under the valve pits than beneath the ring wall foundations. The applicant further concluded that this nonuniform loading condition created the differential settlements and the localized areas of foundation overstress.

The staff does not agree with the applicant's conclusions as to cause. Based on the results of the soils investigations of the fill in the tank farm area, on the results of plate load tests and on the observed total and differential settlements which did occur, the staff concludes the behavior of the tank foundation is not indicative of a well compacted fill.

To correct the BWST foundation problem the applicant proposed three actions which included:

1. Surcharge the valve pits to reduce the amount of differential and future settlements. This action was completed by February 1982 over a four month period.

2. Integrally construct a new reinforced concrete ring beam around the periphery of the existing cracked ring.

3. Relevel the tank (Unit 1) which had experienced the largest settlements to the original construction tolerance.

Based on 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 which has been completed for both the valve pits and ring beam foundations. The future settlements which are estimated to occur during plant operation have been enveloped and addressed in the structural analysis for the new ring beams. For the above reasons, the staff and its consultant conclude that the BWST foundations are acceptable and will provide a stable and safe foundation.

Several remaining review issues are listed in Table 2.5 of this SSER for the BWST. These issues deal with the development of a long term settlement monitoring plan during plant operation, and FSAR documentation on the as-built conditions for the new ring beam foundations, and releveling operations which remain to be completed.

2.5.4.4.4 Permanent Dewatering

To eliminate concerns for 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 and 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 of this SSER.

2.5.4.4.5 Excavation and Backfill

In this Section the foundation treatment of the plant fill soils supporting the seismic Category I piping systems is described following a brief summary of the settlement problem.

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 completed to establish pipe deflection (settlement) profiles. The results of profiling indicated that the present pipe invert elevations have maximum deviations from 6 to 16 inches below the originally intended design invert elevations. The majority of these deviations are in the range of 9 to 11 inches. The allowable placement tolerances for installing the pipe in the field during construction was specified at plus or minus 2 inches from the established design invert elevations. Allowing for the lower tolerance of minus 2 inches during installation, which tolerances were reported to have been verified in the field, would indicate that pipe settlements of 4 to 14 inches 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 a future settlement for the piping system to be a maximum of 3 inches during the 40-year period of plant operation. The staff agrees that the estimated 3 inches maximum settlement is a conservative upper bound 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 which will include the control measures to be required in restricting placement of heavy loads over buried piping and conduits.

The applicant has also committed to providing its plan for addressing the staff's concern where underground piping and conduits are crossed by the freezewall. The freezewall is a temporary barrier to prevent groundwater from entering the underpinning excavations for the control tower, EPA's and FIVP. This concern has developed because of a modification to the originally proposed freezewall crossing design and which has the potential for creating differential settlements along the piping.

Some of the piping lines have already been relieved of stresses due to differential settlement by excavating down to the installed pipes, cutting the lines and then refitting the pipes. The extent of this completed work and also future planned rebedding work is shown on Figure 7 of the applicant's March 16, 1982 submittal. Figure 7 also shows the areal extent where excavation, pipe replacement and backfill for the 36-inch and 26-inch diameter service water pipes is to be completed just north of the SWPS and Circulating Water Intake Structure.

Excavation, rebedding and backfill for the 26-inch service water lines 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 SSE loading condition. If failure of the non-Category I permanent dewatering system were assumed, then there may not be sufficient time to either repair the dewatering system and/or shut the plant down because of the closeness of this problem area to the cooling pond, where recharge of the ground water has been demonstrated to occur rapidly (approximately within three days).

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

Concerns for differential settlement have been addressed by requiring the service water piping to be encircled with 6-inch thick polyethylene planks that are commercially named Ethafoam 220. The length of piping to be wrapped with this compressible product within the K-KRETE is 40 feet and spans between the portion of piping on the full depth of K-KRETE (to elevation 610) 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-inch differential settlement over this length has been assumed and shown to be tolerable.

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. Discussions on future settlement monitoring of underground piping is presented in Section 2.5.4.6 of this SSER.

The above discussion on excavation and backfill details for the 36-inch and 26-inch diameter service water pipelines is based on information presented by the applicant at the July 27-30, 1982 Design Audit. It is anticipated that 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.

Based on 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 if 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 on information received at the July 27-30, 1982 Design Audit. The staff anticipates that this information, particularly completion of Table 2-14, 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 which will occur will be on the very stiff to hard clay natural soils beneath the underpinned control tower and electrical penetration areas. The gross bearing pressures for these structures, respectively, are 15 KSF and 11 KSF 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 same structures and are 20.6 KSF (CT), 19.8 KSF (EPA) and 5.7 KSF (DGB), respectively.

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.

Based on our review of the information provided by the applicant in the FSAR and technical reports, including the design audit information, the staff and its consultants conclude that the resulting margins of safety against bearing capacity type failure are acceptable and sufficiently conservative.

2.5.4.5.2 Vertical Movement

Control Tower. The downward movement of the south end of the control tower relative to the south end of the spent fuel pool in the auxiliary building has been 0.24 inch during the period July 1978 through August 1981. Since the control tower was completed more than a year before settlement observations were begun, and since the largest settlements of the poorly compacted fill are likely to have occured early in the loading, it is reasonable to expect that differential settlements of 0.5 to 1.0 inch, or more, may have occurred from the beginning of loading to date.

Electrical Penetration Areas. The downward movement of the east end of the east EPA relative to the adjacent control tower has been 0.2 inch 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 EPA's for the period July 1978 to January 1982 has been 0.5 to 0.7 inch as shown on 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:

a. Maximum settlement of control tower relative to auxiliary building — 0.25 inch

b. Maximum settlement of auxiliary building relative to control tower — 0.25 inch

The staff and its consultants consider estimate a. above to be the most reasonable estimate and find it acceptable. 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 elevation 659 in the auxiliary building, after underpinning is complete, to strengthen that critical location.

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 inch. The total settlement of the SWPS has been about 3/8 inch and is shown on FSAR Figure 2E.1-27.

The fact that the differential settlement noted above for the SWPS is small indicates either (a) the poorly-compacted fill under the overhang has not settled significantly or (b) 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 inch.

The staff considers those estimates of differential settlements for the underpinned SWPS to be reasonable and acceptable.

Diesel Generator Building. The settlement history of the DGB and the staff and its consultant's evaluation of the settlement's impact are discussed in Subsection 2.5.4.4.2. The settlement history of the DGB is presented on FSAR Figures 2.E.1-6 through 2.E.1-12.

Borated Water Storage Tanks. The settlement history, the applicant's proposed remedial fix and the staff and its consultant's evaluation of future foundation stability of the BWST's are discussed in Subsection 2.5.4.4.3. The settlement history of the BWST's are shown on 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 inches and 2.3 inches for reactor units 1 and 2, respectively. These estimated settlements include a settlement of 0.6 inch resulting from lowering of the groundwater to Elevation 590 by the permanent dewatering system. As shown on FSAR Figure 2E.1-2, the average settlement actually recorded in the field up to January 1982 is approximately 0.75 inch for both reactor building units with the maximum settlement of 1.1 inch occurring beneath Unit 1.

The staff and its consultants consider the estimated settlements for the reactor containment buildings to be conservative and acceptable.

Underground Piping. The settlement of seismic Category I underground piping has previously been discussed in Subsection 2.5.4.4.5.

2.5.4.5.3 Strain and Horizontal Movements

There have been no measurements made of the horizontal movement of structures to date, but settlements that may take place while underpinning the control tower and EPA's 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 will be measured at critical locations.

The staff and its consultants consider the strain and horizontal movement monitoring program (locations, frequency of readings, etc.) which 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 elevation 634 were designed to rasist at- rest lateral earth pressures using the equivalent fluid pressure concept. The adopted design equivalent fluid unit weights are presented on Table 2.5-15 of the FSAR. 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 elevation 627 in combination with SSE loading.

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

The staff and its consultants conclude that the methods used to estimate lateral earth pressures on seismic Category 1 subsurface walls are conservative and acceptable and in accordance with current state of the art engineering practice.

2.5.4.5.5 Liquefaction Potential

In February 1978 the staff in its review of the Midland FSAR forwarded Request 362.2 to the applicant seeking documentation on the method which 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 PSAR. In its 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 which were drilled in August and September of 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. Based on the results of all completed exploration programs, including the later 1978 and 1979 standard penetration test 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 against liquefaction for the natural granular soils that the applicant has presented in FSAR Section 2.5.4.8 utilize the results of standard penetration test (SPT) blowcounts. On the basis of the high SPT values recorded in the natural soils in the extensive subsurface investigation programs which 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 same subsurface exploration program completed in late 1978 and early 1979, following discovery of the diesel generator building (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, electrical penetration areas, railroad bay, cantilevered portion of the service water pump structure and a portion of the service water piping.

In July 1979 the applicant reported the findings of its liquefaction studies using the results of the 1978 and 1979 explorations. In this study the applicant had adopted a peak ground surface acceleration of 0.12g, a groundwater level at elevation 627 (operating level of cooling pond) and conservatively adopted a Magnitude 7.5 earthquake for relating cyclic stress ratio causing liquefaction with 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 railroad bay area and was not a problem at the auxiliary building control tower area. In order to alleviate its concerns for liquefaction potential, the applicant ultimately chose to provide a permanent dewatering system which is discussed in Section 2.5.4.4.4.

In May 1980, the staff's consultant, the Corps of Engineers, concluded an independent liquefaction analysis using the Seed-Idriss simplified method. In the Corps study a groundwater level at elevation 610 was selected based on the applicants stated intention to maintain, by pumping, groundwater below this elevation, a Magnitude 6 earthquake and a peak ground surface acceleration of 0.19g. The results of the Corps study indicated that fill soils are safe against liquefaction for earthquakes that would produce a peak ground surface acceleration up to 0.19g if the groundwater was maintained below elevation 610. 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 elevation 610 are the diesel generator building area and the railroad bay area. The problem with loose granular backfill soils previously identified in other areas (electrical penetration areas, cantilevered portion of the service water pump structure and service water piping) is acceptably resolved by the proposed underpinning and 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 earthquakes with a peak ground surface acceleration up to 0.19g, which is more severe than the earthquakes used to establish the site-specific response spectrum at top of fill, provided the groundwater is maintained below elevation 610. SER section 2.4.6.2 discussed the permanent dewatering system and the staff's basis for reasonable assurance that the groundwater will be maintained below elevation 610 during plant operation.

2.5.4.5.6 Dynamic Loading

Section 2.5.2 of the SER provides the staff's assessment of the SSE and OBE design earthquakes to be used for the design of the Midland plant. The site-specific response spectrum approach was used and was independently checked by our consultant who utilized the SHAKE Computer Code one-dimensional wave propagation analysis to study possible local amplification effects on 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 resulted in the identification of a problem with the applicant's input into its wave propagation analysis where 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 spectrum approach. A major conclusion of our consultants 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 spectrum approach, is more conservative than the spectrum developed by application of the SHAKE results.

A check was made on the seismically induced settlements that were estimated by the applicant to range from 0.25 inch to 0.50 inch (Table 4-1A in Responses to NRC Requests Regarding Plant Fill). We conclude 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 1510 KSF for the plant fill in the shear strain range of 10^{-2} to 10^{-3} percent. We consider these values to be reasonable and acceptable for use in dynamic analysis and conclude that the applicant's decision to allow ±50 percent 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 prescribed for underpinning of the auxiliary building area and SWPS.

2.5.4.6.1.1 Measurements

(References describing the instruments, location and monitoring frequency are given for each type of measurement).

Auxiliary Building

a. Total and differential settlements of the control tower, EPA's, and FIVP's and total settlement of the auxiliary building. Drawings C1490 (6/21/82), C1491 (7/16/82), C1493 (7/16/82).

b. Differential horizontal movements between adjacent structures. Drawings C1490 (6/21/82), C1491 (7/16/82), C1493 (7/16/82).

c. Strains in concrete at critical locations. Drawings C1495 (5/21/82) and C1493 (5/21/82).

d. Settlement of some temporary and all permanent underpinning piers relative to superstructure, at top and bottom of piers. Figure 2.___ of this SSER (Source: Applicant's testimony of Nov. 1980, Fig. AUX 32).

e. Concrete stress in selected temporary and all permanent underpinning piers by means of Carlson stress meters near top and bottom. Fig. 2.___ of this SSER (Source: Applicant's testimony of Nov. 1980, Fig. AUX 32).

f. Crack mapping. (Jan. 25, 1982 submittal by Applicant).

g. Water levels in observation wells and piezometers. (Drawing SK-G-566 Rev. 1 (5/14/82) and Specification 7220-C-198 (1/18/82), as amended at June 25, 1982 meeting during conference call of July 1, 1982, and during Design Audit of July 27-30, 1982.)

h. Small-diameter test holes (between 1 in. and 4 in.) 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 ground water under pressure exists in 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. If water pressures are low, excavation to the bearing level will continue. If water pressures are shown to be high in the test holes, special dewatering (e.g., wellpoint or other suitable means) will be used to lower the water table at that pier to at least two feet below the bearing level. The hole beneath the final bearing level will be grouted. Although the available information indicates that the bearing stratum is a fairly homogeneous hard clay, it is possible that special pier dewatering will be needed. These holes will be used by the applicant as a conservative measure to confirm subsurface conditions before reaching the bearing level (Design Audit July 27-30, 1982).

i. Fines in discharge from dewatering wells. (Applicant's letter of April 22, 1982, p. 19. This reference deals with the SWPS. The same monitoring will be performed at the auxiliary building).

Service Water Pump Structure (SWPS)

a. Total settlements at six locations around the structures and differential settlement between the north end of the overhang and the portion now founded on till (Applicant's letter of April 19, 1982, p. III-9, Meeting, June 24-25, 1982; Design Audit July 27-30, 1982; Drwg Fig. SWPS-14 dated April 10, 1982).

b. Strain of the concrete at critical locations near the intersection between the overhang and the deep portion. (Applicant's letter of April 19, 1982, p. III-9; Drwg Fig. SWPS-14 dated April 10, 1982).

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

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

e. Length and width of existing cracks and of any new cracks that develop throughout the structure. (Applicant's letter of April 19, 1982, p. III-10).

f. Water levels in observation wells and in piezometers in the fill and in the sandy clay till. (Applicant's letter of April 22, 1982, Meeting June 25-26, 1982; Conference call July 1-2, 1982).

g. Fines in the dewatering wells discharge. (Applicant's letter of April 22, 1982, p. 19; Conference call, July 1-2, 1982).

2.5.4.6.1.2 Criteria

Acceptance Criteria for Auxiliary Building

The differential settlements between the southerly ends of the control tower and main auxiliary building, and between the southerly ends of the EPA's and the main auxiliary building will be used to control underpinning construction. Alert limits have been set at which the Applicant will begin a re-evaluation of the behavior of the structure. Also, action limits have been established at which the Applicant will implement contingency plans (Specification C-200) to minimize subsequent movements. Limits which were agreed to by both the staff and its consultants and the applicant at the July 27-30, 1982 Design Audit are as follows:

	Alert Limit in.	Action Limit in.
Phase 2 Construction	0.10	0.15
Phase 3 Construction		
Step 3.1	0.15	0.25
Subsequent Steps	0.15[1]	0.25[1]
Phase 4 Construction[3]	0.20[2]	0.40[2]

(1) These values may be raised to 0.20 and 0.30 in., respectively, if each extensometer on the structure shows a strain change smaller 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.

(2) 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.

(3) Phases of construction are shown on Drawing C-0101.

If the differential settlements shown in the above table reach 0.5 in., the Applicant will initiate discussions with NRC for consideration of and concurrence with future actions prior to implementing those actions.

After the full jacking load has been applied to the permanent underpinning for the EPA's and the CT, settlement will be monitored until it has been shown that secondary compression of the bearing stratum is occurring.

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

Seems out of place

Acceptance Criteria for SWPS

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 any extensometer:

Action Limit: This limit has not been established.

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.

Width of cracks:

Any new cracks exceeding 0.01 in. width and existing cracks exceeding 0.030 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 feet below the pier bottom, the groundwater level will be lowered a minimum of 2 feet below the bottom of the pier excavation.

Fines in well discharge:
Same as for Auxiliary building.

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

One pier will be load tested at the auxiliary building and one at the service water pump structure. An additional pier will be tested at the service water pump structure 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 test procedures have been reviewed and are acceptable to the staff. (Design Audit July 27-30, 1982; Meeting June 25-26, 1982, Telephone conference July 1-2, 1982, Applicant's submittal dated June 14, 1982.).

The monitoring programs and the technical and administrative procedures for evaluating and using the settlement and strain data during underpinning for both the auxiliary building and SWPS have been reviewed and are acceptable to the staff and its consultants.

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 1 underground piping. In this Section only the settlement monitoring program is covered. Strain monitoring is discussed in Section 3.9 .

The applicant in its March 16, 1982 submittal to the NRC (Enclosure 1, "Future Monitoring Program of Buried Service Water Piping for Midland Plant Units 1 and 2") 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 ten settlement marker locations.

We have reviewed the proposed locations and using the same selection criteria as the applicant have concluded that five additional markers are required. At the July 27-30, 1982 Design Audit the applicant agreed to install two additional markers on line 26"-OHBC-16, one on 26"-OHBC-55 and one on 26"-OHBC-54 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.

The applicant has committed to increasing the frequency of its settlement monitoring from the originally proposed rate of once every 90 days (March 16, 1982 submittal) to monthly readings for the first six months after markers have been installed. This increased frequency is intended to develop background and trends until readings have stabilized (0.10 inch settlement from previous monthly reading). If after six months the settlements have not stabilized, monthly readings are to continue until stabilization has been reached.

Based on our review of the information presented in the FSAR and technical reports and on the above applicant commitments, 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 1 piping.

2.5.4.6.3 Long Term Settlement Monitoring

The applicant has committed to providing a technical specification covering long term settlement monitoring during plant operation that is acceptable to the Staff. The technical specification is to be provided in the fall of 1982 and will include identification of total and differential settlement action and alert levels with remedial measures if these levels are reached. The settlement monitoring technical specification will be required to address all seismic Category 1 structures and piping systems.

2.5.4.7 Remaining Review Issues

The remaining OL safety review issues listed in Table 2.5 are primarily 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.

Table 2.5

Remaining Review Issues

Involved Structures	Review Issue
All seismic Category 1 structures and piping	Technical specification covering long term settlement monitoring
Control Tower, EPA's, SWPS BWST, 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.
Diesel Generator Building	Resolution of basis for error band on surveyed settlements and the assessment of the effects of the measured and future differential settlements (See SSER SEction 2.5.4.4.2).

2.5.4.8 Conclusions

In summary, based on our review of the information provided and identified in the preceding sections, the staff and its consultants conclude that the site and plant foundations, except for the diesel generator building, for which analyses are pending, are acceptable and will be adequate to safely support the seismic Category 1 structures, underground piping and conduits at the Midland Plant, Units 1 and 2.

This conclusion is subject to the satisfactory evaluation of the staff on the remaining review issues identified in Section 2.5.4.7.

August 10, 1982

For your comments
Shafer

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

MIDLAND PLANT -
REMEDIAL SOILS CONSTRUCTION RELEASE -
FILE 0485.16 SERIAL 18640

REFERENCES:
1) ACRS Summary Report dated April 19, 1982
2) ACRS Letter dated May 14, 1982
3) Memo from D S Hood, dated July 19, 1982, "Summary of June 25, 1982 Meeting on Soils Related Request for Information"
4) FSAR Amendment No 44 dated June 28, 1982

The Soils Remedial Project (which encompassed completed and planned remedial work, analysis of existing structures and those to be modified, and development of construction methods) has been reviewed in detail by the NRC. Numerous technical submittals, relating to the soils subject matter, have been docketed. A special summary report (Reference 1) was prepared, covering the entire scope of the remedial soils effort, for the ACRS subcommittee. The Soils Project was reviewed in detail by this subcommittee which concluded that no soils issues were unresolved as documented in Reference 2 above. A meeting (Reference 3) was held to discuss the information submitted in response to NRC's requests and to identify open items in the Draft SSER. Most of the technical issues were resolved in this meeting. The FSAR was amended in Reference 4 to include the remedial soils modifications. The NRR completed its review of the project during an audit and concluded that all technical issues had been resolved from July 27 - 30, 1982.

Enclosures 1 through 5 describe the remedial measures for the Auxiliary Building, Feedwater Isolation Valve Pits and Permanent Dewatering System, Service Water Pump Structure, Borated Water Storage Tank, Underground Utilities and Diesel Generator Building along with the respective acceptance criteria and commitments. Enclosures 6 and 7 are the reference documents

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which form the basis of NRR staff review of Phase II of the auxiliary building underpinning work and the references for the remaining soils remedial work.

In consideration of the thoroughness of NRC's review, the completion of the ACRS Subcommittee review, and the updating of the FSAR, Consumers Power Company requests an expedited construction release to allow implementation of the Remedial Work as described in the enclosures. This release, a critical path element, is needed immediately to maintain the construction schedule.

The Company and Region III recently signed a procedure for controlling detailed authorization of work activities by the Region. This procedure allows the Region to define the level of detail desired in granting specific concurrence for work activities. After NRC releases construction, the new Company/Region III procedure will provide added assurance that the NRC is maintaining necessary regulatory control over soils remedial work at the Midland Site.

ENCLOSURES

(1) Auxiliary Building, Feedwater Isolation Valve Pits (FIVP) and Permanent Dewatering System

(2) Service Water Pump Structure (SWPS)

(3) Borated Water Storage Tank (BWST)

(4) Underground Utilities

(5) Diesel Generator Building (DGB)

(6) Basis for the Staff Concurrence for Phase II

(7) References for Remaining Soils Remedial Work

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ENCLOSURE 1

AUXILIARY BUILDING, FEEDWATER ISOLATION VALVE PITS AND PERMANENT DEWATERING

Auxiliary Building Underpinning and Permanent Dewatering to include:

a. Excavation, including lagging and walers, of an access shaft below elevation of 609' at the east and west ends of the EPAs. (The access shaft to elevation of 609' was completed pursuant to letter to J W Cook from D G Eisenhut dated May 25, 1982).

b. Locating, drilling, installing and operating wells for a temporary construction dewatering system. Monitoring of the installed system is based on an acceptance criteria of limiting soils particles to 10 ppm on a 0.05 mm filter media. An informational sample will also be obtained using a 0.005 mm filter. Information from the informational sample will be available on site for review by the NRC. Permanent wells may also be used for the temporary construction dewatering system provided that the acceptance criteria is 10 ppm on a 0.005 mm filter.

c. Locating, drilling and installing dewatering wells and accessories, for operating a temporary freezewall system. Utilities crossing through the freezewall have been excavated to prevent frost heave affects and heave is being monitored. Backfilling of excavation under and around utilities will be based upon a report developed from actual heave readings. This report will be submitted to NRC for concurrence prior to beginning this backfill. NRR

d. Installation and operation of a temporary monitoring system to measure building conditions during underpinning activities. The monitoring system will include relative and actual displacement and strain instrumentation at critical locations. The acceptance criteria varies with the phase of construction. Attachment 1 to this enclosure provides numerical values for alert and action levels which are applicable during the underpinning work. Attachment 2 to this enclosure provides definitions for alert and action levels. The data obtained by this system is used to evaluate building conditions during the actual underpinning discussed in the activities in the following steps.

e. Excavating, installing steel sets and lagging drifts for access to the underpinning pier excavations.

f. Installation of Pier 11 West and associated pier load test. The load sequence will be 0% to 50% to 25% to 130% of the load corresponding to the bearing pressure allowed for the seismic loading combination. After the completion of the test, the load will be brought down to the design load.

g. Dig, reinforce, place pier instrumentation, place concrete and jack remaining temporary piers. Load transfer to the piers is to be completed and the jacks to be locked-off at the piers when the following criteria is met:

i) 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 inch for a continuous one hour period. When condition (i) is satisfied,

ii) 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 inch in a continuous 24 hour period. When this condition is satisfied, the pier will be locked-off.

When excavating in cohesionless (natural or fill) soils, the groundwater will be maintained 2 feet below the advance of excavation. In addition, a probing program will be employed in selected piers. These piers are E/W 4, E/W 7, E/W 10, E/W 12, CT 1, CT 6 and CT 12. The probing will be with a 4-inch maximum auger from 5 feet above to 5 feet below the design bearing elevation. If water is encountered while drilling, the stratum will be sufficiently dewatered to provide a stable bearing condition.

h. [Modifications to selected steel beam connections for beams supporting slabs adjacent to the Control Tower area to allow the relative displacements discussed in Item (d) above.] ?

i. Excavation for and installation of the Electrical Penetration Area (EPA) grillage beams. In areas where soil support of the EPA is critical (grillage at Piers 8 and 5), bulkheads are designed for "at rest" soil pressure to minimize loss of soil support. The grillage beams will be loaded while monitoring relative settlement of supporting piers and the EPA.

j. Installation of permanent load carrying piers under the control tower and loading. Load transfer to the piers will be complete and the jacks will be locked-off at the specified jacking load when the following criteria is met.

i) The pier will be loaded to 125% of the pier's 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 inch in a continuous one hour period. When condition (i) is satisfied,

ii) the pier load will be reduced to 110% of the pier's 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 inch for a continuous 24 hour period. When this condition is satisfied, the pier will be locked-off.

k. Removal of fill material under the EPA and Control Tower. Upon reaching established elevations, struts between the temporary piers in the turbine building and the containment building will be installed to provide lateral support to the temporary piers. Under the control tower, struts will be installed between the south Auxiliary Building wall and the permanent piers at the control tower south wall to provide lateral support while the permanent walls are being constructed and backfilled.

l. Removal of the existing temporary post-tensioning system from the EPA walls.

m. Construction of the permanent walls in lifts. Upon completion of the first lift under the EPA, struts will be installed between the containment building and the permanent wall and between the permanent wall and the temporary piers under the turbine building. These struts provide lateral support to the temporary piers during construction of the second lift which necessitates removal of the initial struts installed during general excavation. In the control tower, the completion of the first lift will be followed by installation of compacted backfill and installation of a concrete slab between the south wall of the Aux Building and the south control tower permanent wall to provide lateral support to the permanent wall.

n. Transfer of EPA load from the temporary piers under the turbine building to the permanent wall under the EPA and transfer of Control Tower load from the individual permanent piers to the integrated permanent wall. Load transfer to the wall will be complete; the jacks locked off, wedges and shims driven between the wall and the supported structure and grouted in place when all the following criteria are met:

   i. The jacking load for the permanent underpinning will be maintained at the specified value for at least 30 days and;

   ii. a semilogarithmic plot of settlement versus time approaches a straight line, and; Secondary consolidation ?

   iii. the settlement increment in the last 30 days of sustained load does not exceed 0.05 inches, and;

   iv. the settlement increment in the last 10 days of sustained load does not exceed 0.01 inches.

o. Installation of rock bolts, grouted in dowels and closure sections of the underpinning wall.

p. Installing a permanent support system for the FIVP consisting of a concrete jacking slab resting on controlled compacted fill over the till. The fines portion of the fill will be nonplastic as determined by hydrometer or Atterberg limits testing and inspection by Resident Geotechnical Engineer. The backfill will be properly moisture conditioned by soaking immediately prior to compaction. Compaction acceptance criteria will be 95% modified proctor or 85% relative density based on tests performed prior to placement. Equipment will be qualified by the test fill method. Also includes jacking of the FIVP against the jacking slab and filling of the gap between the existing foundation and the jacking slab by lean concrete.

grout?

q. Removal of the temporary grillage beam support system of the FIVP.

r. Crack monitoring during various underpinning stages of the critical areas. The critical areas include portions of slab at El 659'-0", portions of the shear walls near column rows 5.3 and 7.8, and portions of the intersection of EPA walls and the control tower wall, including the EPA wall, the control tower wall and the slabs at that intersection. The alert and action levels, as defined in Attachment 2 to this enclosure, for the crack widths at the critical locations are as follows:

alert: any new crack exceeding 10 mils or any crack exceeding 30 mils

action: any crack reaching 60 mils

TO WHAT CRITERIA

s. Backfill the completed permanent walls, tunnels, drifts and access shafts.

t. Repair, in the control tower and EPA, of accessible cracks 20 mils (0.020 inches) and larger in the accessible exterior walls and accessible interior walls below the permanent water table which exhibit weeping characteristics by epoxy injection. Application of sealant to accessible exterior concrete walls.

u. Locating, drilling, installing and operating a permanent site dewatering system including installation of piping, electrical and control systems. Monitoring of the installed system is based on acceptance criteria of limiting soils particles to 10 ppm on a 0.005 mm filter media.

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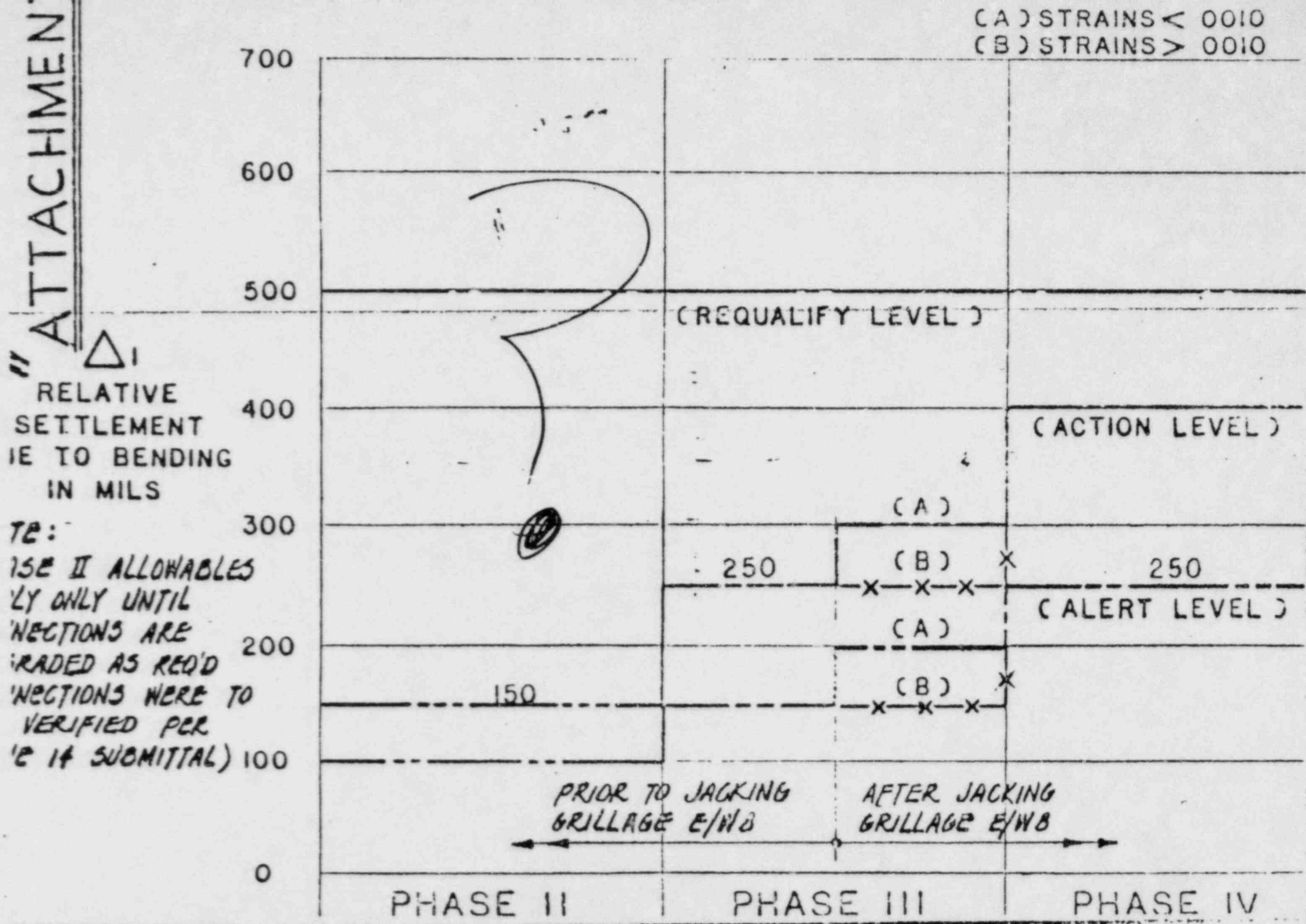
"ATTACHMENT-

SETTLEMENT MONITORING MATRIX



Alert Level*

All values up to the alert level are considered to be within normal working ranges.

Settlement readings should be reviewed by the resident structural engineer daily. In general, for readings below the alert level, attention should be focused on the value of the readings versus the construction progress and any indication of trends that would indicate the alert level may be exceeded.

Once the alert level is exceeded, the site resident engineer must inform engineering in Ann Arbor of the situation. The data including information from the other appropriate data mechanisms should be evaluated in total. Where trends exist that indicate the action level is likely to be reached, plans should be evaluated to remedy the situation. (Note: It is recognized that the evaluation may well conclude that no changes are warranted.)

Action Levels*

Values in excess of the action level must be reviewed by the resident structural engineer and as soon as possible by engineering in Ann Arbor.

Plans should be initiated to modify the condition that caused the settlement reading to exceed the action level. Consumers Power Company must be informed of the revised plan so that the NRC can be advised of the situation. The revised plan shall be intitiated immediately upon verbal notification by the resident structural engineer. (Note: It is recognized that the evaluation may well conclude that no changes are warranted.) If continuous movement beyond action level occurs, immediate action shall be taken.

Add "Requalify Level"
DSH

* - Crack width levels correspond to these definitions for Alert and Action.

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SERVICE WATER PUMP STRUCTURE

Service Water Pump Structure (SWPS) Underpinning to include:

a. Locating, drilling, installing and operating a temporary construction dewatering system. The water surface will be maintained 2 feet below the excavation if sand is present. Monitoring of the installed system is based on an acceptance criteria of limiting soils particles to 10 ppm on a 0.05 mm filter media. An informational sample will also be obtained using a 0.005 filter which will be available for on-site NRC review.

b. Locating, drilling and installing piles for excavation of access tunnel and adjacent underground utilities. Placing of concrete backfill.

c. Excavation and lagging for exterior access along the north and east face of the SWPS.

d. Excavation of interior access shaft along Circulating Water Pump Structure side of SWPS.

e. Installation and operation of a temporary monitoring system to measure building conditions during underpinning activities. The monitoring system will include both relative displacement and strain instrumentation. The acceptance criteria for the relative displacement instruments has been established as follows: Alert = 50 mils; Action = 70 mils. The acceptance criteria for the strain instrumentation are as follows: For 20-foot gage length - Alert = 0.0007, Action = 0.0014 and for the 5-foot gage length - Alert = 0.0001, Action = 0.0002. (See Attachment 1 to this enclosure for definition of Alert and Action limits.) The data obtained by this system is used to evaluate building conditions during the following steps.

f. Installation of Pier 1 East and associated pier load test. The load sequence will be 0% to 50% to 25% to 130% of the load corresponding to the bearing pressure allowed for the seismic loading combination. After the completion of the load test, the load will be brought down to the design load.

g. Dig, reinforce, place pier instrumentation, place concrete and jack remaining piers. These activities are to be controlled by appropriate procedures. Load transfer to the piers will be completed and the jacks will be locked-off when the following criteria is met:

   i) 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 inch for a continuous one hour period. When condition (i) is satisfied,

   ii) 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 inch in a continuous 24 hour period. When this condition is satisfied, the pier will be locked-off.

h. Removal of the existing temporary post-tensioning from the north-south exterior walls of the building.

i. Final load transfer to the completed underpinning system. The load transfer will be controlled by appropriate procedures. Load Transfer will be controlled by the following acceptance criteria:

   i. The jacking load for the permanent underpinning will be maintained at the specified value for at least 30 days and;

   ii. a semilogarithmic plot of settlement versus time must approaches a straight line, and;

   iii. the settlement increment in the last 30 days of sustained load will not exceed 0.05 inches, and;

   iv. the settlement increment in the last 10 days of sustained load will not exceed 0.01 inches.

j. Install anchor bolts, grouted in dowels, closure pits and sections of the underpinning.

WHAT CRITERIA?

k. Backfill and complete construction.

l. Crack monitoring, during various underpinning stages, of critical areas in the overhang portion of the SWPS resting on fill. The critical areas are the north-south walls of the overhang and the roof slabs at the junction of the overhang portion of the building with the deeper portion of the building. The alert and action levels, as defined in Attachment 1 to this enclosure for the crack widths in the critical areas are as follows:

   alert: any new crack exceeding 10 mils or any crack exceeding 30 mils

   action: any crack reaching 60 mils

m. Repair of exterior accessible cracks 20 mils (0.020 inches) and larger and interior cracks below the permanent water table which exhibit weeping characteristics by epoxy injection. Coating the splash zone of the exterior surface of the south wall with a waterproofing compound. Coating of accessible exterior walls with a sealant.

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Alert Level*

All values up to the alert level are considered to be within normal working ranges.

Settlement readings should be reviewed by the resident structural engineer daily. In general, for readings below the alert level, attention should be focused on the value of the readings versus the construction progress and any indication of trends that would indicate the alert level may be exceeded.

Once the alert level is exceeded, the site resident engineer must inform engineering in Ann Arbor of the situation. The data including information from the other appropriate data mechanisms should be evaluated in total. Where trends exist that indicate the action level is likely to be reached, plans should be evaluated to remedy the situation. (Note: It is recognized that the evaluation may well conclude that no changes are warranted.)

Action Levels*

Values in excess of the action level must be reviewed by the resident structural engineer and as soon as possible by engineering in Ann Arbor.

Plans should be initiated to modify the condition that caused the settlement reading to exceed the action level. Consumers Power Company must be informed of the revised plan so that the NRC can be advised of the situation. The revised plan shall be intitiated immediately upon verbal notification by the resident structural engineer. (Note: It is recognized that the evaluation may well conclude that no changes are warranted.) If continuous movement beyond action level occurs, immediate action shall be taken.

Requalify

* - Crack width and strain levels correspond o these definitions for Alert and Action.

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FIGURE 3

BORATED WATER STORAGE TANK

Borated Water Storage Tank (BWST) modifications to include:

a. Repair of cracks larger than 10 mils in the existing ring wall was completed pursuant to letter to J W Cook from D G Eisenht dated May 25, 1982.

b. Drilling holes for and installing new dowels in the existing ring wall and foundation.

c. Excavating for and installing the new ring beam around the existing ring wall. The installation shall include placing rebar and concrete for the new ring beam and additional lean concrete below the beam.

d. Releveling of the empty tank including jacking of tank after the anchor bolts are disconnected, leveling of the existing ring wall by grout, releveling of the oil sand layer below the tank bottom plate, and reattaching of the tank to the foundation by anchor bolts. Installation of strain gauges on the tank walls and bottom. WHERE? The releveling process will be stopped and the condition evaluated if measured stress (from strain data) exceeds 1.25 x anticipated stress level as shown below:

Location	Anticipated Stress
Bottom Plate	4328 psi
Tank Wall 1st stage lift	7000 psi
Tank Wall 2nd stage lift	6200 psi

e. Excavation for outside observation pits in the critical areas of the new ring beam (i.e., near the depth transition zones of the ring beam) to monitor cracks from outside. The new beam is to be monitored for cracks for six months after the tanks are filled initially with water in these critical areas. If any crack in these areas reaches 30 mils, the tanks shall be emptied and the condition evaluated. At the end of the six month period, an evaluation of the effect of cracks, in the critical areas, on the structural strength of the foundation shall be made. This evaluation will be submitted to the NRC Staff for review and concurrence.

f. Installation of 5-foot long extensometers, in critical areas, to monitor strain during construction, after completion of the new ring beam, and during plant operation.

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UNDERGROUND PIPING

Pipe Reinstallation to include:

a. Locating and installing piles for excavation of soil beneath the affected service water piping as referenced in Enclosure 2.

b. Temporarily supporting the rebedded piping in accordance with the design drawings and complete removal of the fill to the 610 foot elevation.

c. Refitting and welding the replaced and rebedded piping.

d. Place ethafoam around the service water pipe.

e. Backfill the excavation with a ~~proprietary~~ commercial product, "K-crete," which is a mixture of sand, cement and fly-ash, and release the temporary pipe support system.

all the way up?

Monitoring Stations Installation and Check-Out to include:

a. Locate field stations and anchor stations on the service water piping.

b. Excavate locally and install the strain gauges and settlement markers as required.

c. Wire and calibrate the vibrating strain gauge.

d. Perform a "check-out" test on the strain gauges and settlement markers. This provides baseline data for future monitoring.

e. Backfill and compact the local excavations.

How?

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DIESEL GENERATOR BUILDING

<u>Diesel Generator Building repair to include:</u>

a. Crack mapping to locate all accessible cracks 20 mils or larger in the interior and exterior surfaces of reinforced concrete structural walls and cracks below permanent water table in the interior surfaces of walls which exhibit weeping characteristics.

b. Repair of cracks identified in (a) and application of sealant to the wall.

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BASIS FOR STAFF CONCURRENCE FOR START OF PHASE 2*

1. Letter to R Vollmer from R T Hamilton, dated July 8, 1975, transmitting Bechtel quality assurance topical BQ-TOP-1, Revision 1A

2. Letter to H R Denton from J W Cook, dated September 30, 1982, submitting the Auxiliary Building Dynamic Model, Technical Report on Underpinning the Auxiliary Building and Feedwater Isolation Valve Pits

3. Letter to H R Denton from J W Cook, dated November 16, 1981, on Response to the NRC Staff Request for Additional Information Pertaining to the Proposed Underpinning of the Auxiliary Building and Feedwater Isolation Valve Pits

4. Hearing testimony by CPC witnesses (Johnson, Burke, Gould, Corley and Sozen) on remedial underpinning work for the Midland Auxiliary Building, November 19, 1981

5. Hearing testimony of D Hood, J Kane and H Singh concerning the Remedial Underpinning of the Auxiliary Building Area, dated November 20, 1981

6. Hearing testimony of F Rinaldi, dated November 20, 1981

7. Letter to H R Denton from J W cook, dated November 24, 1981 on Test Results, Auxiliary Building, Part 2, Soil Boring and Testing Program

8. Letter to H R Denton from J W Cook, dated December 3, 1981, with Addendum to Technical Report on Underpinning the Auxiliary Building and Feedwater Isolation Valve Pits

9. Letter to H R Denton from J W Cook, dated January 6, 1982, on Auxiliary Building Underpinning - Freezewall; Effects of Freezewall on Utilities and Structures

10. Letter to H Denton and J Keppler from J W Cook, dated January 7, 1982, transmitting general Quality Plan for Underpinning Activities and Quality Plans and Q-listed activities for SWPS and Auxiliary Building Underpinning

11. Design audits of January 18-20, 1982 (Summary dated March 10, 1982); February 1-5, 1982; March 16-19, 1982; and meeting of February 23-26, 1982, (Summary dated March 12, 1982)

12. Letter to H R Denton from J W Cook, dated February 4, 1982, on Auxiliary Building Access Shaft - Augering Method for Soldier Pile Holes

13. Letter to J W Cook from R L Tedesco, dated February 12, 1982, on Staff Concurrence for Activation of Freezewall

14. Letter to H R Denton from J W Cook, dated March 10, 1982, on Protection of Excavation Face - Auxiliary Building Underpinning Shaft

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15. Summary of March 8, 1982 Telephone Conversation regarding Soil Spring Stiffnesses for Auxiliary Building Underpinning and Phase II Construction, dated March 11, 1982

16. Letter to H R Denton from J W Cook, dated March 31, 1982, on Response to the NRC Staff Request for Additional Information Required for Completion of Staff review of Phases 2 and 3 of the Underpinning of the Auxiliary Building and Feedwater Isolation Valve Pits

17. Letter to J Keppler from J W Cook, dated April 5, 1982, describing Quality Assurance for Remedial Foundation Work

18. Letter to H Denton from J W Cook, dated April 26, 1982, transmitting quality assurance topical CPC-1-A, Revision 12

*Note: This is the same list as Enclosure 1 to the May 25, 1982 letter from Eisenhut to Cook, "Completion of Soils Remedial Activities Review."

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ENCLOSURE 7

REFERENCES FOR REMAINING REMEDIAL SOILS WORK

1. Letter to J W Cook from R L Tedesco, dated March 26, 1982, "Staff Concurrence for Grouting of Cracks in Concrete Foundation of BWST"

2. Letter to H R Denton from J W Cook, Serial 16597 dated March 31, 1982, "Response to the NRC Staff Request for Additional Information Required for Completion of Staff Review of Phases 2 and 3 of the Underpinning of the Auxiliary Building and FIVP"

3. Letter to J W Cook from R L Tedesco, dated April 2, 1982, "Staff Concurrence for Installation and Operation of Construction Dewatering and Observation Wells for the SWPS"

4. Letter to H R Denton from J W Cook, Serial 16629 dated April 19, 1982, "Summary of Soils-Related Issues at the Midland Nuclear Plant (ACRS)"

5. Letter to H R Denton from J W Cook, Serial 16656 dated April 22, 1982, "Response to NRC Staff Request for Additional Information Required for Completion of Staff Review of BWST and Underpinning of SWPS"

6. Letter to J G Keppler/H R Denton from J W Cook, Serial 16172 dated April 23, 1982, "BWST Foundation OL Design Calculations"

7. Letter to H R Denton from J W Cook, Serial 16884 dated April 30, 1982, "Effects of Cracks on Serviceability of Concrete Structures and Repair of Cracks"

8. Letter to H R Denton from J W Cook, Serial 17225 dated May 14, 1982, "Response to NRC Staff Request for Additional Information Required for Completion of Staff Review of Underpinning of Auxiliary Building"

9. Letter to J W Cook from D G Eisenhut, dated May 25, 1982, "Completion of Soils Remedial Activities Review"

10. Letter to J W Cook from E G Adensam, dated May 26, 1982, "Issuance of Amendments No 3 to Construction Permits - Midland Plant, Units 1 and 2"

11. Letter to H R Denton from J W Cook, Serial 17228 dated June 1, 1982, "Response to the NRC Staff Request for Settlement-Related Analyses for the Diesel Generator Building"

12. Letter to H R Denton from J W Cook, Serial 17304 dated June 7, 1982, "Additional Information on Soils Remedial Action at Midland"

13. Letter to J W Cook from R L Tedesco, dated June 11, 1982, "Transmittal of ACRS Interim Report"

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14. Letter to H R Denton from J W Cook, Serial 17319 dated June 14, 1982, "Response to NRC Staff Request for Additional Information Required for Completion of Staff Review of Soils Remedial Work"

15. Letter to H R Denton from R B DeWitt, Serial 17320 dated June 14, 1982, "Relationship of Observed Concrete Crack Widths and Spacing to Reinforcement Residual Stresses"

16. Memo from D S Hood, dated July 19, 1982, Summarizing the June 25, 1982, "Meeting on Soils Related Request for Information"

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UNITED STATES OF AMERICA
NUCLEAR REGULATORY COMMISSION

ATOMIC SAFETY AND LICENSING BOARD

Before Administrative Judges:
Charles Bechhoefer, Chairman
Dr. Frederick P. Cowan
Ralph S. Decker*
Dr. Jerry Harbour*

Paton / Hodgdon
Wilcove / Chandler
Rutberg
FF

In the Matter of CONSUMERS POWER COMPANY (Midland Plant, Units 1 and 2)	Docket Nos. 50-329 OM 50-330 OM Docket Nos. 50-329 OL 50-330 OL July 7, 1982

MEMORANDUM AND ORDER
(Reopening Record on QA Matters
and Establishing Schedule for
Prehearing Conference and Discovery)

1. By letter to the Licensing Board dated June 29, 1982, the NRC Staff confirmed earlier advice provided in a telephone conference call on April 28, 1982, that it was considering supplementing previous testimony by Region III in this proceeding. See Memorandum and Order dated April 28, 1982. The Staff advised that it has now determined that it is necessary to do so. The Board construed the Staff's letter as a request to reopen the record and on July 2, 1982 initiated a telephone conference call to discuss the ramifications of this request.

* Mr. Decker is a member of the Board for parts 1-3 of this Memorandum and Order. Dr. Harbour is a member of the Board for parts 4-6 of this Memorandum and Order.

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Participating in the call were:

Members of the Board (Messrs. Bechhoefer, Cowan, Decker and Harbour)
Mr. Michael Miller, for Consumers Power Co.
Mr. William Paton, for the NRC Staff
Ms. Barbara Stamiris, pro se
Mr. Wendell H. Marshall, pro se
(Ms. Mary Sinclair could not be reached. Ms. Stamiris agreed to inform Ms. Sinclair of the substance of the discussion.)

From the earlier (April 28) telephone discussion, the Board had been apprised that James Keppler, the Director of Region III, might wish to modify or supplement his earlier testimony that he had "reasonable assurance" that the construction QA program with respect to soils matters would be implemented satisfactorily. During the recent (July 2) conference call, we expressed the view that, if this were to be the case, we would have difficulty in issuing the partial initial decision on QA and management attitude matters prior to hearing Mr. Keppler's revised testimony. That decision could therefore not be issued during the time frame projected in our April 30, 1982 Memorandum and Order. LBP-82-35, 15 NRC ___, ___(slip op. p. 3).

The Applicant expressed its reluctance to have issuance of the first partial initial decision delayed, because of the time impact on matters remaining to be litigated in the OM and OL proceedings. It sought to have the first partial-initial decision cover all QA and management-attitude issues, with those issues affected by the forthcoming Staff testimony left open for possible modification.

Although we had agreed to follow the general course of action suggested by the Applicant with regard to other previously continued QA and management-attitude issues, none of those issues is so basic as the matters concerning which the Staff now wishes to supplement its earlier testimony. Moreover, the Staff attorney (Mr. Paton) advised that there were substantial differences of opinion between the Applicant and Staff regarding certain factual material, and he predicted as much as three weeks' hearings on these questions. Mr. Paton did not know the content of Mr. Keppler's revised testimony. He stated that, because of the open factual questions, Mr. Keppler's testimony could not be provided to the Board and parties prior to mid or late August. (The Staff agreed to inform the Board and parties as soon as it could do so of the date when it expects Mr. Keppler's testimony to be completed.)

The Board determined that the issues sought to be reopened by the Staff were too basic and too fundamental to the Board's decision to be treated as had the QA issues previously left open. We therefore granted the Staff's request to reopen the record and announced that we would defer our first partial initial decision until after we had heard the additional Staff testimony as well as any further testimony offered by the Applicant and other parties on the reopened QA issues.

2. The Board directed that the Staff's revised testimony be filed at least four weeks in advance of the reopened hearing and that the Applicant file responsive testimony two weeks after the Staff's filing. (Other parties may file testimony, if they wish, at the same time as the Applicant.) The Board asked that the Staff's testimony discuss in detail the bases for the Staff's position, including the changes, if any, from the Staff's earlier testimony. At the Applicant's suggestion, the Board asked

the Staff to have available at the reopened hearing not only Mr. Keppler but also any QA inspectors who might have more detailed knowledge of significant matters dealt with by Mr. Keppler, to the extent their presence might assist in creating an adequate record.

3. The Board pointed out that the reopened hearing sessions could also be utilized to complete consideration of a number of open questions, including the qualifications of QC inspectors, questions asked by the Board concerning the adequacy of the QA program for underpinning activities, and certain matters discussed in our April 30, 1982 Memorandum and Order. The Staff questioned whether the QA program for underpinning should be considered at the reopened QA hearings or at the time the corrective actions are considered. The Board left that question open during the conference call but has now determined that its questions concerning the QA program for underpinning should be heard at the reopened session. With respect to the other matters, all parties agreed, and we ruled, that they should also be considered at the reopened hearings.

We note that the matters discussed in our April 30 Memorandum and Order concerning which we seek additional testimony include the coverage of the QA program for soils-related activities, and activities covered by nonconformance reports NCR #M01-4-2-008 Rev. 1 (February 25, 1982); #M01-9-2-038 (March 8, 1982), and Memorandum from Darl Hood, dated March 16, 1982, "Notification of Loose Sands Beneath Service Water Piping." In addition, we would expect that testimony at the reopened hearings would cover such related matters as Staff Inspection Reports 82-05 (DETP) and 82-06 (DETP), NCR #M01-9-2-C51 (April 21, 1982), Bechtel nonconformance reports Nos. 4199 (including stop work order FSW-22) and 4245, and the suggestion in the interim ACRS report of June 8, 1982, that there be "a broader assessment of Midland's design adequacy and construction quality * * *." It will also be appropriate to put into the record the results of the Staff evaluation of Drawing 7220-C-45, which our Memorandum and Order of May 7, 1982, accepted on an interim basis, subject to Staff review, as defining the bounds of Q-listed fill. (Prior to the reopened hearing, the Board may direct the parties' attention to other soils-related QA/QC matters which should be considered).

4. During the conference call we also discussed with the parties the dates for responses to new contentions and to discovery in the OL proceeding. At the Staff's request we provided that the Staff's responses to Ms. Sinclair's new contentions are to be filed by July 21, 1982 and its responses to Ms. Stamiris' new contentions are to be filed by July 28, 1982. Ms. Stamiris sought to postpone the July 9, 1982 date we had fixed for her to furnish a statement of "good cause" for the late filing of her contentions and additional specification of her contentions. We declined to grant that extension and advised her that our "good cause" ruling would take into account whatever date she actually filed her statement. We added, however, that if she served her filing by express mail, she could file her statement as late as Monday, July 12 and that we would consider it in the same light as if she had filed it by regular mail on July 9. We explained that we would have to receive the filing by close of business Tuesday, July 13, because at least one Board member would not be available to receive and review it on a timely basis if it did not arrive by that date.

5. The Board granted Wendell H. Marshall an extension of time to July 26, 1982, within which to answer interrogatories propounded by the Applicant. We permitted the Staff to respond to Ms. Sinclair's discovery requests by July 28, 1982.

6. The Board tentatively established August 12-13, 1982 (and August 14, if necessary) as the dates for the prehearing conference in the OL proceeding. We asked the Applicant to arrange a site tour for the Board and parties, for Saturday, August 14, 1982. The conference will begin at 9:00 a.m. on August 12, 1982, at the Midland County Courthouse Auditorium, 301 W. Main, Midland, Michigan 58640.

For the foregoing reasons, it is, this 7th day of July, 1982

ORDERED

1. That the Staff's request to reopen the record on QA and management-attitude matters is hereby granted, with testimony to be filed and evidentiary hearings scheduled as provided in part 2 of this Memorandum and Order;

2. That the schedules previously adopted for filing responses to contentions in the OL proceeding, and for responding to discovery requests, are modified as provided in parts 4 and 5 of this Memorandum and Order; and

3. That a prehearing conference to consider proposed contentions in the OL proceeding is tentatively scheduled for August 12-13, 1982 (and August 14 if necessary) in Midland, Michigan.

FOR THE ATOMIC SAFETY AND
LICENSING BOARD

Charles Bechhoefer

Charles Bechhoefer, Chairman
ADMINISTRATIVE JUDGE