July 19, 1982

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Docket Nos. 50-329 OM, OL
and 50-330 OM, OL

Mr. J. W. Cook
Vice President
Consumers Power Company
1945 West Parnall Road
Jackson, Michigan 49201

Dear Mr. Cook:

Subject: Draft SSER No. 2 on Soils-Related Issues

Enclosed is a draft copy of the second supplement for the Midland SER. The primary purpose of this SSER, once published, will be to reflect completion of the staff's soils-related OL review. Although the draft is incomplete at this stage, it does identify several open issues to be resolved before this SSER reflects review completion. To this end, a meeting with members of your company has been scheduled for July 21, 1982, in Bethesda, Maryland.

This draft copy is preliminary at this time and does not reflect official staff approvals. Accordingly, no change in previous staff approvals should be inferred from this draft SSER.

Sincerely,

Darrell G. Eisenhut, Director

for/ Robert A. Purple, Acting Assistant
Director for Licensing
Division of Licensing

Enclosure:
Draft SSER No. 2

cc: See next page

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UNITED STATES
NUCLEAR REGULATORY COMMISSION
WASHINGTON, D. C. 20555

July 19, 1982

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and 50-330 OM, OL

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Sincerely,

fr
Robert A. Purple, Acting Assistant
Director for Licensing
Division of Licensing

Enclosure:
Draft SSER No. 2

cc: See next page

MIDLAND

July 19, 1982

Mr. J. W. Cook
Vice President
Consumers Power Company
1945 West Parnall Road
Jackson, Michigan 49201

cc: Michael I. Miller, Esq.
Ronald G. Zamarin, Esq.
Alan S. Farnell, Esq.
Isham, Lincoln & Beale
Suite 4200
1 First National Plaza
Chicago, Illinois 60603

James E. Brunner, Esq.
Consumers Power Company
212 West Michigan Avenue
Jackson, Michigan 49201

Ms. Mary Sinclair
5711 Summerset Drive
Midland, Michigan 48640

Stewart H. Freeman
Assistant Attorney General
State of Michigan Environmental
Protection Division
720 Law Building
Lansing, Michigan 48913

Mr. Wendell Marshall
Route 10
Midland, Michigan 48640

Mr. Roger W. Huston
Suite 220
7910 Woodmont Avenue
Bethesda, Maryland 20814

Mr. R. B. Borsum
Nuclear Power Generation Division
Babcock & Wilcox
7910 Woodmont Avenue, Suite 220
Bethesda, Maryland 20814

Cherry & Flynn
Suite 3700
Three First National Plaza
Chicago, Illinois 60602

Mr. Don van Farrowe, Chief
Division of Radiological Health
Department of Public Health
P.O. Box 33035
Lansing, Michigan 48909

William J. Scanlon, Esq.
2034 Pauline Boulevard
Ann Arbor, Michigan 48103

U.S. Nuclear Regulatory Commission
Resident Inspectors Office
Route 7
Midland, Michigan 48640

Ms. Barbara Stamiris
5795 N. River
Freeland, Michigan 48623

Mr. Paul A. Perry, Secretary
Consumers Power Company
212 W. Michigan Avenue
Jackson, Michigan 49201

Mr. Walt Apley
c/o Mr. Max Clausen
Battelle Pacific North West Labs (PNWL)
Battelle Blvd.
SIGMA IV Building
Richland, Washington 99352

Mr. I. Charak, Manager
NRC Assistance Project
Argonne National Laboratory
9700 South Cass Avenue
Argonne, Illinois 60439

James G. Keppler, Regional Administrator
U.S. Nuclear Regulatory Commission,
Region III
799 Roosevelt Road
Glen Ellyn, Illinois 60137

Mr. Steve Gadler
2120 Carter Avenue
St. Paul, Minnesota 55108

cc: Commander, Naval Surface Weapons Center
ATTN: P. C. Huang
White Oak
Silver Spring, Maryland 20910

Mr. L. J. Auge, Manager
Facility Design Engineering
Energy Technology Engineering Center
P.O. Box 1449
Canoga Park, California 91304

Mr. Neil Gehring
U.S. Corps of Engineers
NCEED - T
7th Floor
477 Michigan Avenue
Detroit, Michigan 48226

Charles Bechhoefer, Esq.
Atomic Safety & Licensing Board
U.S. Nuclear Regulatory Commission
Washington, D. C. 20555

Mr. Ralph S. Decker
Atomic Safety & Licensing Board
U.S. Nuclear Regulatory Commission
Washington, D. C. 20555

Dr. Frederick P. Cowan
Apt. B-125
6125 N. Verde Trail
Boca Raton, Florida 33433

Jerry Harbour, Esq.
Atomic Safety and Licensing Board
U.S. Nuclear Regulatory Commission
Washington, D. C. 20555

Geotechnical Engineers, Inc.
ATTN: Dr. Steve J. Poulos
1017 Main Street
Winchester, Massachusetts 01890

NUREG-0793
Supplement No. ~~1~~ 2

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

Docket Nos. 50-329 and 50-330

Consumers Power Company

U.S. Nuclear Regulatory
Commission

Office of Nuclear Reactor Regulation

August
~~June~~ 1982

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ABSTRACT

This report supplements the Safety Evaluation Report, NUREG-0793, issued May 1982 by the Office of Nuclear Reactor Regulation of the U.S. Nuclear Regulatory Commission with respect to the application filed by Consumers Power Company, as applicant and owner, for licenses to operate the Midland Plant, Units 1 and 2 (Docket Nos. 50-329 and 50-330). The facility is located in the city of Midland in Midland County, Michigan. This supplement provides recent information regarding resolution of some of the open items identified in the Safety Evaluation Report ~~and discusses recommendations of the Advisory Committee on Reactor Safeguards in its interim report dated June 8, 1982.~~ Most of the open items are associated with soil-related problems at the Midland site.

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Hydroligic Engineering

2.4.4 Flood Protection Requirements
[Later]

2.4.6.2 Design of Dewatering System
[Later]

2.4.6.4 Dewatering Monitoring Program
[Later]

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Midland Plant, Units 1 and 2

DRAFT Safety Evaluation Report Supplement

Geotechnical Engineering

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2.5.4.1.2 Site Foundation Description

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2.5.4.8 Conclusions

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~~Midland Plant, Units 1 and 2~~

~~Docket Numbers: 50-329/330~~

~~DRAFT Safety Evaluation Report Supplement - Geotechnical Engineering~~

~~Prepared By: Joseph D. Kane, HGEB, DE, NRR~~

2.5.4 Stability of Subsurface Materials and Foundations

In Section 2.5.4 of the SER ~~May 1982 Safety Evaluation Report~~, 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, ~~May 1982~~ 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 ~~May 1982~~ 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 Section 2.5.4 review has been conducted, the ~~May 1982~~ SER also ~~provides~~ discussed ~~discussions of~~ the following ~~important~~ topics related to the plant fill settlement problem:

a. Discovery of the plant fill deficiencies - Section 1.12

b. Affected safety related structures and utilities - Section 1.12 and Table 2.2

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and a related Licensing Board Order

c. NRC issuance of the Order Modifying Construction Permits and a related Licensing Board Order - 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 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. 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 is typical of a glaciated plain. A loose to very dense, ~~surface~~ 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 feet. 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.

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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 ~~8~~ 6 ft thick reinforced concrete mat
Service Water Pump Structure (deeper portion)	Very stiff to hard 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	___ ft thick reinforced concrete mat
Service water pump structure	Plant fill	617(1)	3 ft thick reinforced(1) concrete mat

Structure	Supporting Foundation Soil	Original Foundation Elevation	Original Foundation Type
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(3a) pads
Borated water storage tanks	Plant fill	629	Continuous reinforced(3a and 4) concrete ring wall on 1.5 ft thick by 4 ft wide footings.

Notes:

(1) To be modified with permanent underpinning wall.

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

(3) Subjected to surcharging with sand fill.

(3a) Tanks filled with water.

(4) New ring wall foundation to be constructed. Resetting for Unit 1 tank.

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

2.5.4.1.3 Site Investigations

* [Input into the final SSER will include our summary of the sub-surface investigations that have been completed at the Midland site (e.g., number of borings and exploratory investigations, type of drilling and sampling, geophysical investigations, etc.). Pertinent references and figures will be cited.

The staff evaluation will con~~d~~clude that the site investigations are acceptable and adequate in identifying the important subsurface features and foundation conditions and they 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

[Input into the final SSER will describe the laboratory and field testing that was completed (e.g., scope, types of testing, etc.) and the range in results of significant soil properties (density, permeability, shear strength, compressibility characteristics, shear wave veolcities) under both static and dynamic loading. These properties will be related to the specific foundation

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* Text within brackets are temporary for purposes of this draft input.

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layering described in section 2.5.4.1.1. Pertinent references and figures that provide greater details on the test results will be given.

The staff evaluation will 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".]

2.5.4.3 Foundation Profiles and Design Properties

[Input into the final SSER will include a staff evaluation of the pertinent soil profiles and sectional views that present the results of the subsurface investigations in relation to the final horizontal and vertical locations of all Category I structures and utilities. The important static and dynamic soil properties adopted in plant design will be discussed and related to the soil profiles.

The staff evaluation will conclude that the soil profiles and sectional views are adequate and acceptable in correctly representing the results of the subsurface investigations and that the adopted design properties are reasonable.]

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

The main auxiliary building is founded on the very stiff to hard clay natural soil with foundations ranging between elevations 562 to 579 feet. Beyond the main building at the southerly portion, the control tower and electrical penetration areas (EPA's), which are structurally connected to the main auxiliary building, are founded at elevation 609 feet on inadequately compacted plant fill varying up to 30 feet in thickness. Large volumes of concrete used as a replacement for structural backfill in the excavations for the deeper auxiliary building and reactor buildings 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 plant fill at elevation 615.5 feet. The FIVP's are structurally separated from other buildings but they do house a seismic Category I piping that penetrates several structures. A soil profile view depicting the pertinent foundation information is presented on Figure 2._ of this supplement (Source: Figure AUX-38 of the applicant's November 19, 1981 hearing testimony) ~~before the ASLB~~.

The low SPT blowcounts indicated at the auxiliary building area in the plant fill in the late 1978 subsurface investigations caused concern for future differential settlements. Since the control tower and EPA's were not designed to cantilever from the main auxiliary building, the differential settlements could potentially cause structural stresses higher than allowable values, particularly if the structures were subjected to other ~~higher~~ stresses required by design load combinations. A one-foot deep void had also been discovered in one of the borings beneath the mud mat under the control tower in the late 1978 investigations. Evidence of cracking at several locations on the auxiliary building were additional reasons for concern.

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 fill to the competent hard clay natural soil on which the main auxiliary building is also founded. The permanent underpinning wall will ultimately be connected to the bottom of the existing mat foundations after jacking of the structure loads has been held long enough on the permanent wall to reduce future settlements to minimal values.

Foundation treatment for the inadequate plant fill beneath the FIVP's consists of excavating the fill and an upper portion of the hard clay and replacing it with approximately 30 feet of compacted granular fill and 4 feet of concrete fill. The two fills will be separated by a jacking slab that will be used to remove the load of the FIVP structures from the existing temporary supports and into the granular fill. This procedure will allow the major settlement of the granular fill to occur while the jacks are in place and before transfer of the final load to the permanent foundation is completed. By performing this procedure, future settlement values are anticipated to be minimal. Presently the FIVP's are temporarily supported by an overhead steel structure assembly with bolting to the existing concrete structure. ~~that~~ The overhead assembly transfers the load to the adjacent turbine building and buttress access shafts. Underpinning details and foundation treatment of the FIVP are presented on Figures 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).

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 permanent underpinning wall fix is an acceptable solution for eliminating the plant fill problem in the auxiliary building area and, if properly carried out in the field, will provide a stable and safe foundation.

Several remaining review issues related to underpinning in the auxiliary building area are listed in Table 2.3 ~~following section 2.5.4.7~~ of this supplement. We consider these issues to be related to resolution of final design details, fulfillment in the field of important construction controls and FSAR documentation that is required to confirm actual as-built conditions.

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 also 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 deeper clay till supported portion along with the inducement of unacceptable structural stresses into this very rigid structure, 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 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

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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).

Based on the documents provided by the applicant for underpinning the SWPS, the NRC staff and its consultants conclude that the underpinning fix is an acceptable solution for eliminating the fill settlement problem and, if properly carried out in the field, will provide a stable and safe foundation.

The remaining review issues related to the SWPS are summarized in ~~the~~ Table 2.3 ~~provided in Section 2.5.4.7~~ of this supplement.

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. The structure is further described in Section 3.8.3.4 of this supplement. In July 1978, with the generator pedestals and approximately 60 percent of the DGB completed, field settlement measurements 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. ~~and~~ 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, selected 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 ~~cells~~ bays of the DGB and extended horizontally for ~~approximately~~ a 20 foot ~~horizontal~~ distance around the ~~entire~~ east, south and west perimeter of the DGB. Along the north wall, where the DGB is close to the turbine building, the 20 feet depth of sand extended for 19 feet, 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 ~~[illegible]~~ 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 Figures 2.__ through 2.__ of this supplement (Source: Figures 27-10 through 27-13 of the applicant's response to NRC requests regarding plant fill, question number 27).

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 ~~covered~~ discussed in the ~~following~~ Sections 2.5.4.4 and 2.5.4.5 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

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for structural damage (warping) and stress inducement, including cracking of the reinforced concrete which are discussed in ~~other~~ section~~s~~ 3.8 of this SSER. The major objective of the NRC geotechnical engineering staff and its consultants with respect to the adequacy of the DGB 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 operation~~ safe ~~expected~~ 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 full 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. ~~After~~

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~~considerable delay which was caused by the applicant's appeal of this staff request for explorations and laboratory testing,~~ 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 the future settlements being adopted by the applicant for use in their structural analysis of the DGB is sufficiently conservative. The future settlements being used 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 values of future settlement for the DGB which are acceptable to the staff are correctly presented on Figure 2.__ of this SSER, which corresponds to Figure 1-3 in the applicant's June 1, 1982 submittal ~~which is~~ entitled "Structural Stresses Induced by Differential Settlement of the Diesel Generator Building" for the post surcharge period. In this same Figure 1-3 the applicant has incorrectly indicated the settlements to be used for the presurcharge period in the structural analysis. The correct presurcharge settlement values are given on Figure 2.__ of this SSER (Source: Figure 27-10 in "Responses to NRC Requests Regarding Plant Fill") and should be used in the required structural analysis. Evaluation of the success of the DGB surcharge program is very much dependent on the final results of the structural analysis presented in the

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June 1, 1982 submittal and which is discussed in Section 3.8 of this SSER. The staff does not agree with the applicant's conclusion that the DGB had high structural flexibility prior to November 24, 1978 because the applicant has failed to allow for the rigid 30 inch thick concrete walls which were completed to elevation 654 prior to this time in its structural analysis. In addition, the staff does not find the settlement data analysis presented as attachments to the June 1, 1982 submittal to be acceptable or meaningful because very important settlement records prior to November 24, 1978 were not considered in the settlement data analysis.

The staff ~~recommends~~ requires that the settlements listed on Figure 2.~~1~~ of this SSER (Source: Figure 1-3 of the June 1, 1982 submittal), after correcting for the pre-surcharge period values as previously indicated, ~~be required to~~ be properly addressed in the structural analysis of the DGB.

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 ~~surcharged~~ (filled the tanks with water) ~~the foundations~~ in October 1980 and monitored the resulting foundation settlements.

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In January 1981, the applicant reported differential settlements between the ring wall foundations and the outside portions of the valve pits ~~following the surcharging~~. 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 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.

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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 acceptably 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 ~~the~~ Table 2.3 ~~of~~ of this SSER ~~section 2.5.4.7~~ for the BWST. These issues deal with the development of a long term settlement monitoring plan during ~~years of~~ plant operation, and FSAR documentation on the as-built conditions for the new ring beam foundations, and releveling operations which remain to be completed.

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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 ~~covered~~ in section 2.5.4.5.5 and the staff's evaluation of the proposed permanent dewatering system was ~~is~~ presented in ~~the May 1982~~ SER ~~in~~ Section 2.4.6.2.

The remaining review issues on permanent dewatering are primarily involved with resolution of OL Technical Specification details and are listed on Table 2.3 of This SSER. ~~section 2.5.4.7.~~

2.5.4.4.5 Excavation and Backfill

~~The same foundation treatment fix which has been previously discussed for the FIVP~~ Excavation and replacement with backfill will also be completed beneath seismic Category I piping where loose granular foundation fill soils susceptible to liquefaction have been shown to be present.

[The staff's evaluation of previously submitted reports on underground piping is ~~are~~ not completed.

The issues remaining in geotechnical engineering related to underground piping are ~~as~~ listed in Table 2.3 of this SSER ~~in section 2.5.4.7~~, and are concerned with the adequacy of the reinstallation program for the 26 inch diameter and 36 inch diameter service water piping (excavation and backfill details of foundation support), the long term settlement and strain monitoring programs and FSAR documentation of as-built conditions.]

2.5.4.5 Foundation Stability

2.5.4.5.1 Bearing Capacity

[Input into the final SSER will cover the range of applied bearing pressures (static and dynamic loading) and be related to previously identified foundation layering. The results of computations establishing factors of safety will be provided.

The staff evaluation will conclude that the resulting margins of safety against bearing type failure are acceptable to the staff and are equal to values found acceptable in conservative engineering practice.]

2.5.4.5.2 Vertical Movement

[Input into the final SSER will summarize the settlement history of the important seismic Category I structures and utilities.] The following paragraphs cover only the auxiliary building and service water pump structure.

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 a year before settlement observations were begun, and since the largest settlements of the poorly compacted fill are likely to occur early in the loading, it is reasonable to expect that differential settlements of 0.5 to 1.0 inch, or more, may have occurred to date.

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 settlement of the control tower and the EPA's for the period July 1978 to August 1981 has been 0.5 to 0.7 inch.

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

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

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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 reasonable estimate and finds 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 stresses greater than allowable stresses. To accomplish this limit on stresses, steel plates are to be added to the slab at elevation 659 in the auxiliary building to strengthen that critical location.

The maximum measured differential settlement of the overhang of the SWPS relative to the portion founded on till has been about 0.1 inch. The settlement observations were begun in May 1977, immediately after the foundation mat for the overhang had been placed. Thus, these measurements represent all of the settlement that has occurred.

The total settlement of the SWPS has been about 3/8 inch since May 1977.

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

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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 these estimates of differential settlements For the underpinned ~~AB~~ SWPS to be reasonable and acceptable.

2.5.4.5.3 Horizontal Movement

There have been no measurements made of horizontal movement 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. Strain monitoring instruments are being installed to measure potential horizontal movements between all adjoining structures during underpinning. In addition, horizontal strains that may develop in the SWPS will be measured at critical locations. The staff and its consultants consider the strain monitoring program (locations, frequency of readings, etc.) which has been proposed during underpinning operations by the applicant to be acceptable, however, agreement on acceptable allowable strain limits has not been reached.

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A permanent program for monitoring horizontal movements during [redacted] plant operation has not been provided by the applicant.

2.5.4.5.4 Lateral Loads

Input into the final SSER will describe the computed earth pressures under both static and dynamic loading and design methods will be cited. Pertinent references and figures will be identified.

The staff is essentially in agreement with the applicant on design of lateral loads but the staff needs to complete its review of recently furnished sliding resistance and lateral soil pressure calculations for the SWPS under dynamic loading.

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

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(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

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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 ratios 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 selected the permanent dewatering fix.

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 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

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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 that require properly compacted soils.

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 ~~[illegible] confidence~~ reasonable assurance that the groundwater will be maintained below elevation 610 during plant operation.

2.5.4.5.6 Dynamic Loading

[Input into the final SSER will summarize the geotechnical engineering review efforts and SHAKE computer code studies that were completed to independently evaluate the reasonableness of the site-specific response spectrum for the top of plant fill. Pertinent reports by consultants will be referenced.]

2.5.4.6 Instrumentation and Monitoring

The following monitoring measurements are to be made during underpinning of the auxiliary building area and SWPS. 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 (2/3/82), C1491 (2/3/82), C1493 (5/21/82).
b. Differential horizontal movements between adjacent structures. Drawings C1490 (2/3/82), C1491 (2/3/82), C1493 (5/21/82).
c. Strains in concrete at critical locations. Drawings C1495 (5/21/82) and C1493 (5/21/82).

d. Settlement of all temporary and permanent underpinning piers relative to the superstructure, at top and bottom of piers. Figure 7.__ of this SSER (source: applicant's testimony of ~~(ASLB.~~ Nov. 1980, Fig. AUX 32).

e. Concrete stress in temporary and permanent underpinning piers by means of Carlson stress cells near top and bottom. (ditto ~~(ASLB,~~ 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 722C-C-198 (1/18/82). [Documentation of revisions as agreed upon at June 25, 1982 meeting and in conference call of July 1, 1982 are to be provided by the applicant.]

h. Fines in discharge from dewatering wells. (applicant's letter of April 22, 1982, p. 19). Although this reference deals with the SWPS, this same monitoring will be performed at the auxiliary building, as agreed during conference call of July 1, 1982.

SWPS

a. Total settlements at four locations around the structure and differential settlement between the north end of the overhang and the portion now founded on till (*April 19, 1982, p. III-9; Meeting, June 24-25, 1982).

b. Strain of the concrete near the roof level at the interaction between the overhang and the deep portion. (*April 19, 1982, p. III-9).

* Applicant's letter of

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

d. Concrete-stress levels within the underpinning piers near the top and bottom. *(April 19, 1982, p. III-10).

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

f. Water levels in observation wells and in piezometers in the sandy clay till. *(April 22, 1982; Conference call July 1-2, 1982).

g. Fines in the dewatering wells discharge. *(April 22, 1982, p. 19; Conference call, July 1-2, 1982).

The differential settlements between the control tower and main auxiliary building, and between the EPA's and the main auxiliary building will be used to control underpinning construction. A trigger limit will be set at which the applicant will begin a re-evaluation of the behavior of the structure. Also, a stop limit will be established at which the applicant will stop underpinning, shore up the drifts temporarily, evaluate the behavior of the structure, and alter the construction technique, if necessary, before proceeding. These limits [have not been agreed upon but currently] are as follows for the southerly end of the control tower:

* Applicant's letter of

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	Trigger Limit	Stop Limit
NRC Geotechnical staff	0.1 in.	0.15 in.
Applicant	0.35 in.	0.7 in.

Strain gages at the auxiliary building will be used at two critical zones to monitor the strains in the concrete and to estimate the changes in stress in the reinforcing steel during underpinning. The applicant has proposed that these strains not be used to control construction but that the differential settlements alone be used. The applicant has proposed use of a strain of 0.0014 as a stop limit during underpinning. [The staff has not yet formulated a final position on this proposal.]

With respect to underpinning the SWPS, the following limits and actions to be taken have been established:

Differential Settlement (Meeting, June 24-25, 1982):

Trigger limit: 0.05 in.

Stop limit: 0.07 in.

Strain in Concrete:

[To be resolved during audit.]

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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. (12/31/81, p. 50).

Width of Cracks:

Any new cracks exceeding 0.01 in. width and existing cracks exceeding 0.03 in. width will be evaluated to determine whether underpinning should stop or continue (12/31/81, p. 50).

Water Levels:

Water levels will be monitored to ensure that the ground water 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 will be made by continuous sampling in the six borings for the observation wells. At locations where such pervious strata exist within 2 feet below the pier bottom, the groundwater level will be lowered a minimum of 2 feet below the bottom of the pier excavation. (Meeting, June 24-25, 1982; Conference calls, July 1-2, 1982).

The monitoring programs proposed during underpinning for both the auxiliary building and SWPS are acceptable to the staff. The number of instruments is large and care must be taken to ensure that the significant measurements are interpreted by the applicant on a timely basis.

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The applicant has indicated that information on long term settlement monitoring during ~~years of~~ plant ~~information~~ operation, with action levels and remedial measures identified, will be provided to NRC in a proposed Technical Specification ~~proposal~~ in the fall of 1982.

[2.5.4.7 Remaining Issues

The ~~following~~ OL safety review isssues listed on table 2.3 remain outstanding.] ~~All the listed issues have previously been forwarded to the applicant.~~

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Table 2.3 Remaining Issues

Structure	Issue	Anticipated Method of Resolution
Auxiliary building (Control tower, EPA's and FIVP's)	Resolution of allowable vertical differential settlement and strain that will stop under-pinning construction and require installation of temporary supports.	Meeting with applicant
	Compaction control specification for granular fill beneath FIVP's.	Future applicant submittal
	Procedure for transer-ring final loads to permanent underpinning wall.	Design audit
	Updated construction sequence for Phases 3 and 4.	Future applicanct submittal

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Structure	Issue	Anticipated Method of Resolution
	Resolution of pier and plate load test details on maximum test load, locations and time for performing test.	Meeting with applicant
	Long term settlement and strain monitoring plan during plant operation	Technical specification proposal by applicant (Fall of 1982)
	FSAR documentation on as-built conditions	Future applicant submittal (Following construction completion)
	Design modification at freezewall crossing with duct banks	Future applicant submittal
	Resolution of required depths of construction dewatering walls	Meeting with applicant

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Structure	Issue	Anticipated Method of Resolution
Service Water Pump Structure	Complete staff review of sliding and lateral soil pressure calculations under dynamic loading	Meeting with applicant
	Resolution of pier and plate load test details on maximum test load, locations, and time for performing test	Meeting with applicant
	Resolution of required depths of construction dewatering wells	Future applicant submittal
	Procedure for transferring loads from jacks to permanent wall and locking off	Design audit
	Long term settlement and strain monitoring plan during plant operation	Technical specification proposal by applicant (Fall of 1982)

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Structure	Issue	Anticipated Method of Resolution
	FSAR documentation on as-built conditions	Future applicant submittal (Following construction completion)
Borated Water Storage Tank	Long term settlement monitoring plan during plant operation	Technical Specification Proposal by applicant (Fall of 1982)
	FSAR documentation on as-built conditions (New ring beam and releveling)	Future applicant submittal (Following construction completion)
Underground Piping	Complete staff review of applicant's submittal on proposed reinstallation of 26-inch 36-inch diameter pipes and long term settlement and strain monitoring programs	Meeting with applicant

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Structure	Issue	Anticipated Method of Resolution
	Plant control restricting placement of heavy loads over buried piping and conduits	Future technical specification proposal by applicant
	FSAR documentation on as-built conditions (Reinstallation and monitoring)	Future applicant submittal (Following construction completion)
	Long term settlement and strain monitoring plan during plant operation	Technical specification proposal by applicant.
Diesel Generator Building	Completion of analysis that uses correct settlement values and structure rigidity. Documentation of results with comparison to recorded and predicted settlements	Future applicant submittal
	Long term settlement monitoring plan during plant operation	Technical specification proposal by applicant (Fall of 1982)

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Structure	Issue	Anticipated Method of Resolution
Permanent Dewatering	Resolve availability of 60 day period in view of recharge rate in wells in railroad bay area	Meeting with applicant
	Requirements on permanent dewatering system during plant operation	Technical specification proposal by applicant
Miscellaneous	Long term settlement monitoring plans during plant operation for all structures not previously identified in table	Technical specification proposal by applicant (Fall of 1982)

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2.5.4.8 Conclusions

[Where possible, the staff's conclusion on acceptability of sumitted information has been given. Final overall conclusion on plant safety requires resolution of remaining issues.]

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3.7.1 Seismic Input

The applicant has not completed his evaluation of the seismic Category I structures necessary for shutdown and continued heat removal to determine seismic safety margins resulting from application of site-specific spectra. In addition, the applicant plans to revise the criteria on damping values for cable trays, conduits, piping, tubing and their supports.

Upon completion of the staff's review of these evaluations, an additional supplement to the safety evaluation report will be issued.

3.7.2 Seismic Analysis

Further discussion of the results of the Seismic Safety Margins Evaluation and the request for increase of Damping Values for cable trays, conduits, piping, tubing and their supports will be provided in a future supplement, as discussed in Section 3.7.1.

The applicant was requested by the staff to determine that 1.5 x FSAR seismic response spectra analyses are conservative for the auxiliary building, SWPS, DGB and BWST in comparison to requirements imposed by the use of the site spectific response spectra. The staff has indicated that a comparison of the floor response spectra for each of the two criteria (1.5 x FSAR and Site Specific Response Spectra) could provide such determination. The applicant has provided in his responses a conclusion stating that, "the 1.5 x FSAR response spectra analysis is conservative for the auxiliary building and SWPS underpinnings, and the BWST foundation." However, the applicant has not provided the comparative displays requested by the staff and has limited this evaluation to the DGB, the BWST foundations, and the underpinnings for the auxiliary building and SWPS. The applicant also plans to evaluate the above structures in his Seismic Safety Margins Evaluation. [The staff plans to review the information on the underpinning for the auxiliary building and the SWPS, the DGB and the foundation for the BWST during an audit planned for July 27-30, 1982.] The review of the Seismic Safety Margins Evaluation will be scheduled after the docketing of this information.

Also, the applicant has provided a report that confirms the fact that the techniques used to calculate soil springs are adequate. However, the staff requires that the three peaks in floor response spectra resulting from a variation of +30% of the soil stiffness should be enveloped. The applicant has provided this information as part of Revision 44 to the FSAR. In addition, in his (date) reply to Request 2.8 from Enclosure 8 of the staff's letter of May 25, 1982, the applicant states that the results of the incomplete analyses, designed to dismiss any concerns for possible structure-to-structure interaction between the SWPS and the circulating water intake structure (CWIS), will show that the available 1-inch gap is adequate to accomodate the postulated lateral movements. [The staff intends to review and evaluate this analysis during the structural audit of July 27-30, 1982. Staff conclusions will be added to this supplement following the audit.]

3.7.3 Siesmic Subsystem Analysis

Further discussion on the staff evaluation of the applicant's request for increased in allowable damping values will be provided in a future supplement as identified in Sections 3.7.1 and 3.7.2.

3.8.1 Concrete Containment

Further discussion of the staff evaluation of the applicant's Seismic Safety Margins Report for the containment building will be provided in a future supplement.

3.8.1.1 Ultimate Capacity

By letter of June 8, 1982, the applicant has been asked to perform and provide analyses that determine the ultimate capacity of the Midland containments. The pressure-retaining capacity of localized areas as well as the overall containment structures should be determined using as-built conditions.

The analyses should be made on the basis of the allowable material strength specified in the Code. However, if the actual material properties (such as concrete cylinder compressive strength, mill test results of reinforcing steel and liner plate, strength variations indicated by mill test certificates) and other uncertainties are available, the lower and upper bounds of the containment capacities may be established statistically.

3.8.2 Concrete and Structural Steel Internal Structures Inside Containment

Further discussion on the staff evaluation of the applicant's Seismic Safety Margins Report for the concrete and steel structures inside the containment building will be provided in a future supplement.

3.8.3 Other Seismic Category I Structures

Further discussion on the staff evaluation of the applicant's Seismic Safety Margins Report for other Category I Structures will be provided in a future supplement.

The applicant has designed the new BWST foundation rings and all of the underpinning structures for the auxiliary building, FIVP, and SWPS, to current staff acceptance criteria.

3.8.3.1 Auxiliary Building and Feedwater Isolation Valve Pits

For the auxiliary building, a continuous underpinning wall resting on undisturbed natural material (soil) will be provided under the Control Tower (CT) and Electrical Penetration area (EPA) exterior walls. The modified foundation under each FIVP is as described in Section 2.5.4.4.1 of th s SSER. The proposed underpinning under the EPAs consists of a 6-foot thick reinforced concrete wall that is 38 ft. high and is belled at the base to 10 ft. in thickness. The CT underpinning walls are 6 ft. thick, 47 ft. high and are belled at the base to 14 ft. in thickness. All of the walls are constructed to act as continuous members under the perimeter of the structures. The entire wall system will be founded on undisturbed natural material. The applicant has identified both temporary and permanent underpinning schemes. The temporary support will be used during the construction of the permanent foundation. Jacking forces are applied to the existing structure to provide adequate load transfer from the structure to the underpinning. The jacking force is determined so that the structure is not unduly stressed under dead load and live load conditions. These jacking forces are transmitted from the structure through the permanent underpinning wall to the bearing stratum. Dowels connect the underpinning walls and the existing structures at the vertical and horizontal interfaces. The dowels are designed to transfer shear and tension forces between the structure and the underpinning wall. In addition to the conventional lap splice, Fox Howlett mechanical tapered thread splices will be used in the reinforcing of the underpinning walls. [Conclusions to be provided after audit. See Footnote*.]

3.8.3.2 Service Water Pump Structure

For the SWPS the underpinning consists of a 4-foot thick, reinforced concrete wall that is approximately 30 ft. high with a flared base. This underpinning wall is constructed to act as a continuous member under the perimeter of that portion of the structure founded on backfill material. A predetermined jacking force will be applied to the full perimeter of the SWPS overhang during construction to provide adequate load transer from the structure to the underpinning wall. [Conclusions to be provided after audit. See Footnote*.]

3.8.3.3 Borated Water Storage Tanks

For the BWST foundation, a new reinforced concrete ring located on the periphery of the existing ring represents the proposed remedial fix. Shear connectors transfer shear forces from the existing ring wall to the new adjacent ring beam. [Conclusions were provided in SER and will be further discussed in Final SSER after audit. See Footnote*.]

3.8.3.4 Diesel Generator Building

The DGB is a rectangular box-like reinforced concrete structure covering an area approximately 70 x 155 ft. The exterior walls are 30 inches thick, while three 18 in. interior walls divide the box into four bays approximately equal in size. The foundation of the exterior and interior walls of the DGB consists of a continuous reinforced concrete footing, 10 ft. wide and 2' -6" thick with the base at elevation 628 ft. The walls rise from an elevation of 628 ft. (bottom of footing) to 680 ft. (top of roof slab). The diesel generators rest on 6' -6" thick concrete pedestals. The DGB is located on plant fill.

As discussed in Section 2.5.4 of this supplement, the applicant investigated the excessive differential settlement of the DGB foundation, concluded that the plant fill was not sufficiently compacted and was subject to potential liquefaction, and implemented a surcharge and dewatering program as remedial action. The early investigation also showed that the four electrical duct banks that were supported on the deeper more competent natural clay but which penetrated the diesel generator building from below, were resulting in resistance to the DGB settlement in localized areas thus resulting in formation of cracks. To eliminate this problem a positive clearance between the building foundation and the duct bank was provided prior to placement of the surcharge.

The staff review during the evaluation of the remedial action proposed and completed for the DGB, has focused upon the cause and elimination of the excessive differential settlement condition, the applicant's structural acceptance criteria, the determination of proper soil and structural models to be used for additional analyses and evaluation of present and future conditions of the structure, the evaluation of the cracks developed during the differential settlement and duct impingement load mechanism and in the establishment of an adequate differential settlement and crack monitoring and repair program. The surcharge of the DGB accelerated settlement and produced soils with improved engineering properties. These properties have been used in both the static and seismic re-analyses of the DGB. Differential settlement, both measured and the 40-year prediction, has been included in the Midland load combinations. Differential settlement loads have been included in the applicable load combinations. Also, a new set of soil spring constants with varying properties (one vertical and one horizontal at each foundation boundary node point) representing the non-homogeneous nature of the soil conditions were developed and used in the finite element model. A set of soil spring constants was developed for the long term (settlement, 40 year) and short term (tornadoes, earthquakes) loadings. The applicant has also committed to re-analyze the DGB in accordance with current staff criteria (ACI 349 as supplemented with R-G 1.141).

The applicant has performed three new analyses of the DGB, one for each of the configurations and loadings existing before, during and after surchage. The applicant has proposed to run a hypothetical case in which part of the foundation support has zero spring stiffness and the remaining support equivalent spring stiffnesses. The applicant proposes this case as an upper bound on the differential settlement calculations for the foundation structure. The staff recommendation for settlements to be used for this analysis is given in Section 2.5.4.4.2 of this supplement. [The final SSER will report the staff's conclusions following submittal of the required analysis.]

3.8.3.5 Cracks

The applicant has shown, by example where necessary, that exising cracks do not affect significantly the strength in tension, compression, and shear of properly reinforced concrete elements. Evidence from the field and from the laboratory has been presented to indicate that reinforced concrete structures will develop their design strength even if they do have "precracks", provided the structure has been proportioned and detailed to resist the design load combinations. In addition, the applicant proposed a monitoring plan to detect differential settlement of the structure and the propagation and enlargement of new and existing cracks, along with an independent evaluation

evaluation of conditions which exceed predetermined limits acceptable to the staff, and a crack repair program acceptable to the staff. [Staff conclusions later.]

*Footnote:

[The applicant has responded to various staff requests for information. However, the staff has indicted some concerns and has identified most of them in memoranda dated June 15 and 28, 1982. This information and few additional concerns have been discussed with the applicant in a meeting held in Bethesda on June 25, 1982 (see minutes of meeting). Based on the discussions and commitments taken place at the June 25, 1982 meeting, the staff can conclude that the staff concerns become confirmatory issues to be resolved at the structural audit scheduled for July 27-30, 1982.]

3.8.4 Foundations

Discussion of information on foundations for this supplement is presented in Section 3.8.3.

3.8.5 Masonry Walls

SER Section 3.8.2 noted, as a confirmatiroy issue, that the applicant hai been asked to comply with staff criteria on masonry walls in seismic Category I structures. The issue also was identified as Item 3 in SER Section 1.8. The applicant has provided the criteria that he intends to follow in the evaluation of the masonry walls within seismic Category I structures.

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

The loads and load combinations used in the analysis and design of seismic Category I masonry walls are in conformance with staff criteria and are, therefore, acceptable.

However, the use of concrete expansion anchors to attach piping and equipment to masonry walls is disallowed by staff criteria. The applicant's specifications for the installation of concrete expansion anchors rely upon installation torque to determine the required load capacity of the installed anchors. Test data supplied by the applicant to qualify the use of expansion anchors in masonry walls indicate that there is no reliable relationship between installation torque and load capacity. This fact is highlighted by the following comment taken from the "Report on the Testing of Concrete Expansion Anchors and Grouted Anchors Installed in Concrete Blockwalls", by Bechtel Associates Professional Corporation, August, 1980:

"If the long and short embedment lengths are treated separately, there is no clear relationship between the recorded installation torgue and the tension failure load. This clearly deemphasizes the importance of the installation torque...".

Furthermore, the test data submitted by the applicant indicates that the mode of failure is by bolt slip or pull-out. This is a sudden and unpredictable mode of failure and is unacceptable to the staff.

With the exception of the expansion anchors used to support piping and equipment in masonry walls, the criteria used in the design analysis of the seismic Category I masonry walls to account for anticipated loadings that may be imposed upon the structures during their service lifetime are in conformance with the staff's criteria for masonry walls, and with codes, standards and specifications acceptable to the staff. We conclude that in the event of earthquakes and various postulated accidents, the seismic Category I masonry walls will withstand the specified design conditions without impairment of structural integrity. Conformance with these criteria constitutes an acceptable basis for satisfying, in part, the requirements of GDC 2 and 4. Accordingly, confirmatory issue 3 in SER Section 1.8 is closed, but a new open item is added to SER Section 1.7 regarding expansion anchors used in masonry walls.

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Mechanical Engineering

3.9.3.1 Loading Combinations, Design Transients and Stress Limits

[Later]

The applicant has indicated that the settlement induced stresses in the replaced 36" service water pipe considerably exceed the stress allowable (3Sc), when subjected to an assumed maximum settlement of 1½ inches. He has also stated that these large stresses are fictitious and result from the conservative boundary conditions which were assumed in the analysis. He has, however, not yet been able to provide any analytical justification that if more realistic boundary conditions were to be assumed, the stresses due to settlement would be reduced to 3Sc.

We will require that the applicant perform an analysis with a conservative settlement profile which will show that the stresses due to settlement do not exceed the allowable stress value of 3Sc when subjected to a maximum settlement of 1½ inches. If this cannot be shown, he will be required to provide a soil foundation such that the expected settlement will not induce stresses in excess of the allowable stress value.

DRAFT