DEPARTMENT OF THE ARMY
NORTH CENTRAL DIVISION, CORPS OF ENGINEERS
536 SOUTH CLARK STREET
CHICAGO, ILLINOIS 60605

NCDED

10 JUL

Dr. Robert Jackson
U.S. Nuclear Regulatory Commission
Division of Systems Safety
Mail Stop P-314
Washington, D.C. 20555

Dear Mr. Jackson;

The inclosed Letter Report, covering subtask No. 1 of Interagency Agreement No. NRC-03-79-167 concerning Units 1 and 2 of Midland Nuclear Plant, is hereby transmitted to you from the Detroit District.

Sincerely,

ZANE M. GOODWIN, P.E.
Chief, Engineering Division

1 Incl
As Stated

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ZANE M. GOODWIN, P.E.
Chief, Engineering Division

DEPARTMENT OF THE ARMY
DETROIT DISTRICT, CORPS OF ENGINEERS
BOX 1027
DETROIT, MICHIGAN 48231

REPLY TO
ATTENTION OF
NCEED-T

7 JUL 1980

SUBJECT: Interagency Agreement No. NRC-03-79-167, Task No. 1 - Midland Plant Units 1 and 2, Subtask No. 1 - Letter Report

THRU: Division Engineer, North Central
ATTN: NCDED-G (James Simpson)

TO: U.S. Nuclear Regulatory Commission
ATTN: Dr. Robert E. Jackson
Division of Systems Safety
Mail Stop P-314
Washington, D. C. 20555

1. The Detroit District hereby submits this letter report with regard to completion of subtask No. 1 of the subject Interagency Agreement concerning the Midland Nuclear Plant, Units 1 and 2. The purpose of this report is to identify unresolved issues and make recommendations on a course of action and/or cite additional information necessary to settle these matters prior to preparation of the Safety Evaluation Report.

2. The Detroit District's team providing geotechnical engineering support to the NRC to date has made a review of furnished documents concerning foundations for structures, has jointly participated in briefing meetings with the NRC staff, Consumers Power Company (the applicant) and personnel from North Central Division of the Corps of Engineers and has made detailed site inspections. The data reviewed includes all documents received through Amendment 78 to the operating license request, Revision 28 of the FSAR, Revision 7 to the 10 CFR 50.54(f) requests and MCAR No. 24 through Interim Report No. 8. Generally, each structure within the complex was studied as a separate entity.

3. A listing of specific problems in review of Midland Units 1 and 2 follows for Category I structures. The issues are unresolved in many instances, because of inadequate or missing information. The structures to be addressed follow the description of the problem.

a. Inadequate presentation of subsurface information from completed borings on meaningful profiles and sectional views. All structures.

b. Discrepancies between soil descriptions and classifications on boring logs with submitted laboratory test results summaries. Examples of such discrepancies are found in boring T-14 (Borated water tank) which shows stiff to very stiff clay where laboratory tests indicate soft clay with shear strength of only 500 p.s.f. The log of boring T-15 shows stiff, silty clay, while the lab tests show soft, clayey sand with shear strength of 120 p.s.f. All structures.

c. Lack of discussion about the criteria used to select soil samples for lab testing. Also, identification of the basis for selecting specific values for the various parameters used in foundation design from the lab test results. All structures.

d. The inability to completely identify the soil behavior from lab testing (prior to design and construction) of individual samples, because in general, only final test values in summary form have been provided. All structures.

(1) Lack of site specific information in estimating allowable bearing pressures. Only textbook type information has been provided. If necessary, bearing capacity should be revised based on latest soils data. All structures on, or partially on, fill.

(2) Additional information is needed to indicate the design methods used, design assumptions and computations in estimating settlement for safety related structures and systems. All structures except Diesel Generator Building where surcharging was performed.

e. A complete detailed presentation of foundation design regarding remedial measures for structures undergoing distress is required. Areas of remedial measures except Diesel Generator Building.

f. There are inconsistencies in presentation of seismic design information as affected by changes due to poor compaction of plant fill. Response to NRC question 35 (10 CFR 50.54f) indicates that the lower bound of shear wave velocity is 500 feet per second. We understand that the same velocity will be used to analyze the dynamic response of structures built on fill. However, from information provided by the applicant at the site meeting on 27 and 28 February 1980, it was stated that, except for the Diesel Generator Building, higher shear wave velocities are being used to re-evaluate the dynamic response of the structures on fill material. Structures on fill or partially on fill except Diesel Generator Building.

4. A listing of specific issues and information necessary to resolve them.

a. Reactor Building Foundation

(1) Settlement/Consolidation. Basis for settlement/consolidation of the reactor foundation as discussed in the FSAR assumes the plant site would

not be dewatered. Discuss and furnish computation for settlement of the Reactor Buildings in respect to the changed water table level as the result of site dewatering. Include the effects of bouyancy, which were used in previous calculations, and fluctuations in water table which could happen if the dewatering system became inoperable.

(2) Bearing Capacity. Bearing capacity computations should be provided and should include method used, foundation design, design assumptions, adopted soil properties, and basis for selecting ultimate bearing capacity and resulting factor of safety.

b. Diesel Generator Building.

(1) Settlement/Consolidation. In the response to NRC Question 4 and 27, (10 CFR 50.54f), the applicant has furnished the results of his computed settlements due to various kinds of loading conditions. From his explanation of the results, it appears that compressibility parameters obtained by the preload tests have been used to compute the static settlements. Information pertaining to dynamic response including the amplitude of vibration of generator pedestals have also been furnished. The observed settlement pattern of the Diesel Generator Building indicates a direct correlation with soil types and properties within the backfill material. To verify the preload test settlement predictions, compute settlements based on test results on samples from new borings which we have requested in a separate memo and present the results. Reduced ground water levels resulting from dewatering and diesel plus seismic vibration should be considered in settlement and seismic analysis. Furnish the computation details for evaluating amplitude of vibration for diesel generator pedestals including magnitude of exciting forces, whether they are constant or frequency dependent.

(2) Bearing Capacity. Applicant's response to NRC Question 35 (10 CFR 50.54f) relative to bearing capacity of soil is not satisfactory. Figure 35-3, which has been the basis of selection of shear strength for computing bearing capacity does not reflect the characteristics of the soils under the Diesel Generator Building. A bearing capacity computation should be submitted based on the test results of samples from new borings which we have requested in a separate memo. This information should include method used, foundation design assumptions, adopted soil properties and basis for selection, ultimate bearing capacity and resulting factor of safety.

(3) Preload Effectiveness. The effectiveness of the preload should be studied with regard to the moisture content of the fill at the time of preloading. The height of the water table, its time duration at this level, and whether the plant fill was placed wet or dry of optimum would be all important considerations.

(a) Granular Soils.

When sufficient load is applied to granular soils it usually causes a reorientation of grains and movement of particles into more stable positions plus (at high stresses) fracturing of particles at their points of contact. Reorientation and breakage creates a chain reaction among these and adjacent particles resulting in settlement. Reorientation is resisted by friction between particles. Capillary tension would tend to increase this friction. A moisture increase causing saturation, such as a rise in the water table as occurred here, would decrease capillary tension resulting in more compaction. Present a discussion on the water table and capillary water effect on the granular portion of the plant fill both above and below the water table during and after the preload.

(b) Impervious and/or Clay Soils.

Clay fill placed dry of optimum would not compact and voids could exist between particles and/or chunks. In this situation SPT blow counts would give misleading information as to strength. Discuss the raising of the water table and determine if the time of saturation was long enough to saturate possible clay lumps so that the consolidation could take place that would preclude further settlement.

Discuss the preload effect on clay soils lying above the water table (7 feet $\pm$) that were possibly compacted dry of optimum. It would appear only limited consolidation from the preload could take place in this situation and the potential for further settlement would exist.

Discuss the effect of the preload on clays placed wet of optimum. It would appear consolidation along with a gain in strength would take place. Determine if the new soil strength is adequate for bearing capacity.

Conclusion: Since the reliability of existing fill and compaction information is uncertain, additional borings and tests to determine void ratio (granular soils) relative density, moisture content, density, consolidation properties and strength (triaxial tests) would appear to be desirable in order to satisfactorily answer the above questions. Borings should be continuous push with undisturbed cohesive soil samples taken.

(4) Miscellaneous. A contour map, showing the settlement configuration of the Diesel Generator Building, furnished by the applicant at the meeting of 27 and 28 February 1980 indicates that the base of the building has warped due to differential settlements. Additional stresses will be induced in the various components of the structure. The applicant should evaluate these stresses due to the differential settlement and furnish the computations and results for review.

c. Service Water Building Foundation.

(1) Bearing Capacity. A detailed pile design based upon pertinent soil data should be developed in order to more effectively evaluate the proposed pile support system prior to load testing of test piles. Provide adopted soil properties, reference to test data on which they are based, and method and assumptions used to estimate pile design capacity including computations. Provide estimated maximum static and dynamic loads to be imposed and individual contribution (DL, LL, OBE, SSE) on the maximum loaded pile. Provide factor of safety against soil failure due to maximum pile load.

(2) Settlements.

(a) Discuss and provide analysis evaluating possible differential settlement that could occur between the pile supported end and the portion placed on fill.

(b) Present discussion why the retaining wall adjacent to the intake structure is not required to be Seismic Category I structure. Evaluate the observed settlement of both the service water pumphouse retaining walls and the intake structure retaining wall and the significance of the settlement including future settlement prediction on the safe operation of the Midland Nuclear Plant.

(3) Seismic Analysis. Provided the proposed 100 ton ultimate pile load capacities are achieved and reasonable margin of safety is available, the vertical pile support proposed for the overhang section of the Service Water Pump Structure will provide the support necessary for the structure under combined static and seismic inertial loadings even if the soil under the overhang portion of the structure should liquefy. There is no reason to think this won't be achieved at this time, and the applicant has committed to a load test to demonstrate the pile capacity. The dynamic response of the structure, including the inertial loads for which the structure itself is designed and the mechanical equipment contained therein, would change as a result of the introduction of the piles. Therefore:

(a) Please summarize or provide copies of reports on the dynamic analysis of the structure in its old and proposed configuration. For the latter, provide detailed information on the stiffness assigned to the piles and the way in which the stiffnesses were obtained and show the largest change in interior floor vertical response spectra resulting from the proposed modification. If the proposed configuration has not yet been analyzed, describe the analyses that are to be performed giving particular attention to the basis for calculation or selection, of and the range of numerical stiffness values assigned to the vertical piles.

(b) Provide after completion of the new pile foundation, in accordance with commitment No. 6, item 125, Consumers Power Company memorandum dated 13 March 1980, the results of measurements of vertical applied load and absolute pile head vertical deformation which will be made when the structural load is jacked on the piles so that the pile stiffness can be determined and compared to that used in the dynamic analysis.

d. Auxiliary Building Electrical Penetration Areas and Feedwater Isolation Valve Pits.

(1) Settlement. Provide the assumptions, method, computation and estimate of expected allowable lateral and vertical deflections under static and seismic loadings.

(2) Provide the construction plans, and specifications for underpinning operations beneath the Electrical Penetration Area and Feedwater Valve Pit. The requested information to be submitted should cover the following in sufficient details for evaluation:

(a) Details of dewatering system (locations, depth, size and capacity of wells) including the monitoring program to be required, (for example, measuring drawdown, flow, frequency of observations, etc.) to evaluate the performance and adequacy of the installed system.

(b) Location, sectional views and dimensions of access shaft and drift to and below auxiliary building wings.

(c) Details of temporary surface support system for the valve pits.

(d) Dewatering before underpinning is recommended in order to preclude differential settlement between pile and soil supported elements and negative drag forces.

(e) Provide adopted soil properties, method and assumptions used to estimate caisson and/or pile design capacities, and computational results. Provide estimated maximum static and dynamic load (compression, uplift and lateral) to be imposed and the individual contribution (DL, LL, OBE, SSE) on maximum loaded caisson and/or pile. Provide factor of safety against soil failure due to maximum pile load.

(f) Discuss and furnish computations for settlement of the portion of the Auxiliary Building (valve pits, and electrical penetration area) in respect to changed water level as a result of the site dewatering. Include the effect of houyancy, which was used in previous calculations, and fluctuations in water table which could happen, if dewatering system becomes inoperable.

(g) Discuss protection measures to be required against corrosion, if piling is selected.

(h) Identify specific information, data and method of presentation to be submitted for regulatory review at completion of underpinning operation. This report should summarize construction activities, field inspection records, results of field load tests on caissons and piles and an evaluation of the completed fix for assuring the stable foundation.

e. Borated Water Tanks.

(1) Settlement. The settlement estimate for the Borated Water Storage Tanks furnished by the applicant in response to NRC Question 31 (10 CFR 50.54f) is based upon the results of two plate load tests conducted at the foundation elevation (EL 627.00±) of the tanks. Since a plate load test is not effective in providing information regarding the soil beyond a depth more than twice the diameter of the bearing plate used in the test, the estimate of the settlement furnished by the applicant does not include the contribution of the soft clay layers located at depth more than 5' below the bottom of the tanks (see Boring No. T-14 and T-15, and T-22 thru T-26).

(a) Compute settlements which include contribution of all the soil layers influenced by the total load on the tanks. Discuss and provide for review the analysis evaluating differential settlement that could occur between the ring (foundations) and the center of the tanks.

(b) The bottom of the borated tanks being flexible could warp under differential settlement. Evaluate what additional stresses could be induced in the ring beams, tank walls, and tank bottoms, because of the settlement, and compare with allowable stresses. Furnish the computations on stresses including method, assumptions and adopted soil properties in the analysis.

(2) Bearing Capacity. Laboratory test results on samples from boring T-15 show a soft stratum of soil below the tank bottom. Consideration has not been given to using these test results to evaluate bearing capacity information furnished by the applicant in response to NRC Question 35 (10 CFR 50.54f). Provide bearing capacity computations based on the test results of the samples from relevant borings. This information should include method used, foundation design assumptions, adopted soil properties, ultimate bearing capacity and resulting factor of safety for the static and the seismic loads.

f. Underground Diesel Fuel Tank Foundation Design

(1) Bearing capacity. Provide bearing capacity computation based on the test results of samples from relevent borings, including method used, foundation design assumptions, adopted soil properties, ultimate bearing capacity and the resulting factor of safety.

(2) Provide tank settlement analysis due to static and dynamic loads including methods, assumptions made, etc.

(3) What will [illegible] effects of uplift pressure on the stability of the tanks and the associated piping system if the dewatering system becomes inoperable?

g. Underground Utilities:

(1) Settlement

(a) Inspect the interior of water circulation piping with video cameras and sensing devices to show pipe cross section, possible areas of crackings and openings, and slopes of piping following consolidation of the plant fill beneath the imposed surcharge loading.

(b) The applicant has stated in his response to NRC Question 7 (10 CFR 50.54f) that if the duct banks remain intact after the preload program has been completed, they will be able to withstand all future operating loads. Provide the results of the observations made, during the preload test, to determine the stability of the duct banks, with your discussion regarding their reliability to perform their design functions.

(c) The response to Question 17 of "Responses to NRC Requests Regarding Plant Fill" states that "there is no reason to believe that the stresses in Seismic Category I piping systems will ever approach the Code allowable." We question the above statement based on the following:

Profile 26" - OHBC-54 on Fig. 19-1 shows a sudden drop of approx. 0.2 feet within a distance of only 20 feet. Using the procedure on p. 17-2,

$$\sigma_b = E(e) = E\left(\frac{D}{2R}\right) = E\left(\frac{D}{2}\right)\left(\frac{8\delta}{L^2}\right)$$

$$\sigma_b = 30000\ \left(\frac{26}{2}\right)\left[\frac{8(0.2)(12)}{(20x12)^2}\right] = 130.0\ \text{KSI}$$

Furthermore, the Eq. 10(a) of Article NC-3652.3, Sec. III, Division 1, of the ASME code requires that some Stress Intensification Factor "i" be assigned to all computed settlement stresses. Yet, Table 17-2 lists only 52.5 KSI stress for this pipe. This matter requires further review. Please respond to apparent discrepancy and also specify the location of each computed settlement stress at the pipeline stationing shown on the profiles. More than one critical stress location is possible along the same pipeline.

(d) During the site visit on 19 February 1980, we observed three instances of what appeared to be degradation of rattlespace at penetrations of Category I piping through concrete walls as follows:

<u>West Borated Water Tank</u> - in the valve pit attached to the base of the structure, a large diameter steel pipe extended through a steel sleeve placed in the wall. Because the sleeve was not cut flush with the wall, clearance between the sleeve and the pipe was very small.

<u>Service Water Structure</u> - Two of the service water pipes penetrating the northwest wall of the service water structure had settled differentially with respect to the structure and were resting on slightly squashed short pieces of 2 x 4 placed in the bottom of the penetration. From the inclination of the pipe, there is a suggestion that the portions of the pipe further back in the wall opening (which was not visible) were actually bearing on the invert of the opening. The bottom surface of one of the steel pipes had small surface irregularities around the edges of the area in contact with the 2 x 4. Whether these irregularities are normal manufacturing irregularities or the result of concentration of load on this temporary support caused by the settlement of the fill, was not known.

These instances are sufficient to warrant an examination of those penetrations where Category I pipe derives support from plant fill on one or both sides of a penetration. In view of the above facts, the following information is required.

(1) What is the minimum seismic rattlespace required between a Category I pipe and the sleeve through which it penetrates a wall?

(2) Identify all those locations where a Category I pipe deriving support from plant fill penetrates an exterior concrete wall. Determine and report the vertical and horizontal rattlespace presently available and the minimum required at each location and describe remedial actions planned as a result of conditions uncovered in the inspection. It is anticipated that the answer to Question (1) can be obtained without any significant additional excavation. If this is not the case, the decision regarding the necessity to obtain information at those locations requiring major excavation should be deferred until the data from the other locations have been examined.

(e) Provide details (thickness, type of material etc.) of bedding or cradle placed beneath safety related piping, conduits, and supporting structures. Provide profiles along piping, and conduits alignments showing the properties of all supporting materials to be adopted in the analysis of pipe stresses caused by settlement.

(f) The two reinforced concrete return pipes which exit the Service Water Pump Structure, run along either side of the emergency cooling water reservoir, and ultimately enter into the reservoir, are necessary for safe shutdown. These pipes are buried within or near the crest of Category I slopes that form the sides of the emergency cooling water reservoir. There is no report on, or analysis of, the seismic stability of post earthquake residual displacement for these slopes. While the limited data from this area do not raise the specter of any problem, for an important element of the plant such as this, the earthquake stability should be examined by state-of-the-art methods. Therefore, provide results of the seismic analysis of the slopes leading to an estimate of the permanent deformation of the pipes. Please provide the following: (1) a plan showing the pipe location with respect to other nearby structures, slopes of the reservoir and the coordinate system; (2) cross-sections showing the pipes, normal pool levels, slopes, subsurface conditions as interpreted from borings and/or logs of excavations at (a) a location parallel to and about 50 ft from the southeast outside wall of the service water pipe structure and (b) a location where the cross section will include both discharge structures. Actual boring logs should be shown on the profiles; their offset from the profile noted, and soils should be described using the Unified Soil Classification System; (3) discussion of available shear strength data and choice of strengths used in stability analysis; (4) determination of static factor or safety, critical earthquake acceleration, and location of critical circle; (5) calculation of residual movement by the method presented by Newmark (1965) or Makdisi and Seed (1978); and (6) a determination of whether or not the pipes can function properly after such movements.

h. Cooling Pond.

(1) Emergency Cooling Pond. In recognition that the type of embankment fill and the compaction control used to construct the retention dikes for the cooling pond were the same as for the problem plant fill, we request reasonable assurance that the slopes of the Category I Emergency Cooling Pond (baffle dike and main dike) are stable under both static and dynamic loadings. We request a revised stability analysis for review, which will include identification of locations analyzed, adopted foundation and embankment conditions (stratification, seepage, etc.) and basis for selection, adopted soil properties, method of stability analysis used and resulting factor of safety with identification of sliding surfaces analyzed. Please address any potential impact on Category I pipes near the slopes, based on the results of this stability study. Recommendations for location of new exploration and testing have been provided in a separate letter.

(2) Operating Cooling Pond. A high level of safety should be required for the remaining slopes of the Operating Cooling Pond unless it can be assured that a failure will not: (a) endanger public health and properties, (b) result in an assault on environment, (c) impair needed emergency access. Recommendations for locations of new borings and laboratory tests have been submitted in a separate letter. These recommendations were made on the assumptions that the stability of the operating cooling pond dikes should be demonstrated.

i. Site Dewatering Adequacy.

(1) In order to provide the necessary assurance of safety against liquefaction, it is necessary to demonstrate that the water will not rise above elevation 610 during normal operations or during a shutdown process. The applicant has decided to accomplish this by pumping from wells at the site. In the event of a failure, partial failure, or degradation of the dewatering system (and its backup system) caused by the earthquake or any other event such as equipment breakdown, the water levels will begin to rise. Depending on the answer to Question (a) below concerning the normal operating water levels in the immediate vicinity of Category I structures and pipelines founded on plant fill, different amounts of time are available to accomplish repair or shutdown. In response to Question 24 (10 CFR 50.54f) the applicant states "the operating groundwater level will be approximately el 595 ft" (page 24-1). On page 24-1 the applicant also states "Therefore el 610' is to be used in the designs of the dewatering system as the maximum permissible groundwater level elevation under SSE conditions." On page 24-15 it is stated that "The wells will fully penetrate the backfill sands and underlying natural sands in this area." The bottom of the natural sands is indicated to vary from elevation 605 to 580 within the plant fill area according to Figure 24-12. The applicant should discuss and furnish response to the following questions:

(a) Is the normal operating dewatering plan to (1) pump such that the water level in the wells being pumped is held at or below elevation 595 or (2) to pump as necessary to hold the water levels in all observation wells near Category I Structures and Category I Pipelines supported on plant fill at or below elevation 595, (3) to pump as necessary to hold water levels in the wells mentioned in (2) above at or below elevation 610, or (4) something else? If it is something else, what is it?

(b) In the event the water levels in observation wells near Category I Structures or Pipelines supported on plant fill exceed those for normal operating conditions as defined by your answeer to Question (a) what action will be taken? In the event that the water level in any of these observation wells exceeds elevation 610, what action will be taken?

(c) Where will the observation wells in the plant fill area be located that will be monitored during the plant lifetime? At what depths will the screened intervals be? Will the combination of (1) screened interval in cohesionless soil and (2) demonstration of timely response to changes in cooling pond level prior to drawdown be made a condition for selecting the observation wells? Under what conditions will the alarm mentioned on page 24-20 be triggered? What will be the response to the alarm? A worst case test of the completed permanent dewatering and groundwater level monitoring systems could be conducted to determine whether or not the time required to accomplish shutdown and cooling is available. This could be done by shutting off the entire dewatering system when the cooling pond is at elevation 627 and determining the water level versus time curve for each observation well. The test should be continued until the water level under Category I structure, whose foundations are potentially liquefiable, reaches elevation 610 (the normal water level) or the sum of the time intervals allotted for repair and the time interval needed to accomplish shutdown (should the repair prove unsuccessful) has been exceeded, whichever occurs first. In view of the heterogeneity of the fill, the likely variation of its permeability and the necessity of making several assumptions in the analysis which was presented in the applicant's response to Question 24a, a full-scale test should give more reliable information on the available time. In view of the above the applicant should furnish his response to the following:

If a dewatering system failure or degradation occurs, in order to assure that the plant is shutdown by the time water level reaches elevation 610, it is necessary to initiate shutdown earlier. In the event of a failure of the dewatering system, what is the water level or condition at which shutdown will be initiated? How is that condition determined? An acceptable method would be a full-scale worst-case test performed by shutting off the entire dewatering system with the cooling pond at elevation 627 to determine, at each Category I Structure deriving support from plant fill, the water level at which a sufficient time window still remains to accomplish shutdown before the water rises to elevation 610. In establishing the groundwater level or condition that will trigger shutdown, it is necessary to account for normal surface water inflow as well as groundwater recharge and to assume that any additional action taken to repair the dewatering system, beyond the point in time when the trigger condition is first reached, is unsuccessful.

(2) As per applicant response to NRC Question 24 (10 CFR 50.54f) the design of the permanent dewatering system is based upon two major findings: (1) the granular backfill materials are in hydraulic connection with an underlying discontinuous body of natural sand, and (2) seepage from the cooling pond is restricted to the intake and pump structure area, since the plant fill south of Diesel Generator Building is an effective barrier to the inflow of the cooling pond water. However, soil profiles (Figure 24-2 in the "Response to NRC Requests Regarding Plant Fill"), pumping test time-drawdown graphs (Figure 24-14), and plotted zones of influence (Figure 24-15) indicate that south of Diesel Generator Building, the plant fill material adjacent to

the cooling pond is not an effective barrier to inflow of cooling pond water. The estimated permeability for the fill material as reported by the applicant is 8 feet/day and the transmissivities range from 29 to 102 square feet/day. Evaluate and furnish for review the recharge rate of seepage through the fill materials from the south side of the Diesel Generator Building on the permanent dewatering system. This evaluation should espacially consider the recovery data from PD-3 and complete data from PD-5.

(3) The interceptor wells have been positioned along the northern side of the Water Intake Structure and service water pump structures. The calculations estimating the total groundwater inflow indicate the structures serve as a positive cutoff. However, the isopachs of the sand (Figures 24-9 and 24-10) indicate 5 to 10 feet of remaining natural sands below these structures. The soil profile (Figure 24-2) neither agrees nor disagrees with the isopachs. The calculations for total flow, which assumed positive cutoff, reduced the length of the line source of inflow by 2/3. The calculations for the spacing and positioning of wells assumed this reduced total flow is applied along the entire length of the structures. Clarify the existence of seepage below the structures, present supporting data and calculations, and reposition wells accordingly. Include the supporting data such as drawdown at the interceptor wells, at midway location between any two consecutive wells, and the increase in the water elevations downstream of the interceptor wells. The presence of structures near the cooling pond appears to have created a situation of artesian flow through the sand layer. Discuss why artesian flow was not considered in the design of the dewatering system.

(4) Provide construction plans and specification of permanent dewatering system (location, depths, size and capacity of wells, filterpack design) including required monitoring program. The information furnished in response of NRC Question 24 (10 CFR 50.54f) is not adequate to evaluate the adequacy of the system.

(5) Discuss the ramifications of plugging or leaving open the weep holes in the retaining wall at the Service Water Building.

(6) Discuss in detail the maintenance plan for the dewatering system.

(7) What are your plans for monitoring water table in the control tower area of the Auxiliary Building?

(8) What measures will be required to prevent incrustation of the pipings of the dewatering system. Identify the controls to be required during plant operation (measure of dissolved solids, chemical controls). Provide basis for established criteria in view of the results shown on Table 1, page 23 of tab 147.

(9) Upon reaching a steady state in dewatering, a groundwater survey should be made to confirm the position of the water table and to insure that no perched water tables exist.

Dewatering of the site should be scheduled with a sufficient lead time before plant start up so that the additional settlement and its effects (especially on piping) can be studied. Settlement should be closely monitored during this period.

j. Liquefaction Potential.

An independent Seed-Idriss Simplified Analysis was performed for the fill area under the assumption that the groundwater table was at or below elevation 610. For 0.19 g peak ground surface accceleration, it was found that blow counts as follows were required for a factor of safety of 1.5:

Elevation ft	Minimum SPT Blow Count*[1] For F.S. = 1.5
610	14
605	16
600	17
595	19

The analysis was considered conservative for the following reasons (a) no account was taken of the weight of any structure, (b) liquefaction criteria for a magnitude 6 earthquake were used whereas an NRC memorandum of 17 Mar 80 considered nothing larger than 5.5 for an earthquake with the peak acceleration level of 0.19 g's, (c) unit weights were varied over a range broad enough to cover any uncertainty and the tabulation above is based on the most conservative set of assumptions. Out of over 250 standard penetration tests on cohesionless plant fill or natural foundation material below elevation 610, the criteria given above are not satisfied in four tests in natural materials located below the plant fill and in 23 tests located in the plant fill. These tests involve the following borings:

SW3, SW2, DG-18, AX 13, AX 4, AX 15, AX 7, AX 5, AX 11, DG 19, DG 13, DG 7, DG 5, D 21, GT 1, 2.

Some of the tests on natural material were conducted at depths of at less than 10 ft before approximately 35 ft of fill was placed over the location. Prior to comparison with the criteria these tests should be multiplied by a factor of about 2.3 to account for the increase in effective overburden pressure that results from the placement and future dewatering of the fill.

1*For M = 7.5, blow counts would increase by 30%.

Of the 23 tests on plant fill which fail to satisfy the criteria, most are near or under structures where remedial measures alleviating necessity for support from the fill are planned. Only 4 of the tests are under the Diesel Generator Building (which will still derive its support from the fill) and 3 others are near it. Because these locations where low blow counts were recorded are well separated from one another and are not one continuous stratum but are localized pockets of loose material, no failure mechanism is present.

In view of the large number of borings in the plant fill area and the conservatism adopted in analysis, these few isolated pockets are no threat to plant safety. The fill area is safe against liquefaction in a Magnitude 6.0 earthquake or smaller which produces a peak ground surface acceleration of 0.19 g or less provided the groundwater elevation in the fill is kept at or below elevation 610.

k. Seismic analysis of structures on plant fill material.

(1) Category I Structures. From Section 3.7.2.4 of the FSAR it can be calculated that an average V_s of about 1350 ft/sec was used in the original dynamic soil structure interaction analysis of the Category I structures. This is confirmed by one of the viewgraphs used in the 28 February Bechtel presentation. Plant fill V_s is clearly much lower than this value. It is understood from the response to Question 13 (10 CFR 50.54f) concerning plant fill that the analysis of several Category I structures are underway using a lower bound average V_s = 500 ft/sec for sections supported on plant fill and that floor response spectra and design forces will be taken as the most severe of those from the new and old analysis. The questions which follow are intended to make certain if this is the case and gain an understanding of the impact of this parametric variation in foundation conditions.

(a) Discuss which Category I structures have and/or will be reanalyzed for changes in seismic soil structure interaction due to the change in plant fill stiffness from that envisioned in the original design. Have any Category I structures deriving support from plant fill been excluded from reanalysis? On what basis?

(b) Tabulate for each old analysis and each reanalysis, the foundation parameters (v_s, ν and ρ) used and the equivalent spring and damping constants derived therefrom so the reviewer can gain an appreciation of the extent of parametric variation performed.

(c) Is it the intent to analyze the adequacy of the structures and their contents based upon the envelope of the results of the old and new analyses? For each structure analyzed, please show on the same plot the old, new, and revised enveloping floor response spectra so the effect of the changed backfill on interior response spectra predicted by the various models can be readily seen.

(2) Category I retaining wall near the southeast corner of the Service Water Structure. This wall is experiencing some differential settlement. Boring information in Figure 24-2 (Question 24, Volume 1 Responses to NRC Requests Regarding Plant Fill) suggests the wall is founded on natural soils and backfilled with plant fill on the land side. Please furnish details clarifying the following:

(a) Is there any plant fill underneath the wall? What additional data beyond that shown in Figure 24-2 support your answer?

(b) Have or should the design seismic loads (FSAR Figure 2.5-45) be changed as a result of the changed backfill conditions?

(c) Have or should dynamic water loadings in the reservoir be considered in the seismic design of this wall? Please explain the basis of your answer.

5. In your response for the comments and questions in paragraph 4 above, if you feel that sufficiently detailed information already exists on the Midland docket that may have been overlooked, please make reference to that information. Resolution of issues and concerns will depend on the expeditious receipt of data mentioned above. Contact Mr. Neal Gehring at FTS 226-6793 regarding questions.

FOR THE DISTRICT ENGINEER:

D. McCallister

P. McCALLISTER
Chief, Engineering Division