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Dec. 7, 1978

14/136

To: DARL Hood

FROM: Gene Gallagher

Subject: Meeting Notes of Dec. 4, 1978 held at the Midland Site (Docket No 50-329 & 50-330)

The following is a summary of my understanding of the the technical discussion of the meeting held at the Midland site on Dec 4, 1978 regarding the excessive settlement of the Diesel Generator Bldg & foundations and other Category I Structures.

1. History by Bechtel - the Bechtel representative identified the Category I Structures & the types of material supporting the structures.
   - a. Containment - Glacial Till
   - b. Borated Water Storage Tank - Plant Fill
   - c. Diesel Generator Bldg & Pedestal - Plant Fill
   - d. Aux Bldg. - Part Glacial Till & Part Plant F.
   - e. Service Water Intake - Glacial Till

The settlement monitoring program began in June '78; to date the settlements are as follows:

- Containment - 1/4" to 5/8"
- Aux Bldg - Approx. 1/8"
- Service Water Pump house - 0 to 1/8"
- Borated Water Storage - Approx 1/8"
- Diesel Generator Bldg - 3 to 4"

The 4 electrical duct banks rising into the D.G Bldg were cut loose to remove the settlement restriction on the North side. When the duct banks were cut loose, settlement on the order of 2" took place on the north side of the D.G Bldg.

TO DARL HOOD
FROM GENE GALLAGHER

2. Soils Exploration - Bechtel discussed the soil exploration program including the boring program & laboratory testing of the foundation material. The conclusion that was made by Bechtel is that the material varies in strength properties i.e. unconfined compressive strength from 200 PSF to 4000 PSF & shear strength from 100 PSF to 2000 PSF. The soils classification ranged from CL to ML.

Bechtel also discussed possible causes based on input from Dr. R. Peck. Some of these causes were: 1. Variable quality of material used in the plant fill, however, the quality control records do not indicate the variation 2. Fill may have been placed on the dry side of optimum moisture & then when the water table rose inundating the fill the material become "soft".
3. Initial fill may have been placed satisfactory but after installing pipe trenches & duct banks the fill may have been disturbed.

3. Consultants Recommendations - Dr. Peck stated the following:

a. Compacted fill made up mainly of glacial ti
b. Evident from Dutch cones curve that the looser zones are zones or lenses (local areas)
c. Water content is higher than at the time the fill was placed
d. Short of removing all the fill above the hard glacial till a "preload" program would be best approach.

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e. Bldg would not have settled as much if the material had not been so wet (moisture content is high).

f. Bearing capacity is not a problem for the footings.

g. Preload purpose would be to consolidate the fill materials down to the original till.

h. The settlement with the preload would tend to be rapid (a few weeks to a few months). The "structure would go along for the ride".

i. The Preload is a necessary first step even though other measures might be necessary.

j. The main open question is what might happen to the rate of settlement as the water table rises & saturates the fill.

k. Preload would take place in the New Year & be cleaned out around July '79.

Mr. Dunnicliff spoke about the instrumentation program to monitor the settlement of the foundation material & structures during the preload. The purpose of the instrumentation would be to see if the surcharge is doing its job & if it is doing any harm to the structures or utility lines under & around the bldg.

a. INSTRUMENTATION for the structure would include optical survey measurements as well as monitoring of cracks using electrical devices. FOUR LOCATIONS for the electrical devices have been chosen; 2 on the exterior of the east wall & 2 on the west wall

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of bay 4 in the D.G. Bldg. A mapping of the cracks would be developed.

b. Foundation monitoring would include devices to measure settlement & pore water pressure. A total of 60 anchors would be installed (20 groups of 3 at different elevations. A total of 40 piezometers are to be installed to measure the pore water pressure.

The consultants indicated that 6" settlement would not be a surprise & that up to as much as 18" could occur. The preload would be made up of 15 TO 20 feet of sand piled in & around the D.G. bldg. No more than a 5 foot differential would be permitted.

4. Bechtel summarized the activities completed in program & planned for the future:

a. activities completed - 1) boring program 2) isolation of the electrical duct banks on the north side of the bldg.

b. Activities in Progress - 1) foundation settlement Monitoring program 2) Preparation for preload (instrumentation) 3) actual Preload of the structure & foundation & 4) filling the cooling pond to maximum elevation (Elev 627) 5) Complete the rest of the D.G. Bldg Structure

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c. Activities Planned - 1) after removal of surcharge assure contact between footings & soil material 2) Verify utilities & structure integrity

5. Project Schedule - Bechtel presented the following project schedule information:

Construction is 58% completed as of NOV 7.
Engineering is 80% complete
Structural Concrete is 97% complete
Fuel Load target date NOV. 1980
Earliest req'd for 1 Diesel generator, JAN 1:
Current Completion for 1 D.G, JAN '80
LATEST for 1 D.G June '80 (understa

6. Response to open items in NRC Inspection Report Nos 50-329/78-12 & 50-330/78-12:

[illegible] htel addressed the open items included in a [illegible] inspection report. CPCo stated that a written response would be sent to Region [illegible] to resolve the conflict between the FSAR & site implementing procedures:

a. Conflict between FSAR Table 2.5-14 & Tab 2.5-10 regarding the description of fill material & what was actually used in the random fill: Bechtel stated that this conflict was an "oversight" & that an FSAR amendment would be issued.

b. Conflict between FSAR Table 2.5-21 & spec C-2 regarding number of passes for compaction: Bechtel stated that FSAR table 2.5-21 is for the embankments for the cooling pond dikes.

c. FSAR Section 3.8.5.5 regarding expected settlement: Bechtel stated the ½ inch indicated in the FSAR was "NOT correct, a mistake". FSAR will be amended to correct the "mistake".

some mistakes in PSAR's

d. Conflict between FSAR Figure 2.5-47 & project drawing regarding foundation elevation Bechtel stated the elevations in the FSAR was "a mistake".

e. Conflict in spec C-210 regarding compaction effort: Bechtel stated that FCR C-3 dated 10/31/75 clarified this conflict & permitted the "Bechtel Modified Proctor" using 20,000 FT-LB's compactive effort rather than the ASTM standard of 56000 ft. The NRC inspector asked Dr. Peck if this was appropriate. Dr Peck stated that "Greater COMPACTION would have been in the right DIRECTION."

f. Conflict between DAmes & Moore recommendation regarding lift thickness of 6 to 8 inches & Bechtel spec permitting up to 12 inches: Bechtel stated that this should NOT matter. However, the NRC was informed that no test qualifications on the random fill material using 12 inches was performed to qualify such lift thicknesses.

Dr. Peck was asked his opinion. He stated that, "the thicker the layer the more difference in compaction through the thickness of the layer".

g. ± 2% tolerance in moisture content permitted in spec C-210: Bechtel stated that this tolerance is in line with industry practice.

Dr. Peck was asked his view on this ± 2% tolerance. He stated the important question is "± 2% of what material". Since the material used in the fill was variable the ± 2% tolerance could cause a problem is the material is not consistent.

h. Cracks in the Building Structure: Bechtel stated that all cracks greater than the ACI 318-71 limit would be identified & repaired after the preload program.

i. FSAR question 362.2: Bechtel stated that the answer had been sent to NRR via amendment 15 dated 11/78. NRR was to review the response

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Encl

Closing comments to the meeting were made by NRC's Lyman Heller. In summary he stated that the purposed solution is at the risk of the licensee & that NRR would review & evaluate the result to the original compaction requirements as set forth in the committments in the PSAR/FSAR

Gene Gallagher
Reactor Inspector, Region III
Div. of Inspection & Enforcement

CC LYMAN HELLER, NRR

MIDLAND Copy given in deposition documents -J. Kane 7/17/80

28/86

DEPARTMENT OF THE ARMY

DETROIT DISTRICT, CORPS OF ENGINEERS
BOX 1027
DETROIT, MICHIGAN 48231

COE Letter Report
Subtask No. 1

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

July 17, 1980
Copies furnished to the following reviewers for their comments by July 25, 1980:
Ray Gonzales
Dick McMullen (T. Cardone)
Leon Reiter (J. Kimball)
Tony Cappucci
Frank Rinaldi

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.

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16pp.

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

Specific information - See my comments of 7/18/80 for information and observations to be requested of CPCo

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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. Vol. 4 FSAR, Table 2.5-14 Gives only F.S.

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. Table 2.5-14 indicates results will be provided by amendment.

(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.

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(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. — What should discussion cover? What information in addition to Figs. 27-2 & 27-6 does the COE want? Fig 27-2 gives pond levels Fig. 27-6 thru 27-49 gives piezometric levels

(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.

— What structures have foundations below this elevation?

El 63?

Discuss the preload effect on clay soils lying above the water table (7 feet ±) 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.

Has been handled in separate memo
Delete

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.

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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.

Clarify w/ COE that this is the portion of structure that is being questioned

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

What in response to question 24e is not acceptable? Their statement of no settlement versus field trip observation of actual differential settlement

(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 temporary 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. ~~Describe the sequence of dewatering operations as underpinning operations are to proceed.~~

(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 bouyancy, 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. Table 2.5-14 indicates bearing capacity information will be provided by amendment

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. Is not but should be listed in Table 2.5-14

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

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(3) What will be 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. .9-1 shows a sudden drop of approx. 0.2 feet within a distance of only 20 feet. Using the procedure on p. 17-2,

7/25/80 — Spoke to T. Cappucci

*ACTION required
Incorporate the MEB comments when transmitting the COE report to CPCo

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

This method assumes constant curvature which is unlikely and is not a conservative assumption. MEB will pursue this concern by asking for an alternate analysis that is more conservative

$$\sigma_b = 30000\ \left(\frac{26}{2}\right)\left[\frac{8(0.2)(12)}{(20\times12)^2}\right] = 130.0\ \text{KSI}$$

This factor will be greater than 1 where there is a change in thickness (e.g. at elbows, tees & valves. For straight pipe sections the factor =1

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 allowable 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.

This statement covers something that is obvious. Generally we are concerned with locations of maximum stress

(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:

(Phone conversation on 7/28/80)
Tony Cappucci recommends deletion of sentence beginning w/ "Furthermore" because the computation is for a straight length of pipe (no tees, valves, etc.) & there would be no stress intensification factor > 1.
To clarify comparison, T. Cappucci recommends addition of words "as allowable" after 52.5 KSI

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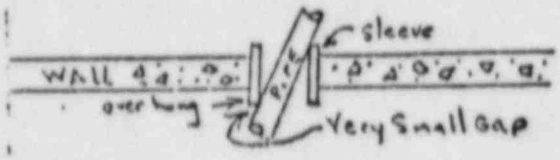
<u>West Borated Water Tank</u> - in the valve pit attached to the base of the structure, a large diameter steel pipe extended through a steel sleeve placed in the wall. Because the sleeve was not cut flush with the wall, clearance between the sleeve and the pipe was very small.



<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. Request submittal of piezometer reading (See FSAR Fig. 2.5-63 & 2.5-64)

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:

what is difference

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 cones 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 especially 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.

Provide you plans for conducting

(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.

Should note either here or in cover letter (The conclusions in Par. J could be affected by the final resolution of the seismic input for the Midland project.)

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 acceleration, 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

For Use @ May 5-7, 1981 Meetings

Rec'd 4/23/81 · J Kane

29/86

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

NCDED-G

21 APR 1981

Mr. George Lear
U.S. Nuclear Regulatory Commission
Division of Engineering
Mail Stop P-214
Washington, DC 20555

Dear Mr. Lear:

The inclosure containing review comments prepared by the Detroit District regarding Amendment 85 on the Midland Nuclear Generating Plant in partial completion of Interagency Agreement No. NRC-03-79-167 is hereby transmitted to you.

Sincerely,

Zane M. Goodwin

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

1 Incl
As Stated

Also coordinate COE letter of response to Interrogatory 3. on problems w/ CoCo

F. Rinaldi Hoped to review by 6/5/81

Rec'd T. Cappucci comments by 5/29/81 (Pgs. 11 & 13)

Coordinate w/ GSB, SEB, MEB & HES.
also Lyman, George and Dan

8105060341 4p.

DEPARTMENT OF THE ARMY

DETROIT DISTRICT, CORPS OF ENGINEERS
BOX 1027
DETROIT, MICHIGAN 48231

REPLY TO
ATTENTION OF

16 APR 1981

NCEED-T

SUBJECT: Interagency Agreement No. NRC-03-79-167, Task No. 1 - Midland Plant, Unit 1 and 2, Subtask No. 3 - Review Comments on Amendment 85

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

TO: Mr. George Lear
U.S. Nuclear Regulatory Commission
Chief, Hydrologic & Geotechnical Engr. Br.
Division of Engineering
Mail Stop P-214
Washington, DC 20555

1. The Detroit District has reviewed the information received from the applicant through Amendment 85 to the operating license request, Revision 10 to the 10 CFR 50.54(f) requests. The information received addresses all the questions (Question 39 thru 48) raised by the Corps of Engineers in their letter report which was forwarded to the Nuclear Regulatory Commission on 7 July 1980, which subsequently was transmitted to the applicant on 4 August 1980 for his response.

2. The review comments are inclosed. The purpose of these review comments is to identify the discrepancies noted in the applicant's response and apprise the NRC of the Corps of Engineers views as to the safety of the foundations of the structures deriving support from fill as well as from natural soil.

3. A listing of the specific discrepancies noticed during the review are as follows:

a. The shear strength parameters used in the analyses are not the representative parameters for the soils for which the analyses have been performed. The bearing capacity of the foundation soils for the Borated Water Tanks and the Diesel Generator Building appears to have been done on the basis of the shear strength parameters obtained from the test results on the soil samples which do not represent the soil conditions prevailing beneath these structures.

b. The evaluation of the settlements for the Borated Water Tanks, Diesel Generator Building, Service Water Structure and the Reactor Buildings have

Indicates issues impacted by current boring & testing program

~~8105060343~~ 16pp.

been based on either assumed values of the Young's modulus or on compressibility coefficients obtained from the questionable preload test.

c. In most of the cases of the settlement evaluations, only the immediate settlements have been considered. The consolidation and the secondary settlements have not been considered. (Reactor Buildings, Service Water Building Foundation, etc.)

4. A listing of the specific discrepancies in the applicant's response to Question 39 through 48 are given.

Question 39 - Reactor Building Foundation

(1) Settlement/Consolidation. The applicant's response to Question 39(1) indicates that the settlements due to the dewatering have been computed on the basis of the Young's Modulus of the soil determined from the load-settlement relations between May 17, 1977 and March 11, 1978. The determination of the Young's Modulus using load-settlement requires use of the soil's poisson ratio and the influence factor of the footing. Further, the settlement that occurred immediately after the application of the load should be known and be used. The applicant has not explained how these parameters were determined. The Young's Moduli determined by the procedure shown on page 39-8, should have been used to determine the settlements due to the dewatering instead of using constrained modulus used by the applicant. The Young's Modulus obtained by backfiguring is based on the appropriate confining pressure and as such is appropriate for computing the settlements caused by dewatering load. The consolidation and the secondary settlements have not been added to the total settlement. The applicant should address the primary consolidation settlements and the time for them to occur due to the load caused by the dewatering. Presently, we are not certain whether the information provided in FSAR is enough to evaluate the time-settlement relation or additional consolidation tests will be required. Identify the consolidation test results being used in the determination of the primary consolidation settlements. The applicant should also address the secondary consolidation settlements due to the dewatering load, even though such settlements appear to be negligible due to the high overconsolidation ratio of the glacial till over which the Reactor Buildings are founded.

The applicant should * update the observed settlements and loading records as promised in response to Question 362.9 and compare the observed settlements with predicted settlements. He should also develop a technical specification for monitoring settlements, which should establish tolerable, total, and differential settlement limits during the plant operation. (Give locations & frequency)

* Want to evaluate any effect due to temp. dewatering (both settlement & load history)
Last records provided on Sept. 14, 1980 gave readings up to May 1980 which was before dewatering.
Involved FSAR Figures - 2.5-89 (Turbine), 2.5-90 (Auxil. Bldg.); 2.5-91 (Reactor) and Table 2.5-14A
Report of Sept 14, 1980 Figures are nos. 2 thru 5.
Do provided graphs by CPCo give confidence that future settlements will be minimal & acceptable?

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(2) Bearing Capacity. The shear strength values used in the analysis of the bearing capacity of the soil under the Reactor Buildings were taken from the weighted average of the undrained shear strength of the soil samples obtained mostly from the cooling pond dikes area. A review of Table 2.5-6, (FSAR Volume 3) and the borings by the Michigan Drilling Company indicates that of all the samples tested for undrained shear strength, only one was taken from the area of the Reactor Buildings. Therefore, the shear strength used for the bearing capacity analysis is not representative of the soils on which the Reactor Building is founded. The drained shear strength parameters ($\phi = 32°$, C = 590 PSF) used in design of bearing capacity under static loads, also appear to be based on the average of the shear tests on the samples obtained from the entire plant area. In view of these facts, the response of the applicant is not satisfactory. The applicant must evaluate the shear strength parameters from the soil samples obtained from the soil mass below or near the Reactor Building foundation. The information obtained from the Dames & Moore boring Nos. 1, 2, 3, 4 and 15 might be used to determine shear strength parameters for the bearing capacity analyses of the Reactor Buildings. Limited information available from the tests performed on the samples obtained from these borings are presented in FSAR Volume 4. The applicant might choose to use this information provided he can demonstrate that the test results available are within the depth of influence for estimating bearing capacity.

Question 40 - Diesel Generator Building.

-Request updated settlement & loading vs. time records of monuments to view effect of temp. dewatering & add'l. loading of generators.

(1) Settlement/Consolidation. (a) The applicant has not furnished the requested information pertaining to the settlements of the Diesel Generator Building. The settlements computed on the basis of the compressibility parameters obtained from the preload test are questionable because of these reasons:

(i) There is questionable evidence to confirm that preload was held long enough to eliminate 100% of the primary consolidation.

(ii) Because of the flexibility of the footings, the surcharge loads were not evenly distributed. The foundation soils with relatively more compressible fill (southeast corner) have been subjected to a load intensity less than that of the surcharge, therefore, the applicant's statement that, "the stresses prevailing during surcharging at all depths in the fill beneath the building exceeded those that will prevail while the structure is operational," is questionable.

Is this an appropriate place to bring in problem w/ response to Interrog. No?

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(iii) The sudden drop in the piezometer levels after removal of the surcharge is due to negative pore pressures as the soil tries to swell. This is a normal reaction. After swell is complete, the piezometer readings should return to the normal water level in the ground. However, in this situation, they generally returned to some value greater than the ground water level which could indicate the presence of excess pore pressures.

The raise in piezometer levels to a height greater than groundwater levels after the dissipation of negative pore pressures are indicative that excess pore pressures were not completely dissipated at the time of surcharge removal. See piezometer 12, 17, 23, 25, 29, 34, 36, 40 and 43. 47

Check w/ COE - Statement on settlement induced by machine vibrations - also OK for seismic shakedown in view of recent borings

(2) Bearing Capacity. The bearing capacity analyses for the Diesel Generator Building furnished by the applicant are based on the shear strength parameters (Ø,C), which are not representative of the soil fill beneath the Diesel Generator Building. The numerical values of the angle of internal friction, Ø, and the cohesion, C, were determined on the basis of the results of consolidated undrained tests on five samples taken from the areas of the Tank Farm (Series T borings) and the Transformers (Series TR borings). A review of the boring logs indicates that all of the five samples were obtained from the zones of stiff to hard clay (blowcounts varies from 12 to 19), with dry densities ranging from 114.4 pcf to 117.9 pcf, liquid limits ranging from 20% to 35% and plasticity index ranging from 9 to 20. Three of the samples (T9-8, T16-5, TR2-2) had been overconsolidated (anisotropically consolidated) to the overconsolidation (consolidation stress) ratio ranging from 1.1 to 2.2 prior to testing, which stiffened the samples and changed their shear strength characteristics in comparison to those which were not overconsolidated (anisotropically consolidated). The basis for doing such overconsolidation (anisotropically consolidating) test should be given. Thus, it is evident that samples used to determine shear strength parameters are not representative and as such, the information obtained by these tests indicate a soil type which does not exist in the effective Diesel Generator Building area. The soil types beneath the Diesel Generator Building range from layers of soft to hard clay as well as loose to very dense sand. An attempt to determine shear strength parameters by mixing the soil samples from layers of various soil types would result in misleading information as to strength. Selection of samples for testing as requested in 30 June 1980 letter from A. Schwencer to J. W. Cook, should follow the guidance in Regulatory Guide 1.138 paragraph C.5.a, and cover not only the typical foundation condition, but also the extreme and critical zones. The resulting shear strength test results obtained should then be considered in evaluation of the bearing capacity for the foundation soil beneath the Diesel Generator Building.

See Fig. 35-3 Vol. 6 pg. 167

R.G. 1.138-80 par. c.4.a

See results on glacial till, FSAR, Vol. 4, pg. 2.5-43, Table 2.5-6
App A logs for borings 1, 2, 3, 4 & 15 (pgs. 2A-2A)
App. B pg 2B-7

(3) Preload Effectiveness. As discussed in our review comments on the applicant's response of Question 40-1, the preload program may have not been effective in eliminating 100% of the primary consolidation, under the surcharge load of 2.2 KSF. We are not in agreement with the applicant's statement that the preload program carried out at the Diesel Generator Building has demonstrated to have been successfully completed. The

compressibility parameters obtained from the preload test are questionable and, therefore, future settlement predictions of the Diesel Generator Building based on these parameters should be verified with the results from the requested laboratory consolidation tests. Validity of Figure 27-9 (Revision 6), in which the comparison of measured and predicted settlments is made, is questionable due to the reasons given in our review comments on the response of Question 40-1. Raising of the cooling pond's water level to elevation 627 at the beginning of April 1979, did not saturate the soil up to elevation 625 beneath the Diesel Generator Building during the surcharge, as stated by the applicant. The drops in the piezometer levels to elevation 622± on removal of surcharge indicates the water table to be at elevation 622±, resulting in considerable capillary action in the fill material below the footing (el = 628). The effect of such capillary action is to resist settlement. A rise in moisture, causing saturation, such as cut-off water during rain, would decrease capillary action causing more settlement. In addition, it has not been established whether the clay fill was installed wet or dry of optimum moisture. If placed the dry side of optimum, the preload, even with the rise of the watertable, may not have consolidated the clay sufficiently to preclude further settlement.

Comparison of moisture contents (Figs. 40-5 & 40-6) w/opt. moisture content do not tell us whether fill was placed wet of optimum (CPCo response pg. 40-7, last paragraph). Since pond was raised from El. 608 to 622± in March 1978 and samples tested for moisture content were taken in borings completed in late 1978 - we do not know placement moisture contents. Comparison of Figs. 40-5 & 40-6 do give confidence that soils below El. 620 had been significantly wetted due to pond raising to El. 622 in Mar. 1978 (before Jan 1979 surcharg

(4) Miscellaneous. The contour map (Figure 40-9) furnished by the applicant in response to Question 40-4, clearly shows warping of both the north and the south walls indicating curvatures created by bending moments. This warping would continue to grow with time, because of the future settlements of the east and the west ends about a rigid pivot in the center provided by the condensate pipe which has been reconnected after the removal of the surcharge load. An analysis of stresses induced by the warping should be performed taking into account the differential settlement over the life span of the plant (40 years). The applicant should refer to the answers for Interrogatory 8 (Nuclear Regulatory Commission staff answer to interrogatory filed by the applicant, 25 February 1981) for the comments on the analyses which are needed to evaluate effects of structural cracks.

SEB

Question 41 - Service Water Building Foundation.

Must evaluate new proposal by CPCo. Is there a problem w/adequacy of knowing soil conditions and soil parameters?

Design Change - No longer required

(1) Bearing Capacity. The use of drained shear strength parameters to analyze the ultimate bearing capacity of the proposed piles is not justified. The ultimate pile load capacity from the load test would simulate an undrained condition, (even a long duration pile load test would not create a drained condition at the tip of the pile in this case); a static pile load analysis should be performed using undrained parameters. The shear strength parameters used in determination of the side frictions (F_1, F_2, F_3) and point resistance (F_4) are not the representative values for the soil condition prevailing at the locations where the piles will be driven. The same values of Ø and C are used for sand as well as clay (see sheet 2 of Attachment 41-1). The applicant has used shear parameters for a soil type

Design change- No longer Required

which he has created by mixing the test results of samples of Series T, TR and CT. In Attachement 41-1, the depth of fill considered in evaluating F_1 and F_2 is 27.5 feet, but the actual depth of the fill reported in Borings Logs CH-1 through CH-6A (Volume 9 of the applicant's response to 10 CFR 50.54(f) Questions) indicates approximately 45' of fill material in the area where the underpinning piles will be driven. The computations of the ultimate pile capacity should be revised using 38.5' (45'-6.5') of fill instead of 27.5' used previously. The ultimate pile load capacity from the load test, R_u, shown to be 280 tons on page 41-3 should be revised considering the increased negative skin friction due to the increase in the fill material. Further, it appears that the determination of R_u at 280 tons (page 41-1) has been computed by multiplying the design load (100 tons = normal dead plus live loads on each pile) of the piles with a factor of safety of 2.5 and then adding to this value the negative skin friction of 30 tons (computed in Attachment 41-1). However, in our opinion the above approach of evaluating the ultimate pile load capacity from the load test is not correct. The factor of safety of 2.5 must be applied to the external load of 100 tons on the pile top plus the computed skin friction and the product then be added to the skin friction again [2.5 (100 tons + NSF) + NSF].

(2) Settlements.

(a) Paragraph 1 of the applicant's response to Question 41, Part 2a indicates that vertical load on piles was calculated based on an appropriate spring stiffness of the underpinning piles and the subgrade modulus of the mat foundation resting on natural soil. However, in our opinion, the stiffness of the cantilevered portion of the Service Water Structure will be a factor in computing the underpinning pile load. Provide total computed pile loads due to dead and live loads as well as total vertical and horizontal loads due to seismic actions, along with the detailed analysis for the spring stiffnesses of the underpinning piles. The settlement values provided by the applicant indicates a time dependent settlement of 0.1 inch for the portion of the Service Water Structure founded on glacial till and 0.05 inch for the portion to be supported on underpinning piles. The analyses for these settlements have many questionable assumptions and rationalizations such as:

(i) Application of pile loads over an area of 15' x 3.5' (sheet 5 of 6 Attachment 41-2) at the tip elevation is not appropriate. According to Bjerrum et al (1957), such a simplified method underestimates the settlements.

(ii) It is not known whether the soil moduli used in the analyses are for drained or undrained conditions. For a long term settlement, soil modulus for drained condition should be used.

Design Changed - No longer Required

(iii) The simplified approach used by the applicant is used in conjunction with one dimensional consolidation theory.

(iv) Secondary settlement has not been considered in evaluation of long term settlement.

(v) The applicant's planning to jack the underpinning piles after the dewatering settlement takes place is not realistic. Dewatering settlement is a time-dependent settlement and it might take many years to complete. The dewatering settlement of the area under the pile tip is estimated to be 0.48 inch (sheet 3 of 6 Attachment 41-2, Line 2), but it is not known what compressibility parameters were used to compute this settlement. In view of these facts, the differential settlement problem still remains unresolved. The approach outlined for computing settlement of pile group in Pile Foundation Analysis and Design, Paulos and Davis, John Wiley and Sons, may be used.

Refer to pg. 41-6 of Amend. 85

(b) The analyses indicating a factor of safety of 2.2 against failure for the slope behind the retaining wall near the Circulating Water Intake Structure is based on soil parameters that may not be applicable to the type of fill material behind the wall. The applicant should base the analyses on the representative shear strength parameters from the test results on samples taken near the retaining walls. A thirty feet (30') distance between the top edge of the failure plane and the nearest safety related Diesel Fuel Storage Tanks shown in Figure 41-4 does not appear adequate. Provide, (1) the groundwater condition considered in the analysis, (2) loading conditions (e.g. earthquake, seepage, drawdown, etc.) considered in the slope analysis which resulted in the safety factor of 2.2, (3) the identifications of boring logs, soil samples and the laboratory test results which are the basis for the allowable shear strength parameters provided on page 41-6.

(3) Seismic Analysis.

(a) and (b). The analyses furnished and the additional work the applicant has committed to perform would insure the seismic safety of the foundations, provided the representative soil parameters have been used in the analyses. (Is COE satisfied they are representative. What is needed to answer?

Question 42 - Auxiliary Building, Electrical Penetration Areas and Feedwater Isolation Valve Pits. This will be affected by Spencer-White presentation

SEB

(1) Settlement. The applicant's response that "Settlement of the Feedwater Isolation Valve Pit (FIVP) and the caisson of the Electrical Penetration Area (EPA) will be identical" is not correct. The caissons of the EPA and the concrete fill of the FIVP would not act monolithically. The continuity of the top few feet of the FIVP concrete fill around the casings of

SEB

the caissons in the EPA would not establish adequate structural bond between concrete fill and the caissons. In the case it happens, the poor soil fill around the caissons below the concrete fill is still compressible and the problem still remains unsolved.

(2) (a) Temporary Dewatering System - The Corps is in agreement with the applicant's response.

SEB

(b) Figure 42-68 shows the location of the access shaft. However, the location and the dimensions of the drift are not shown. The technical specifications for the work provided in Attachment 42-2 do not specify anything about the drifts. Item 3b of Attachment 42-2 indicates that the caissons will be extended at least 4' into the till; with this constraint the caissons' tip might end up with different elevations because of the sloping natural till surface caused by the foundation excavation of the containment buildings. In the design of the bearing capacity of the soils under the caissons tip, the effect of this factor has not been considered. Item 3d, states that the caissons should have a vertical resistance capacity sufficient to produce a static moment of at least 325,000 foot-kips at column rows 5.3 and 7.8. The meaning of this statement is not clear. Item 4 of Attachment 42-2 provides a very brief outline of caisson load testing. But it is not clear what remedial measures will be taken if a completed caisson fails to meet the load test. A caisson filled with concrete cannot be driven further. An empty shell test (EST) by loading to 1.0 times the design load prior to placing concrete appears unrealistic, because with only 4' penetration in glacial till it is not possible to obtain frictional resistance adequate to perform load test with 1.0 times the design load (frictional resistance of fill should be neglected for load test). In item 5.2.1e, the applicant proposes to complete, test and wedge each caisson tight to the structure under a load equal to 1.5 times the design load, on a one by one basis. This procedure does not appear feasible; a previously wedged caisson under the bottom of the structure might be released when jacking for next caisson is applied under the structure.

Check w/ SEB
Criteria given on pg. 42-2.1s SEB in agreement

(c) Temporary Surface Support - The response of the applicant for the temporary support system for the valve pit is vague. Additional design information should be provided to assess the stresses on members required for temporary support.

(d) The applicant's response indicates that the caissons capacities have been determined on the basis of the shear strength parameters, determined from the soil samples obtained from other areas. On sheet 3 of 6 Attachment 42-3, in the equation for ultimate bearing capacity, Q_f, the last term accounts for the contribution due to adhesion between the caisson surface and the soil. The cohesion value 6 K.S.I. used in this term must be

Pg. 42-2 & Fig. 42-68 Sects. A&B How will caissons be terminated in concrete backfill zone? Will sloping concrete backfill be excavated & a min. of 4' into glacial till required?

Pg. 42-3 Need to address CPCo desire to complete final design by subcontractor & NRC need to agree on adequacy of any future changes

what are CPCo plans for requiring qualified personnel (submit experience & qualifications of key personnel) and monitoring (settlement, lateral displacements, instruments, limiting values requiring action multiplied by a reduction factor, α, to obtain the adhesion. For stiff clay as encountered at the tip of the caissons, using the full value of the cohesion as adhesion is not justified. Also, in computing load at the base of each caisson, the concrete fill and the soil between the caissons should be considered. This will have an effect of reducing the factor of safety. In case of an earthquake, an undrained condition would prevail in the soil around the caissons, therefore, an analysis for the caissons' group capacity and factor of safety based on an undrained condition are required. The applicant has not performed analysis for the caissons group capacity, considering the SSE earthquake. It is our understanding that the 4,000 kips, which the caissons have to transmit to the glacial till, do not include dynamic load due to a potential earthquake.

SEB

is this LL & DL but not earthquake

(e) Settlement of Auxiliary Building due to change in water level during dewatering. See review comment of 42(1).

(f) The applicant's response is acceptable.

(g) The applicant's response is acceptable.

Questions 43 - Borated Water Tanks.

(1) Settlement. Since the soils beneath the tanks consists of not only granular type but also clay, the major part of the settlement will be consolidation settlement and secondary settlement. Consolidation and secondary settlements are time-dependent and might continue for the full operation life of the tanks. Therefore, settlement measured from full scale load test, as proposed by the applicant would not provide the accurate settlement. To accelerate the settlements, the tank must be surcharged with a load considerably more than the load which it has been designed to carry. However, because of the tanks fixed volumetric capacity, the surcharge load cannot be increased in excess of its design load. Blowcount plots shown in Figures 31-3 and 31-4 show variations in blowcounts from a minimum of 6 to a maximum of 43 in the area of the East Borated Water Tank, and from a minimum of 4 to a maximum 57 in the area of the West Borated Water Tank, indicating that soil layers of variable density and consistency exist under the tanks. Therefore, the information obtained from plate load tests cannot be used to determine the settlements. The application of the theory of elasticity requires soil moduli for drained and undrained conditions to determine time dependent and immediate settlements. It is not known what values the applicant has used to determine the differential settlements. To review the differential settlements, the numerical values of *Young's modulus of the soils and the methods used to determine them are required. Secondary settlements also need to be evaluated to determine the structural adequacy of the tank bottoms.

SEB

*Even when we have the Young's modulus will we be satisfied without consolidation tests that estimate both primary & secondary consolidation? Do we want consolidation testing?

Were borings and sampling completed @ borated water tank locations?

Ask how arrived @ (Pg. 43-3)

(b) The differential settlement of 1-1/2", using elastic plate theory, appears to be computed on assumed value of soil moduli; therefore, it does not present the potential differential settlement. The soil moduli ranging from 260 kips per cubic foot to 490 kips per cubic foot used to determine differential settlements for the ring walls are not realistic for the soil conditions prevailing under the tanks. The above values of soil moduli are applicable to soils with consistencies ranging from very stiff to very hard. Under the Borated Water Tanks, the soil consistencies vary from soft to very stiff. Provide actual settlement records of the Borated Water Tanks, and indicate the effect the settlement has on the piping between the tanks and the Auxiliary Building. The records should include the loading history.

What not added by COE

An acceptable method: Appropriate foundation pressure Total estimated settlement estimate @ several locations

This was promised in response to Q6 as well as on piping between tanks and Auxil. Bldg. Loading history should also be furnished. Do these records potentially indicate secondary consolidation? Compare observed settlements with anticipated settlements.

(2) Bearing Capacity. The shear strengths used in the analysis of the bearing capacity of the soils under the Borated Water Tanks are not appropriate to the soils conditions prevailing under the tanks. Figure 35-3, used to obtain the undrained shear strength, was constructed from the results obtained from the tests on the soil samples taken from the various locations of the plant area. These samples had densities ranging from 114.6 pcf to 131.3 pcf, water content 9.3% to 16.2%, and liquid limits ranging from 18% to 35%. Thus, the samples were not identical, and therefore, shear strengths obtained from Figure 35-3 are misleading. It is advisable to compute the bearing capacity of the soils using the soil parameters of the soil beneath the tanks. Attachment 43-1 shows the bearing capacity analyses. On sheet 2 of Attachment 43-1, there appears to be some computational error in evaluating effective confining pressure. The σ_v (617) should be the average of pressure at elevation 600 (bottom of fill) and elevation 635 (top of fill). Also, the numerical value of 0.55 for the coefficient of lateral pressure at rest, K_o, is for over consolidation ratio (OCR) 2 which should not be used for fill material. A OCR of 1 is appropriate for the fill material, the K_o for this OCR is 0.49. The applicant should perform analysis for the factor of safety using the results from the shear testing of the soil samples taken near the Borated Water Tanks area and within the depth zone influenced by the bearing capacity analysis. Has potential to require borings & testing @ borated water tank if existing results are inadequate.

Question 44 - Underground Diesel Fuel Tank Foundation Design

(1) Bearing Capacity. The applicant's response is acceptable.

(2) Settlement. Although the soil under the Diesel Generator Building and under the Diesel Fuel Storage Tanks are of the same classifications, their strengths, compressibilities and the permeabilities are not necessarily the same in numerical values. The use of classifications to evaluate the fundamental properties (shear strengths, compressibilities, and permeabilities) is not a sound engineering practice, particularly for the use in design of a Category I Structure of a nuclear power plant. The settlement

evaluation of the Diesel Fuel Storage Tanks performed by the applicant by comparing the soil classifications under the Diesel Fuel Storage Tanks with those under the Diesel Generator Building are not acceptable. In addition, boring log DF-5 (Figure 33-1) indicates a layer of loose sand below the pads, which is susceptible to densification resulting in some settlement under a dynamic load. Therefore, settlements due to dynamic load should be estimated.
Not considered major. Rough estimate OK

(3) Uplift Pressure on Tanks. The applicant has not performed any analyses to demonstrate the effect of uplift pressure on the stability of the tanks. The stability of the tanks in uplift cannot be assured unless the applicant can demonstrate, by analysis, that an acceptable factor of safety against uplift of the tanks does exist. The applicant is requested to provide the results of the analysis for uplift resistance. (What is safety margin resisting uplift? assuming dewatering system failure)

Question 45 - Underground Utilities

MEB

(1) (a) Settlement - From the applicant's reponse it appears that it has no plan to perform inspection of the interior of the water circulating pipings for cracks and openings after the removal of the surcharge load as requested in part (1)(a) of Questions 45. The applicant has made reference to the measurements of the deformations during surcharge for line 96-2YBJ-4, which was reported in response to Questions 19, 10 CFR 50-54(f). However, it has made no attempt to compute the pipe stresses from the measured deformations, and as such the measured deformations do not provide any information regarding the adequacy of the pipe. In absence of the requested information, it is not possible to check the adequacy of the pipings which were affected by the surcharging of the Diesel Generator Building.

Per T. Cappucci's remarks on 5/29/81
Pipe is NOT Cat. I. From engineering standpoint it would appear appropriate that overstressing of pipe should be checked because of surcharging

SEB

(b) Duct Banks - The applicant's response to Question 7, 10 CFR 50.54(f), indicates that reinforcing bars in the duct banks had exceeded the yield strain under the building load which the duct banks carried prior to their isolation from the walls of the Diesel Generator Building. This implies that permanent deformations have occurred in the reinforcing bars and cracks wider than normally permitted in reinforced concrete structures have already developed in the duct banks. In response to Question 30, 10 CFR 50.54(f), the applicant has provided the results of its seismic analyses for the duct banks, but it is not known whether or not it has taken into account the effects of permanent strains in the reinforcing bars created by the previous load. This aspect should be further reviewed by the appropriate engineering section of the Nuclear Regulatory Commission. Per F. Rinaldi on 6/1/81 - CPCo says duct banks are not Category I

MEB

(c) Buried Piping - Applicant has stated it will respond after consultation with the NRC.

MEB

(d) We concur with the applicant's response, except the response to Question 45(d)(1). In the applicant's response to Question 45(d)(1), the last column in Table 45-1, which is entitled "Building Displacement to Pipe (1)," gives minimum rattlespace requirements at penetrations of Category I free-field piping supported on plant fill into various structures. In that column of the table, the quantities given for the eight penetrations of the Diesel Generator Building are "V < .015 inch and H < 0.03 inch." For the nine penetrations for the Auxiliary Building, the quantities given are "V < .036 inch and H < 0.129 inch." These numbers seem much too small. What the ranges imply is that less than 1/8 inch relative displacement is expected between the building and the nearby free field. The applicant should provide detailed information as to (a) the sources of the numbers mentioned above, (b) describe how they were computed, (c) what percentage of the free-field maximum displacement implicit in the shock spectrum or of the displacement obtained by double integration of the free-field acceleration are these rattlespace values. In addition, we are addressing the following two review comments to the applicant for his response.

From Haulala Jan 16, 1981 memo

(1) Since the structures are quite stiff, most of the relative movement between the pipe and the structure that would occur in a seismic event would be due to relative movements between the base of the structure and the free-field at the elevation of the penetration. Relative movements of the free-field at the two levels could be roughly estimated by H V_{max}/V_s where H is the vertical distance between the base of the structure and the penetration, V_{max} is the free-field maximum particle velocity, and V_s is the shear wave velocity of the fill. Alternatively, the effect of an H/V_s time shift in a free-field ground motion vs time plot could be used to compute relative displacement of two points in the free-field. In addition, for heavy structures the question of whether the structure foundation moves with the free field should be considered.

Paul Haulala

(2) Table 45-1 indicates that everywhere there is much more than the applicant's stated minimum rattlespace requirements, but there are a few places where clearances "C" are less than 1 inch. This is an unacceptable situation, in our opinion. Some future settlement of the plant fill (under its own weight) in the nonsurcharged areas is to be expected. The pipes will move downward further reducing "C." After consideration of the original source for minimum clearances given in Table 45-1 and the range of numbers for the analyses suggested above, the applicant is requested to provide revised minimum clearances and state the action to be taken to achieve them.

(e) The applicant's response that "the analysis of the settlement stresses in the piping is unrelated to the properties of the supporting materials" is correct. The evaluation of the stresses using the radius of whether pipes are bent & overstressed

C.T. Cappucci's comment on 5/?/81
MEB agrees this information is needed, particularly with respect to dynamic analysis, but is uncertain what COE means by "stabilities of the pipes"

MEB

curvature computed from the measured deflections of the piping from their original positions, does not require soil properties of the bedding on which the pipes are laid. However, to review of the stabilities of the pipes near supports and for dynamic analysis it is necessary to know the support conditions (thrust blocks etc.). Therefore, we are reiterating our request that the applicant should furnish the requested information in Question 45(1)(e).

(f) The applicant's response to Question 45(1)(f) is not satisfactory. The shear strength parameters used in the analysis of slope stability of the dikes may not be representative values for the soil conditions prevailing in the soil mass of the dikes. The value of the angle of internal friction, Ø, used in the total stress analysis has been manipulated from the Ø (drained condition) given in FSAR Table 2.5-22 rather than using the actual value obtained from the test results on samples taken from the dikes, or from the test results of the record samplings. The values of the shear strength parameters provided in Table 45-2, page 45-7, are basically taken from the FSAR Table 2.5-22, which are assumed values for the design. Thus, the applicant has not demonstrated that the shear strength parameters of the soil mass in the dikes are identical or better than those of the assumed values for the design of the dikes. The applicant has further attempted to justify the soil parameters selected on the basis of the average blowcounts (Figures 45-4 thru 45-10) of the standard penetration test (SPT). The tests for this area (except boring No. P2-5) do not provide blowcount information for top 15' height of the dikes. As a matter of fact, except boring Series P2 involving five borings across one particular cross section of the emergency cooling pond dike, all of these tests were carried out in the natural soil, therefore, they provide no information about the fill material of the dikes.

From 27° to 25°

Question 46 - Cooling Pond

(1) Emergency Cooling Pond. In paragraph 1 of the response, the applicant has referred to its submission of September 14, 1980, and has stated that as pointed out in the submission, the compaction to construct the cooling pond dike was different from the problem fill in the power block area. A review of the applicant's submission of 14 September 1980, indicates that it has no intention to furnish the requested information. The explanations provided in the submission against making additional borings as requested by the staff has no engineering merits. The applicant has taken no record samplings at all to verify the design assumptions as to the shear strength parameters. It has performed no field control tests for compacted soils in dikes above elevation 620±. The boring logs of the standard penetration tests (SPT), through the dike's fill material conducted for the installation of the piezometers, show no blowcount numbers above elevations 620± with one

Support condition - fdn. soil of future settlement estimate, thrust blocks, concrete cradle
details bends, end supports

exception which is boring No. P2-4 where a blowcount number of 7 has been recorded at elevation 625.7±. Thus, the results of the standard penetration test furnished by the applicant provide no information regarding the soil conditons for approximately the top 15' of the dikes. Further, the blowcount records from boring No. P1-2 and P1-3 (see boring logs furnished with the response to Question 46) indicate soft clay in the east dike below elevation 620. In absence of the requested information, it is not possible to review the applicant's response.

(2) Operating Cooling Pond - The applicant's response to Question 46(2) is not satisfactory. Our comments on the response to Question 46(1) are applicable to this question. In addition, the averaging of the blowcounts, which varies from a minimum of 4 (see boring log 611 in Figure 45-6) to a maximum of more than 100 for clays and silt and from a minimum of 10 to a maximum of more than 100 for sand, would provide totally misleading information as to the strength of the soils. Averaging of the blowcounts is acceptable, if all the blowcounts belong to one particular consistency or relative density group. The method adopted by the applicant would not allow for locating weak and strong stratifications of the soils.

We concur with the remaining portions of the applicant's response to Question 45(1)(f). If the appropriate values of shear strength parameters are used, the analyses performed would assure the seismic safety of the foundations of the two Category I reinforced concrete return pipes.

Question 47 - Site Dewatering

(1) (a) We concur with the applicant's response.

(b) The additional work the applicant has committed to perform in its response of this question will assure the seismic safety of the foundations of Category I structures, deriving support from the plant fill. Therefore, we concur with the response.

HES

(c) The remedial measures completed, and the additional work the applicant has committed to perform, would provide definite data on the adequacy of the analyses that the applicant has relied on to demonstrate safety. For example, this will verify whether or not there are more than 90 days recharge time to reach elevation of 610 as calculated by the applicant in his response to Question 24(a), 10 CFR 50.54(f).

(2) In its response to Question 47(2), the applicant has presented results of the pumping tests and hydrographs (see Figures 47-7 and 47-8) to demonstrate that the plant fill south of the Diesel Generator Building is an

effective barrier to the inflow of the cooling pond water. However, none of these test results can substantiate that the plant fill is an effective barrier. The results indicate that inflows of water from the south side is less than that from the area of the Service Water Structure. However, since the applicant is planning to monitor the water elevations in plant areas, and to perform a full scale test (last paragraph of response to Question 47(1)c), the seepage from the south end will also be accounted for, and if the test indicates more than 90 days recharge time to reach the elevation of 610, the dewatering system will be acceptable.

HES

(3) The applicant has revised the analyses for the inflow in the line-slot on the basis of a combined gravity-artesian flow to design the dewatering system. However, it has reduced the value of the permeability of the aquifer from 31' (used in the previous analysis) to 17' per day obtained from the pumping test of the well No. PD-15A which is the nearest to the locations of the proposed dewatering wells. The method of analysis furnished by the applicant is acceptable to the Corps of Engineers. But the validity of using a reduced permeability of 17' per day should be further reviewed by the appropriate section of the NRC.

HES

Covered by current review on permanent dewatering

(4) The filter pack gradation requirements provided on page 47-12 of the response, appears to have been designed for a aquifer material gradation determined on the basis of the boring logs of Series PD borings. What measures (established gradation of soils within depth interval of screens, modify filter pack gradation) will be required during the well installations and during production pumping to prevent infiltration of soil fines from material finer than the gradation submitted in Figure 47-12?

Acceptance criteria of sand in discharge from an individual well after the completion of its development given on page 47-14 (10 PPM or less) does not provide any information regarding the amount of erosion that will take place over the 40 year life span of the plant. Provide flow rate, sand in flow in terms of PPM (taken at some interval), and quantity of total sand pumped during the development of the wells on the basis of each individual well as well as on the basis of total number of wells. Also provide the criteria of sand in discharge related to flow rate of a single well as well as of the entire system of wells during the production pumping including an estimate of volume of sand material removed in one month and during the 40 year plant life based on your submitted criteria.

HES

(5) We concur with the applicant's response.

(6) The quantity of chemicals in groundwater shown in Table 47-3 indicates the possibility of early incrustation (high percentage of $CaCo_3$, Ph > 7.5, etc.). Therefore, the applicant's maintenance program should also

HES

consider periodical cleaning of the incrustations by an acceptable method. We concur with the rest of the work the applicant has committed to perform in his maintenance program.

HES

(7) We concur with the applicant's response.

(8) We concur with major part of the applicant's responses. However, in our opinion the high percentage of $CaCo_3$ shown in Table 47-3 indicates early possibility of incrustation, and the applicant should stipulate a remedial measure in its maintenance program by periodical cleaning.

HES

48 - Seismic Analysis of the Structures on Plant Fill Material

SEB

(1) (a)(b)(c) The seismic analyses which have been completed, and the additional work the applicant has in process, or committed to perform, will either (a) assure the seismic safety of foundations of the Category I structures deriving support from the plant fill or, (b) provide definite data on the adequacy of the analyses that the applicant has relied on to demonstrate safety. However, in case of the Diesel Fuel Storage Tank Foundation, we disagree with the applicant's response. A seismic investigation as to the settlement of the loose sand indicatd by boring DF-5 needs to be investigated. Not considered major

Paul Hadala

(2) (a)(b)(c) The applicant has furnished the requested information, and we are satisfied with the applicant's response.

5. If you have any question regarding our review comments, please contact Mr. H. N. Singh of our Geotechnical Section at FTS 226-2227. Resolution of discrepancies and concerns will depend on the expeditious receipt of the information mentioned in our review comments in paragraph 4.

FOR THE DISTRICT ENGINEER:

Posey Mill for
P. McCALLISTER
Chief, Engineering Division