6/B3

10/14/82
1 of 1
J. Kane

Subject: Borings in area of service water piping (near CWIS and SWPS) that have indicated potential for liquefaction above El. 610

See 50.54(f) Vol. 8 for boring logs

Boring No.	Date Drilled	Elev. of loose sand	Uncorrected Record. blow counts
CH-2	7/11/79	624-616	4,4
CH-4	7/16/79	625-621	4
CH-5	7/16/79	616-610	9
CH-6	7/17/79	624-610	5,6
Q-7	10/7/78	619-613	7,13,7
SW-7	3/15/79	622-617	12,9
PD-27	12/28/79	624-619	8,4

For these elevations & assuming a = 0.19g GWT @ El. 610, we would need uncorrected blow counts ranging from 10 to 14 to have a min. F.S=1.5

The staff come to recognize that loose sands did exist in the CWIS a d SWPS areas following its review of the logs of borings which had been drilled by the applicant in 1979 following the discovery of the plant fill problem.

DEPARTMENT OF THE ARMY
WATERWAYS EXPERIMENT STATION, CORPS OF ENGINEERS
P. O. BOX 631
VICKSBURG, MISSISSIPPI 39180

IN REPLY REFER TO: WESGA

30 May 1980

MEMORANDUM FOR RECORD

SUBJECT: Visit to Midland Michigan NPP on 27-28 February 1980, A Review of the Midland Plant Units 1 and 2 FSAR (Including Revisions 1-27)

Background and scope

1. The writer visited the Midland Michigan Nuclear Power Plant on 27-28 February in the company of NRC and COE representatives. Bechtel and Consumers Power Company representatives briefed us on 27 February. The attendance list is given in Incl 1. On 28 February we toured several areas of the plant in small groups, were briefed by Bechtel's consultants (see Incl 1) and had an opportunity to ask questions. Inclosure 2 is the agenda for the meeting.

2. The Detroit District of the Corps of Engineers is assisting the Site Analysis Branch of NRC with review of geotechnical aspects of the project relating to safety. My involvement is in support of Detroit District and by prior agreement with the District is limited to geotechnical earthquake engineering issues.

3. Subsequent to the visit, I reviewed the Midland Units FSAR Volumes 1-4 and Volume 7 in a cursory fashion and Sections 2.5-2.56 of the FSAR in detail. The documents I received were complete up through Revision 27. I also performed some analyses whose results are summarized in the following paragraphs and reviewed Volumes 1-7 of "Response to NRC Questions Regarding Plant Fill."

Comments regarding liquefaction potential

4. 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:

Uncorrected blow counts

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

See Encl. 3 curve & calculations how determined

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

WESGA 30 May 1980

SUBJECT: Visit to Midland Michigan NPP on 27-28 February 1980, A Review of the Midland Plant Units 1 and 2 FSAR (Including Revisions 1-27)

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. The curve described in the above tabulation is compared to those for other groundwater tables and earthquake loading conditions in Incl 3.

omitted in Detroit's Repor

5. All of the plotted boring logs of the plant fill area furnished to me by the Detroit District, CE, were reviewed. Out of over 250 standard penetration tests on cohesionless plant fill or natural foundation material below elevation 610 which are shown in Incl 4, the criteria given above are not satisfied in four tests on natural materials located below the plant fill and in 23 tests located in the plant fill. These tests are listed in Incl 5. Some of the tests on natural material (N in the table) were conducted at depths of at less than 10 ft before approximately 35 ft of fill was placed over the location. Those tests are identified by the symbol B and prior to comparison with the criteria 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.

pg. 13 & 14 of Detroit Report

6. 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.

7. In view of the large number of borings in the plant fill area and the conservatism adopted in my 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.

pg. 10 of Detroit Rpt

8. In order to provide the necessary assurance of safety against liquefaction it is necessary to demonstrate the water will not rise above elevation 610 during normal operations or during a shutdown process and 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 as plant fill, different amounts of time are available to accomplish repair or shutdown.

Pg. 9 of Detroit Rpt

9. In response to Question 24 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. Question A, B, and C, which I would like posed to the applicant are as follows:

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?

Pg. 11 of Detroit Rpt

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 answer 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 are and/or where will be the observation wells in the plant fill area 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?

10. 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 in any well reaches elevation 610 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. Question D is as follows:

D. If a dewatering system failure or degradation occurs, in order to assure that plant is shutdown by the time water level reaches elevation 610, it is necessary to initiate shutdown earlier. In

Pg. 12 of Detroit Rpt.

event of failure of 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.

Comments regarding seismically induced settlements

Not presented in Detroit Rpt

11. An independent approximate analysis based on the same references cited on pages 4-5 of the answer to Question 4 given in "Responses to NRC Requests Regarding Plant Fill," the same assumption of dry sand used in the preparation of Table 4-1A of Question 4 and my engineering judgment indicated that the numbers for seismically induced settlement in that table which are for 0.12 g and M = 7 earthquake are also reasonable for 0.19 g and a Magnitude 6 event. However, Seed and Silver (Reference 1 on pages 4-5) claim the limited field check data for the method only confirms its accuracy ±50 percent. Thus, one has to either argue that the capillary action in those sands above the water table would inhibit settlements and thus provide the degree of conservatism needed to overcome the uncertainty about the accuracy of the prediction (as did the applicant in his response to Question 4) or allow for another 1/4 in. of settlement. While this latter course of action is probably available to the applicant at no cost, it is, in my opinion, unneccessary. In view of the field data discussed in the references cited on pages 4-5 of the applicant's answer to Question 4, I am fully satisfied that capillary action does provide all the conservatism needed to view the seismically induced settlements in Table 4-1A as upper bound values for the earthquake shaking described above. Should we ask CPCo whether involved structures can tolerate the additional 1/4" settlement under seismic loading

Comments regarding the natural slopes containing the R/C pipe service water return lines

Pg. 9 of Detroit Report

12. The two reinforced concrete return pipes which exit the service water structure and run along either side of the emergency cooling water reservoir and ultimately enter into the reservoir are necessary for the safe shutdown and are buried within or near the crest of Category I slopes that form the sides of the Emergency Cooling Water Reservoir. The reviewer has been unable to find any report on or analysis of the seismic stability or calculation of postearthquake 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, Question E is as follows:

pg. 9&10 of Detroit Report

E. Have seismic analyses of the slopes leading to an estimate of the permanent deformation of the pipes been performed and if so, please provide a review copy. If none are available, please provide analyses to include the following: (1) a plan showing the pipe location with respect to other nearby structures, the slopes of the reservoir and the coordinate system; (2) cross-sections showing the pipes, normal pool levels, the slopes, the 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 of 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.

Comments regarding the service water structure foundation

Modified in Detroit Report pg. 4

omitted

13. 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, provided proposed 100 ton ultimate pile load capacities are achieved. I have no reason to think they won't be achieved at this time, and the applicant has committed to a field loading test to demonstrate the pile capacity. Calculations were made by the writer to determine the critical buckling load for the 14 in. outside diam concrete filled steel pipe piles assuming them to be laterally unsupported over lengths of 40 and 50 ft with all reasonable assumptions of end fixity and a 3/8-in. pipe thickness. The worst combination of parameters still provides a generous factor of safety against buckling under the proposed ultimate load. Hence, even if the fill material underneath the overhang should liquefy and fail to provide lateral support to the piles, they should be capable of carrying the vertical static and inertial loads anticipated. Fully adequate lateral support is provided by structural connection of the overhang to the rest of the structure. However, 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, Question F is as follows:

Detroit Report pg. 5

F(a). Please summarize or provide copies of reports on the dynamic analyses of the structure in its old and proposed configuration if such are available. 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.

Detroit Report pg. 5

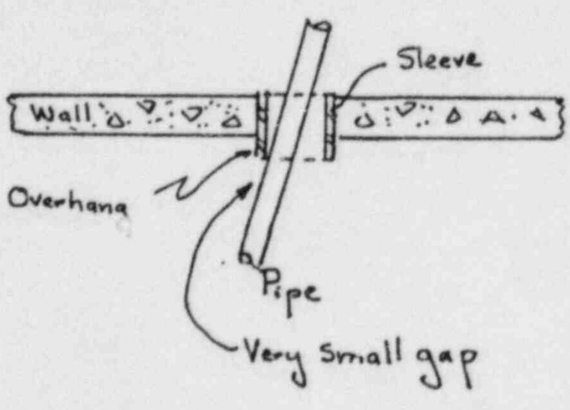
F(b). Provide after completion of the new pile foundation, in accordance with commitment No. 6, item 125, Consumers Power Company memorandum dated 13 March 1980, the results of measurements of vertical applied load and absolute pile head vertical deformation which will be made when the structural load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

Comments regarding rattlespace at Category I pipe penetrations of structure walls

14. During the site visit the writer observed three instances of what appeared to be degradation of rattlespace at penetrations of Category I piping through concrete walls as follows:

a. West borated water storage tank - 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.

Detroit Report pg. 8



b & c. 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 I could not see) 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, I have no way of knowing.

Detroit Repor pg. 8 & 9

These instances are, in my view, 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. Therefore, Questions G and H are as follows:

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

H. 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 H 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.

<u>Comments regarding foundation material properties used in seismic analysis of structures</u>

Detroit Report pg. 14

15. Inclosure 6 shows a summary of cross-hole shear wave velocity (V_s) and load test data from which it can be seen that the V_s for the plant fill is between 500 and 1000 ft/sec. 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 analyses 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 as indicated in Incl 6. It is understood from the response to Question 13 concerning plant fill that the analyses 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 analyses. 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. Questions I, J, and K are as follows:

I. What 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?

J. 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.

shear wave velocity — poisson ratio — mass density

K. 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.

Detroit Report 19.15

Category I retaining wall near the southeast of the service water pump structure

16. 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. Questions L, M. and N are as follows:

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

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

N. 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.

Status of review of geotechnical earthquake considerations

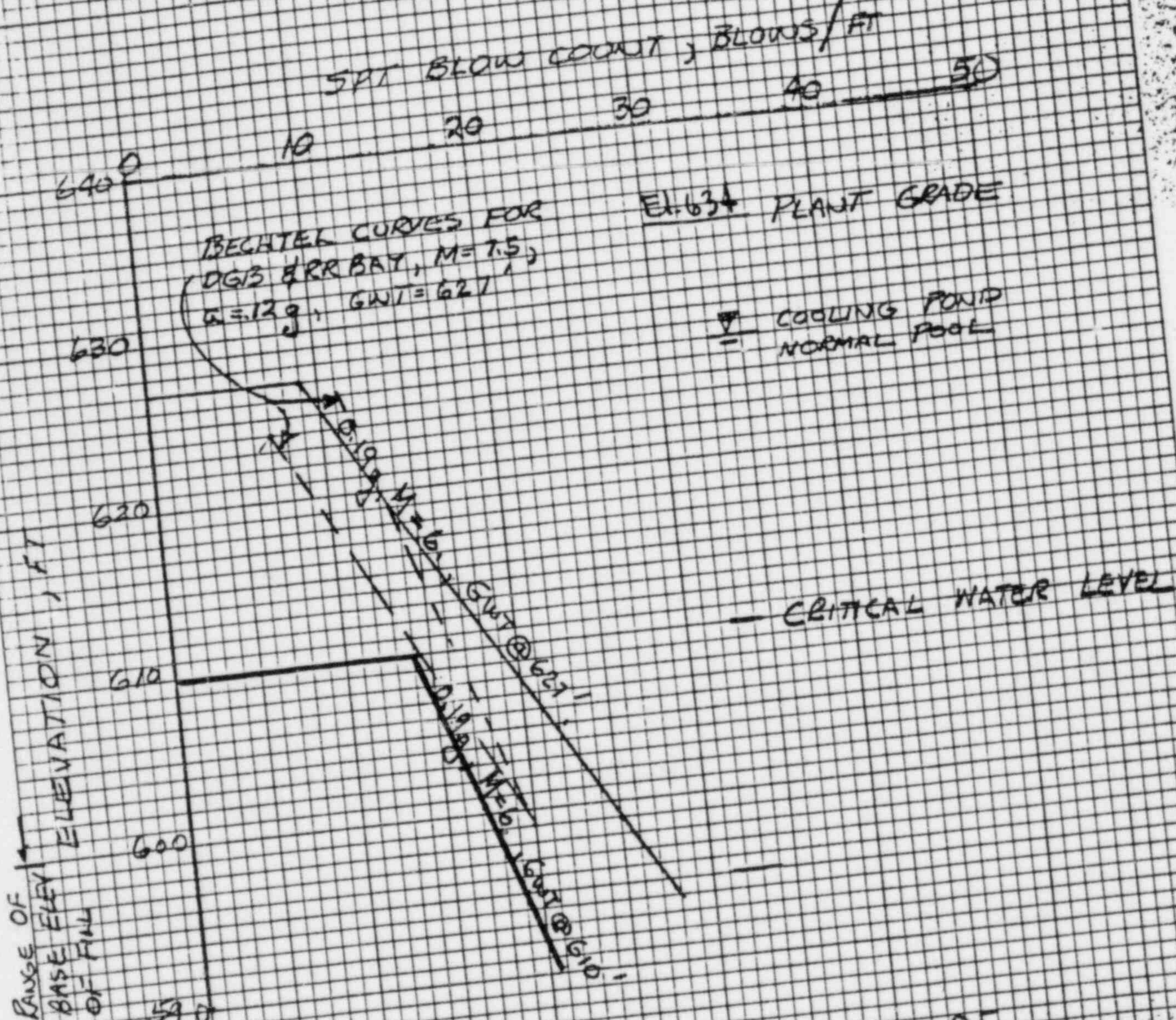
Not included in Detroit Report

17. When formal or informal answers to the questions posed above are available from the applicant, this reviewer can quickly come to conclusions on all geotechnical considerations which influence safety under earthquake excitation. It would be desirable but not mandatory to witness the service water pump structure pile load test and the jacking of that building's load onto the completed piles.

Paul F. Hadala
P. F. HADALA
Engineer
Acting Assistant Chief,
Geotechnical Laboratory

6 Incl
as

CF w/incl:
Mr. Neil Gehring, Detroit Dist
Dr. Lyman Heller/Mr. Joe Kane, NRC
Mr. Jim Simpson, North Central Div



SPT REQUIRED FOR ASSURANCE OF F.S = 1.5 AGAINST LIQUEFACTION OF COHESIONLESS SOILS BY SEED-IDRISS SIMPLIFIED PROCEDURE

Encl 3

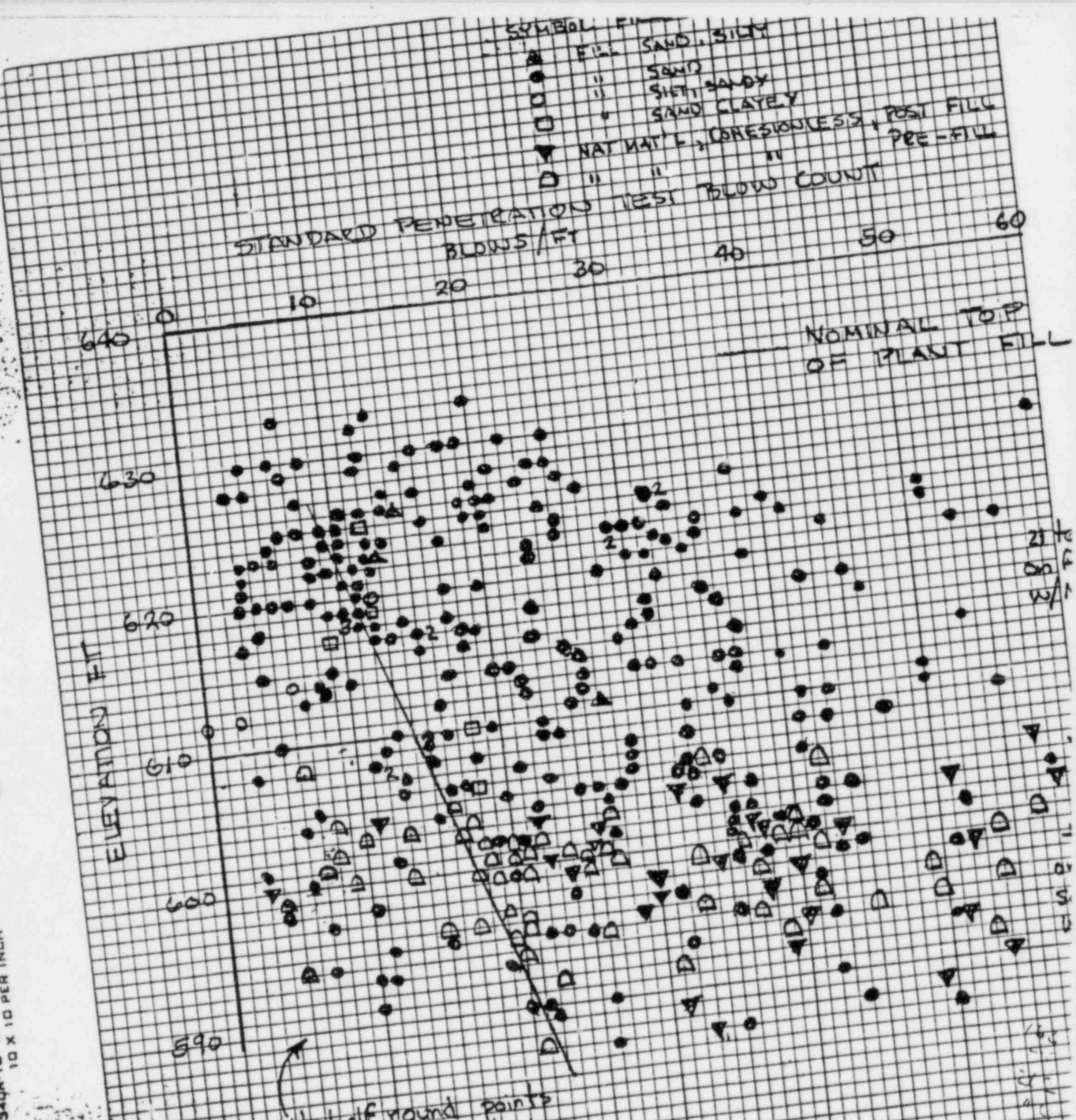
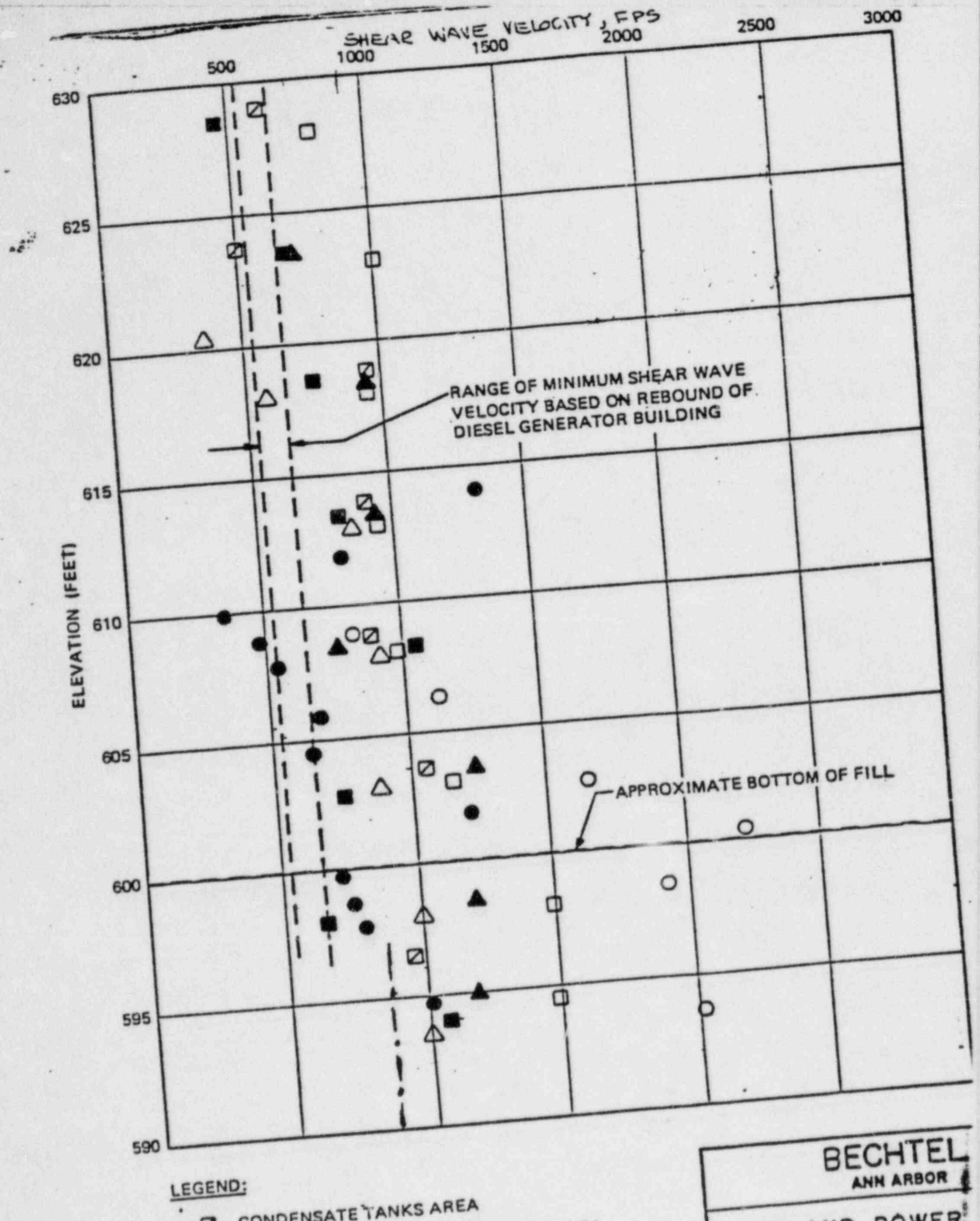


FIGURE Blow Count vs Elevation for STD. Penetration Tests in Sandy Soils in the Plant Fill Area

Encl 4



BECHTEL
ANN ARBOR

MIDLAND POWER

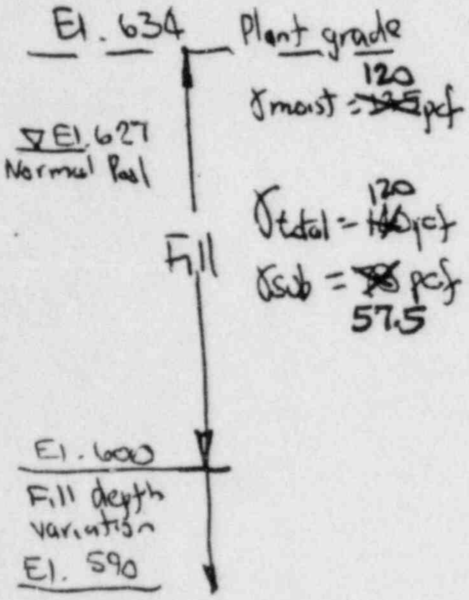
SHEAR WAVE VELOCITY
PLANT AREA FIL

JOB NO.	DRA
7220	FIGURE

Subject: Liquefaction Analysis - Midland

Method - Seed - Idriss Simplified Analysis

<u>Assumptions</u> - GWT was at or BELOW elev. 610
- Peak ground surface acceleration = 0.19 g
- required factor of safety = 1.5 against liquefaction type failure
- Magnitude 6 earthquake



Find - What uncorrected blow count (N) is required to have a F.S. = 1.5 @ El. 610 when GWT is @ Elev. 610 when $a_{max} = 0.19g$?

1. Solve for cyclic stress ratio causing liquefaction τ/σ_o' for use in Seed's curves

$$\frac{\tau}{\sigma_o'} = \text{cyclic stress ratio} = \frac{\tau_{avg}}{\sigma_o'} = 0.65 \times \frac{\overset{a_{max}}{0.19g}}{g} \times \frac{\sigma_o}{\sigma_o'} \cdot r_d$$

Where a_{max} = peak ground surface acceleration

r_d = stress reduction factor (in this case @ 24' depth)

σ_o = total overburden pressure (in this case @ El. 610)

σ_o' = effective overburden pressure @ El-610

5/17/82
J. Kane
2 of 3

Method believed to be used by Paul Hadala

$$\frac{\tau_{avg}}{\sigma_o'} = 0.65 \times \frac{a_{max}}{g} \times \frac{\sigma_o}{\sigma_o'} \cdot r_d$$

$$= 0.65 \times \frac{0.19\cancel{g}}{\cancel{g}} \times \frac{(120\,lb/ft^3 \times 24\,ft)}{(120\,lb/ft^3 \times 24\,ft)} \times 0.94$$

$$\frac{\tau_{avg}}{\sigma_o'} = 0.116$$

From Seed's curves w/ $M=6$ & $\frac{\tau_{avg}}{\sigma_o'} = 0.116$

the corrected blow count $N_1 = 8$ blow/ft

Since $N_1 = C_N \cdot N$

$$N = \frac{N_1}{C_N}$$

where $C_N = 1 - 1.25 \log \frac{\sigma_o'}{\sigma_1}$

$$C_N = 1 - 1.25 \log \overset{0.158}{\frac{1.44}{1.0}}$$

$C_N = 1 - 0.198$

$C_N = 0.802$

$\sigma_o' = 120\,lb/ft^3 \times 24\,ft$

$= 2880\,lb/ft^2$

$= 1.44$ tsf

$\sigma_1 = 1$ tsf

$$N = \frac{N_1}{C_N} = \frac{8}{0.802} = 9.97 \text{ Say } 10$$

w/ F.S. = 1.5 $\quad 1.5 \times N = 1.5 \times 10 = 15$ blows/ft rd @ 24' depth = 0.94

$$\frac{\tau_{avg}}{\sigma_o'} = 0.65 \times \frac{a_{max}}{g} \times \frac{\sigma_o}{\sigma_o'} \cdot rd$$

$$= 0.65 \times \frac{0.19g}{g} \times \frac{(120\,lb/ft^3 \times 24\,ft)}{(120\,lb/ft^3 \times 24\,ft)} \cdot 0.94$$

$$\frac{\tau_{avg}}{\sigma_o'} = 0.116$$

For F.S. = 1.5 —— Here Factor of safety is being applied to cyclic stress ratio

$1.5 \times 0.116 = 0.174$

For M = 6 from Seed's curves w/ $\frac{\tau_{avg}}{\sigma_o} = 0.174$

the corrected blow count $N_1 = 12.5$ blows/ft

$N_1 = C_N \cdot N$

$N = \frac{N_1}{C_N}$

$\sigma_o' = 120\,lb/ft^3 \times 24\,ft = 2880$ psf

$\sigma_o' = 1.44$ tsf

$\sigma_1 = 1$ tsf

where $C_N = 1 - 1.25 \log \frac{\sigma_o'}{\sigma_1}$

$C_N = 1 - 1.25 \log \frac{1.44}{1.0}$

$C_N = 1 - 1.25(.158)$

$C_N = 1 - 0.198$

$C_N = 0.802$

$N = \frac{N_1}{C_N} = \frac{12.5}{0.802} = 15.6$

Adopt N = 16

7/83

J. Kane
Rec'd. 10/21/82

CPC

Testimony of
Dr. Richard D. Woods
Regarding
Liquefaction of Saturated Sand
During an Earthquake
At The Midland Site

~~8210210491~~

Coordinate location of loose sands w/ Dr. Hendron's testimony

8210210491

General Offices: 212 West Michigan Avenue, Jackson, Michigan 49201 • (517) 788-0550

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MIDLAND PROJECT -
MIDLAND DOCKET NO 50-329, 50-330
TESTIMONIES OF WC PARIS AND DR RD WOODS

Attached please find the testimony of William C Paris concerning the permanent dewatering system for the Midland site. Also attached is the testimony of Dr Richard D Woods concerning liquefaction potential at Midland.

The testimony of Dr Woods determines and describes areas of the site for which a dewatering system will operate to prevent possible liquefaction during a design basis safe shutdown earthquake. Mr Paris' testimony describes the design, construction, and operation of the system to dewater the areas identified in Dr Woods' testimony as potentially liquefiable.

Dr Woods has indicated that, because of the hospitalization of an associate, he will be available to testify only on Wednesday, November 3, 1982, in the afternoon, and for a short time on the morning of Thursday, November 4, 1982. We request your consideration in trying to accommodate Dr Woods' schedule problems.

James E Brunner

James E Brunner
Attorney for Consumers
Power Company

JEB/jsn

~~8210210481~~ 4pp.

UNITED STATES OF AMERICA
NUCLEAR REGULATORY COMMISSION

ATOMIC SAFETY AND LICENSING BOARD

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

CERTIFICATE OF SERVICE

I hereby certify that copies of the "Testimony of William C. Paris, Jr. on Behalf of the Applicant Regarding Permanent Dewatering System for the Midland Site" and "Testimony of Dr. Richard D. Woods on Behalf of the Applicant Regarding Liquefaction of Saturated Sand During an Earthquake at the Midland Site" in the above-captioned proceeding were served on the persons listed in the attached Service List either by deposit in the U.S. Mail, First Class, postage prepaid, or by Federal Express as indicated in the Service List, on the 18th day of October, 1982.

Robert L. Rixford
Robert L. Rixford
Bechtel Associates Professional
Corporation

Sworn and Subscribed Before Me this 18 Day of October, 1982

Beverly A. Bross
Notary Public
Washtenaw County, Michigan

My Commission Expires November 30, 1982

BEVERLY A. BROSS
NOTARY PUBLIC, WASHTENAW CO., MICH
MY COMMISSION EXPIRES NOV. 30, 1982

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FEDERAL EXPRESS

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FEDERAL EXPRESS

UNITED STATES OF AMERICA
NUCLEAR REGULATORY COMMISSION

BEFORE THE

ATOMIC SAFETY AND LICENSING BOARD

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

TESTIMONY

OF

DR. RICHARD D. WOODS

ON BEHALF OF THE APPLICANT

REGARDING LIQUEFACTION POTENTIAL OF SATURATED SAND

DURING AN EARTHQUAKE AT THE MIDLAND SITE

~~8210210491~~
62pp.

SS: STATE OF MICHIGAN
COUNTY OF WASHTENAW

UNITED SATES OF AMERICA
NUCLEAR REGULATORY COMMISSION

ATOMIC SAFETY AND LICENSING BOARD

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

AFFIDAVIT OF RICHARD D. WOODS

Richard D. Woods, being duly sworn, deposes and says that he is the author of "Testimony of Richard D. Woods concerning Liquefaction Potential at the Midland Site," and that such testimony is true and accurate to the best of his knowledge and belief.

Richard D. Woods
RICHARD D. WOODS

Sworn and Subscribed Before Me this 15 Day of October, 1982

Beverly A Bross
Notary Public
Washtenaw County, Michigan

My Commission Expires November 30, 1982

BEVERLY A. BROSS
NOTARY PUBLIC, WASHTENAW CO., MICH
MY COMMISSION EXPIRES NOV. 30, 1982

LIQUEFACTION OF SATURATED SAND DURING EARTHQUAKE

1.0 BIOGRAPHICAL INFORMATION

This is the testimony of Dr. Richard D. Woods. My detailed resume is attached. The following is a summary of that resume. I received a Bachelor of Science degree in Civil Engineering from Notre Dame University in 1957 and a Master of Science degree from the same school in 1962. I worked for the Air Force Weapons Center, Albuquerque, New Mexico, on the design of blast resistant underground structures for one year and taught in the Civil Engineering Department at Michigan Technological University for one year before going to the University of Michigan for a Ph.D. in Civil Engineering, which I received in 1967. Since then I have been on the faculty of the Department of Civil Engineering at the University of Michigan, advancing to full Professor in 1976. My research interests have been in the field of soil dynamics and earthquake engineering. I have done part-time consulting in the fields of soil dynamics, earthquake engineering, structural vibrations, and general foundation engineering. My clients have included Bechtel, Corning Glass Works, Rockwell International, Eaton Corporation, TAMS, General Motors, Honeywell Inc., Woodward-Clyde Consultants, and Nuclen (Nuclear Brazil). I have directed research associated with liquefaction phenomena sponsored by the National Science Foundation and have been a consultant to Bechtel, TAMS, Woodward-Clyde, and Nuclen on liquefaction issues. I am a principal in the foundation consulting firm of Stoll, Evans, Woods, and Associates, Ann Arbor, Michigan and am a member of ASCE, ASEE, ASTM, and SSA.

2.0 INTRODUCTION

My testimony is concerned with the evaluation of the potential for liquefaction of loose sands in the plant area at the Midland plant. The liquefaction potential was evaluated using the simplified method based on blowcount as presented by Seed. The maximum ground acceleration was taken as 0.19g and a Richter magnitude of 6.0 was used to correlate with about 5 cycles of significant stress reversal for the Midland site. On the basis of my analysis and the proposed remedial measures, I have concluded that there is reasonable assurance that the plant area is safe with respect to liquefaction of the sand.

3.0 DISCUSSION

When earthquake excitation is part of the design loads for a structure or facility, the potential for liquefaction of any saturated loose sands supporting the structure must be evaluated. Liquefaction is the phenomenon by which cohesionless soil loses shearing strength because of ground shaking and develops a degree of mobility sufficient to permit large permanent displacements or liquid-like flow behavior. Some common manifestations of liquefaction include settlement and tilting of structures, cracking and lateral spreading of slopes and embankments, flow type failures of natural slopes and embankments, and sand boils or sand volcanos.

Whether or not a specific sand formation will liquefy depends on several factors associated with the soil and the earthquake. The primary consideration is whether or not loose sands occur below the groundwater table (GWT). Unless the sands are saturated, there will be no buildup of excess pore pressure or loss of shearing strength associated with the ground shaking. However, if the sands are dense, they will not liquefy even if they are below the GWT. The measure of denseness used in the analysis of liquefaction potential is called relative density. Other factors that influence the potential for liquefaction include the effective confining pressure on the sand and the intensity and the duration of ground shaking. Large, effective confining pressures reduce the potential for liquefaction, whereas more intense and longer durations of shaking increase the potential for liquefaction.

Sands that must be evaluated for liquefaction potential exist in several locations at the Midland plant. Some areas are concentrated under or around Category I structures, whereas other areas are distributed and support embedded pipelines and duct banks. Several techniques are used to remedy the susceptibility of certain sands to liquefaction, depending on their locations and extent. These include preventing saturation of the sand by lowering the GWT and total removal and replacement of the sand with materials that are not subject to liquefaction.

4.0 EVALUATION OF LIQUEFACTION POTENTIAL

Based on the factors influencing the potential for liquefaction, Seed and Idriss (1971) and Seed (1979) proposed an empirical method for evaluating the liquefaction potential for sands at level ground sites. Their method is based on the performance of sand deposits having certain known characteristics in previous earthquakes and a comparison with sands of measured characteristics at the new site when subjected to a specified design earthquake. For any specified location in a sand deposit, a key factor called the cyclic stress ratio can be estimated and is based on site conditions and the specified maximum ground surface acceleration. The relative density of the sand (as indicated by standard blowcount) required to sustain a certain minimum number of cycles of that cyclic stress ratio without liquefaction can be estimated from the experience gained from previous earthquakes. If the in situ standard blowcount at the specified location meets or exceeds the estimated blowcount, no potential for liquefaction exists.

The computations required to perform this evaluation are as follows:

a. Estimate cyclic stress ratio ($\tau av/\sigma_o'$)

$$(\tau av/\sigma_o') = 0.65 \frac{a_{max}}{g} \frac{\sigma_o}{\sigma_o'} \times r_d \qquad (1)$$

where

τ_{av} = average horizontal shearing stress induced by earthquake

a_{max} = maximum horizontal acceleration at ground surface

σ_o = total overburden pressure on sand

σ_o' = initial effective overburden pressure on sand

r_d = stress reduction factor

g = acceleration of gravity

b. Estimate in situ blowcount required to preclude liquefaction.

Values of cyclic stress ratio have been correlated with a modified penetration resistance (N_1) at sites that have and have not liquefied during actual earthquakes. For earthquakes of a Richter magnitude of 6.0,* this correlation is shown in Figure L-1, where all points on and to the right of the curve are safe with respect to liquefaction. The modified penetration resistance is related to standard penetration resistance by:

$$N_1 = C_N N \qquad (2)$$

where

N_1 = modified penetration resistance

C_N = a function of effective overburden pressure and relative density as shown in Figure L-2 (use curve for D_r 40 to 60%)

N = standard penetration resistance

*This magnitude was selected to provide a close correlation, based on number of cycles, with the Midland SSE.

c. Compare N computed from Equation (2) with N in situ.

If the standard penetration resistance measured at a specific location in the ground is equal to or exceeds N computed from Equation (2), the sand at that location will not liquefy under the design excitation.

In the above method of evaluating the potential for a specific sand to liquefy, both the intensity of earthquake shaking and the duration of the earthquake are considered. The intensity is included in Equation (1) for cyclic stress ratio where a maximum ground acceleration of 0.19 g has been used and the number of cycles of significant stress is covered by selection of the curve in Figure L-1, in this case, the curve for an earthquake of a Richter magnitude of 6.0.

This method of liquefaction evaluation presumes that the sand at the specific location being examined is saturated. Therefore, one method of preventing liquefaction is to drain the sand by lowering the GWT. Initial computations showed that some strata or pockets of sand would be susceptible to liquefaction with the GWT at elevation 627 feet, but that by lowering the GWT to 610 feet or below, the potential for liquefaction could be eliminated.

5.0 RESULTS OF EVALUATIONS OF LIQUEFACTION POTENTIAL

Sands for which the potential for liquefaction had to be evaluated occur under portions of two Category I structures and at some other locations around the plant site where pipelines and duct banks are buried. The key parameter reflecting the condition of the sand as measured in situ at each location is the standard penetration resistance, N. N was measured at various elevations in borings throughout the plant site. The locations of all plant site borings including those used in this evaluation of liquefaction potential are shown in Figures L-3, L-4, and L-5.

The method by which the liquefaction potential is resolved for the various locations is described separately in the following paragraphs.

5.1 DIESEL GENERATOR BUILDING AREA

Liquefaction evaluation of sand in this area is based on the blowcount and relative density data obtained from various investigations. Bechtel test borings drilled in September and October 1978 (DG series) and November 1979 (CH series) provided blowcount information before and after placement of surcharge, respectively. Additional data on blowcount were obtained from the Woodward-Clyde Consultants relative density data (FSAR Appendix 2H). These data were obtained during the fill investigation and are based on the COE series borings performed around the diesel generator building in April 1981. The boring location plan of the diesel generator building area is presented in Figure L-4.

Studies of the liquefaction potential are illustrated by the blowcounts versus elevation plots presented in Figures L-6 through L-8. Each figure has two sets of curves representing two GWT elevations (610 and 627 feet) and two factors of safety (1.0 and 1.5). The left-side curves form an approximate boundary that separates liquefaction from no liquefaction zones (i.e., FS = 1.0). The curve on the right represents a boundary of the no-liquefaction condition with a safety factor of 1.5.

The factor of safety as used here means that the cyclic stress ratio computed from Equation (1) was multiplied by 1.5, and then the standard penetration resitance required to satisfy the higher cyclic stress ratio was determined.

Liquefaction is not possible above the GWT, and with the GWT lowered to elevation 610 feet or lower, only two locations beneath the structure representing separate pockets of sand show blowcounts that are potentially liquefiable (Figure L-6). Because of the limited extent of these pockets, they should have no effect on the stability of the structure. Penetration resistance for all other locations representing the major portion of the volume of sand under the diesel generator building (Figures L-6 through L-8) indicates that the sands are safe with respect to liquefaction.

5.2 RAILROAD BAY AREA OF AUXILIARY BUILDING

Three of the Bechtel AX series borings represent soil conditions beneath the railroad bay of the auxiliary building (see Figure L-3). The liquefaction analysis of the sand in this area is presented in the blowcounts versus elevation plot in Figure L-9. The lower set of curves in this figure for factors of safety of 1.0 and 1.5 show that only one location beneath the building had a factor of safety less than 1.5, so liquefaction is not a problem when the GWT is maintained at elevation 610 feet or lower.

5.3 OTHER AREAS

Sands in the plant area outside the diesel generator building and the railroad bay area of the auxiliary building were analyzed for liquefaction potential by separately evaluating three horizontal strata: below elevation 605 feet, between elevations 605 and 610 feet, and above elevation 610 feet.

5.3.1 Plant Area Natural Sands Below Elevation 605 Feet

Sands existing below elevation 605 feet are primarily natural sands, although some fill sands were also placed in backfill around deep structures below elevation 605 feet. To evaluate the liquefaction potential of these sands, the standard penetration resistance in situ was compared with that required to prevent liquefaction, which was computed as described in Section 3.0 using a factor of safety of 1.5. This analysis showed that the sands in the plant area below elevation 605 feet have a few pockets with in situ blowcounts lower than required. The location of these pockets are identified in Figure L-10 with pertinent data from the analysis also shown in the figure. Table L-1 lists all borings in which low-blowcount sands were identified and shows the low-blowcount sands in relation to the other soils above and below.

Some of the low-blowcount pockets are not located near any Category I structure, pipeline, or duct bank. The remaining pockets represent single isolated blowcounts surrounded by soils with significantly higher blowcounts above and below or by nonliquefiable soils above and below (e.g., see boring CT-1, elevation 602.0 feet, Figure L-10, and Table L-1).

Based on this analysis, the natural sands below elevation 605 feet throughout the plant area present no hazard due to liquefaction.

5.3.2 Plant Area Fill Sand Between Elevations 605 and 610 Feet

Sands between elevations 605 and 610 feet are mainly fill sands, but relatively small, localized pockets of natural sands were also encountered in this elevation range. Sands in this stratum were analyzed in the manner described in Section 5.3.1. That analysis showed that scattered pockets of low-blowcount sand exist in the fill. The locations of borings in which these low-blowcount sand pockets were found are shown in Figure L-11, and Table L-2 lists those borings and contains pertinent data relative to the analysis and resolution of liquefaction potential in the low-blowcount sand pockets.

Some of these low-blowcount pockets are located such that they do not affect the stability of Category I structures; some are within zones that will be excavated and backfilled; the remaining are located between high-blowcount sands or other nonliquefiable soils.

Based on this analysis, the fill sands between elevations 605 and 610 feet do not constitute a liquefaction hazard.

5.3.3 Plant Area Sand Between Elevations 610 and 627 Feet Outside of Both Diesel Generator Building and Railroad Bay of the Auxiliary Building

Sands between elevations 610 and 627 feet are fill material. The susceptibility to liquefaction of any loose sands in this stratum depends on their location relative to the permanently dewatered regions as well as other factors.

The locations of borings in which pockets of low-blowcount sands have been identified are shown in Figure L-12. The low-blowcount sand pockets were analyzed for liquefaction potential in the manner described in Section 5.3.1. Table L-3 lists the borings shown in Figure L-12 and provides pertinent data relative to the analysis and resolution of liquefaction potential in low-blowcount pockets.

Two of the areas in this stratum where several pockets of low-blowcount sands occur were south of the diesel generator building and northeast of the railroad bay area. Both of these areas will be within the zone of dewatering and therefore not subject to liquefaction. Another area with pockets of low-blowcount sand occurs northwest of the service water pump structure and the circulating water intake structure. The zones where these sand pockets exist will be excavated to elevation 61) feet and replaced with suitable backfill. Other pockets are bounded by higher blowcount or nonliquefiable materials. Finally, some low-blowcount sand pockets are outside the area and do not influence the stability of structures.

6.0 SUMMARY AND CONCLUSIONS

Limited pockets of loose natural sand and loose fill sand exist in the plant area and under two Category I structures at the Midland plant. The potential for these sands to liquefy during an earthquake with a maximum ground acceleration of 0.19 g and Richter magnitude 6.0 has been evaluated.

For most of the sand pockets which exhibited a potential for liquefaction, remedies are provided which eliminate the potential by permanently lowering the GWT or by totally removing the loose sands and replacing them with suitable materials. For other sand pockets, liquefaction is not a hazard because they occur in location where they do not influence any Category I structures. The remaining pockets are situated in limited zones between other nonliquefiable soils and therefore present no hazard.

Because of the widely scattered occurrence of the loose sand pockets in the plant area, the potential for liquefaction was small before remedial measures were adopted; therefore, after the implementation of remedial measures, the plant area will be safe with respect to liquefaction of the sands.

7.0 REFERENCES

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2. Seed, H.B. (1979), "Soil Liquefaction and Cyclic Mobility Evaluation for Level Ground During Earthquakes," Journal of the Geotechnical Engineering Division, Proceedings of the American Society of Civil Engineers, Volume 105, No. GT2 (Feburary), pp 201-255

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RÉSUMÉ

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August, 1980

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PERSONAL DATA

Age:	45, born U.S. citizen
Physical:	Height 6'; weight 220 lb
Health:	Excellent
Military:	U.S. Marines
Married:	Wife, Dixie Lee (Davis)
	Daughter, Kathleen Ann, age 23
	Daughter, Cecilia Marie, age 15
	Daughter, Karen Teresa, age 12

EDUCATION

High School, J. W. Sexton, Lansing, Michigan, 1953
B.S. Civil Engineering, University of Notre Dame, 1957
M.S. Civil Engineering, University of Notre Dame, 1962
Introductory (non-degree) Course, ASEE-AEC Basic Institute in Nuclear Engineering, North Carolina State College, 1964
Ph.D. Civil Engineering, University of Michigan, 1967

ORGANIZATIONS

American Society of Civil Engineers
American Society for Testing and Materials
American Society for Engineering Education
Chi Epsilon
Society of the Sigma Xi
Seismological Society of America

AWARD

Collingwood Prize of American Society of Civil Engineers, 1969

EMPLOYMENT (Full Time)

1976 to Present	Professor, Civil Engineering, University of Michigan. Courses taught: Basic Soil Mechanics, Field Sampling and Laboratory Testing of Soils, Foundation Engineering, Soil Dynamics, Civil Engineering Dynamics Measurements, Plane Surveying, Statics and Strength of Materials, Reinforced Concrete. Research performed: See separate paragraph below.
1971 to 1976	Associate Professor, Civil Engineering, University of Michigan. Courses taught: Included above.
1967 to 1971	Assistant Professor, Civil Engineering, University of Michigan. Courses taught: Included above.
1965 to 1967	Graduate Student, University of Michigan, supported on NSF Traineeship.
1964	Instructor, Civil Engineering, Michigan Technological University, Houghton, Michigan. Courses taught: Included above.
1963	Project Engineer (GS-11), Air Force Weapons Laboratory, Kirtland, AFB, Albuquerque, N.M. Supervised contracts which were directed at determining engineering properties of soils under dynamic loads.
1960 to 1962	Graduate Student, University of Notre Dame, teaching assistantship, taught surveying camp.
1957 to 1960	Lieutenant, U.S. Marine Corps, Camp Pendleton, California. Six months as platoon leader, movable bridge company. Remainder of service as hydraulic engineering officer preparing evidence for water rights litigation.

EMPLOYMENT (Short Courses and Special Appointments)

1976 Fugro Fellow, University of Florida. On sabbatical leave from University of Michigan. Investigating use of static cone penetrometer with built-in pore pressure transducer to predict liquifaction potential of sands.

1974 Invited Author for Chapter on Soil Dynamics for U.S. Army Corps of Engineers Soils Manual, with F. E. Richart.

1973 Invited Lecturer, Woodward-Clyde Consultants Symposium, Berkeley. Topic: "Seismic Methods to Measure Shear Wave Velocity of Soils and Rock."

1973
1972 Taught Extension Courses (evening), "Applications of Soil Mechanics to Foundation Engineering," 2-10 week lecture series for Commonwealth Associates, Jackson, Michigan.

1972 Visiting Professor, Institute for Soil and Rock Mechanics, University of Karlsruhe, Germany. Taught Soil Dynamics and helped establish soil dynamics laboratory. Research on propagation of Rayleigh Waves in region of obstacles.

1971 Visiting Professor, Indian Institute of Technology, Kanpur, India. Helped establish basic soil dynamics laboratory and field measurements capability.

1971 Invited Lecturer, Earthquake Engineering Seminar, University of Massachusetts, sponsored by National Science Foundation. Lectures on basic vibrations, wave propagation and dynamic soil properties.

1970
1969 Chairman and Principal Lecturer, two 2-day short courses, "Behavior of Soils for the Construction Industry, Continuing Engineering Education Program, College of Engineering, University of Michigan.

1968 Co-Chairman and Lecturer, Two-week short course, "Vibration of Soils and Foundations," Continuing Engineering Education program, College of Engineering, University of Michigan. Lectures on basic vibrations, wave propagation and field and laboratory measurements.

RESEARCH

At University of Michigan

Holographic Interferometry - Investigation of basic wave propagation and surface wave propagation in region of barriers.

Response of Pile Foundations to Dynamic Loads - with F. E. Richart.

Dynamic Properties of Soils - Laboratory and field measurement of compression and shear wave velocity and shear modulus of soils at both low and high amplitudes.

Isolation of Earthwaves by Barriers - Study of effectiveness of trenches and cylindrical holes at screening waves.

Dutch Static Cone Penetrometer - Study of use of penetrometer for identification of soils.

At Michigan Technological University

Mechanics of Slide Dams - Investigation of creation of dams by blasting material from canyon walls.

At Notre Dame University

Preliminary Design of Dynamic Direct Shear Device

CONSULTING EXPERIENCE

Areas of Consulting

Vibration Measurements - on machines, in soil, on structures

Measurement of Dynamic Soil Properties, in lab and in field

Stability of Soil Masses (Reserve Mining tailings delta)

Analysis and Design of foundations for dynamic loads

Site Investigations with Dutch, cone penetrometer

Blasting Damage Evaluations

Blasting Code Drafting

Seismic Site Investigations

Principal Clients

Bechtel Power Corporation, Ann Arbor, Michigan

Attorney General, State of Michigan (Reserve Mining Case)

CONSULTING EXPERIENCE--*Continued*

Giffels and Associates, Detroit, Michigan

Smith, Hinchman and Grylls, Detroit, Michigan

City of Rockwood, Michigan

City of Ann Arbor, Michigan

Honeywell Corporation, Minneapolis, Minnesota

Woodward-Clyde Consultants, Orange, California, Oakland, California and Philadelphia, Pennsylvania

Halpert, Neyer Associates, Farmington, Michigan

U. W. Stoll and Associates, Ann Arbor, Michigan

Eaton Brake Division, Detroit, Michigan

Tippetts-Abbett-McCarthy-Stratton, New York (Tarbela Dam)

Site Engineers, Inc., Cherry Hill and Montclair, New Jersey

Corning Glass Works, Corning, N.Y. and three other plants

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Partos, A., Woods, R.D. and Welsh, J. (1982), "Soil Modification for Relocating Die Forging Operation," International Symposium on Grouting in Geotechnical Engineering, New Orleans, Feb.

Richart, F.E. Jr., and Woods, R.D. (1982), "Foundations for Auto Shredders", Proceedings of International Conference on Soil Dynamics and Earthquake Engineering, Southampton England, July 13-15, Vol. 2, pp.811-824.

See Fig. L-10

TABLE L-1[1]

EVALUATION OF LOW SPT[2] BLOWCOUNTS IN THE PLANT

AREA SANDS BELOW ELEVATION 605 FEET

Boring Number[3]	GSE[4] at Time of Drilling (feet)	SPT Information: Sample Elevation (feet)	Blowcounts: In-situ	Blowcounts: Required For M=6, a=0.19, FS=1.5	Soil Description Other Than Sand	Remarks
AX-13	635.0	595.5	25	-	Sandy clay	High blowcount above and clay below
		593.0	42	-		
		590.5	10	25		
		588.0	17		Silty clay	
		585.5	145	-		
CT-1	634.0	612.0	23	-	Silty clay	High blowcount below and clay above
		607.5	7	-	Silty clay	
		602.0	11	21		
		599.0	24	-		
		597.0	29	-		
DF-5	634.0	606.5	28	-	Silty clay	Clay above and below
		604.0	17	-	Silty clay	
		601.5	8	21		
		599.0	8	-	Sandy clay	
		596.5	10	-	Sandy clay	
DG-7	631.0	602.0	25	-		High blowcount above and clay below
		600.5	17[5]	-		
		599.0	10	21		
		597.5	15	-	Silty clay	
		588.5	43	-	Silty clay	
DG-28	629.0	605.5	16	-		Clay above and high blowcount below
		603.0	15	-	Sandy clay	
		600.5	9	21		
		598.0	37	-		
		595.5	89	-		
Q-12	634.0	607.5	5	-	Silty clay	Not near a structure
		605.0	7	-	Silty clay	
		602.5	13	22		
		600.0	11	23		
		597.5	29	-		
		595.0	75	-		
PD-5B	634.0	605.0	15	-	Silty clay	Clay above and below
		602.5	7	-	Silty clay	
		600.0	4	21		
		597.5	15	-	Silt	
		595.0	27	-	Silty clay	

TABLE L-1 (continued)

Boring Number[3]	GSE[4] at Time of Drilling (feet)	SPT Information: Sample Elevation (feet)	Blowcounts: In-situ	Blowcounts: Required For M=6, a=0.19, FS=1.5	Soil Description Other Than Sand	Remarks
PD-20	634.0	608.5	25	-		Not near a structure
		606.0	19	-		
		603.5	16	22		
		601.0	13	22		
		598.5	52	-		
		596.0	63	-		
PD-20A	634.0	609.0	40	-		Not near a structure
		606.5	23	-		
		604.0	8	21		
		601.5	14	22		
		599.0	50	-		
		596.5	130	-		
PD-20C	634.0	607.0	47	-		Not near a structure
		604.5	30	-		
		602.0	8	22		
		599.5	24	-		
		597.0	63	-		
LOW-9	634.5	605.0	20	-	Silty clay	Clay above and below
		603.0	27	-	Silty clay	
		601.0	9	21		
		599.0	24	-		
		597.0	21	-	Silty clay	
L_N	622.0	595.0	19	-	Sandy clay	Clay above and high blowcount below
		590.5	10	-	Sandy clay	
		586.0	20	22		
		584.5	100+	-		
		582.5	100+	-		

(1) This ta[illegible] excludes the areas directly below the diesel generator building and auxiliary building railroad bay. Blowcounts in these zones are shown in Figures L-6 through L-9.
(2) Standard penetration test
(3) Boring location shown in Figures L-3, L-4, and L-5
(4) Ground surface elevation
(5) Nonstandard spoon used

See Fig. L-11

TABLE L-2[1]

EVALUATION OF LOW SPT[2] BLOWCOUNTS IN THE PLANT AREA FILL
BETWEEN ELEVATIONS 605 AND 610 FEET

Boring[3] Number	GSE[4] at Time of Drilling (feet)	SPT Information: Sample Elevation (feet)	Blowcounts: In-situ	Blowcounts: Required for M=6, a=0.19, FS=1.5	Soil Description Other Than Sand	Remarks
CH-5A	633.8	612.3	6	-		Within excavation zone
		607.3	17	21		
		602.3	30	-	Silty clay	
		597.3	85	-		
PD-20	634.0	611.0	45	-		Not near a structure
		608.5	25	-		
		606.0	19	21		
		603.5	16	-		
		601.0	13	-		
Q-9	634.0	610.5	34	-		Clay below and high blowcount above
		609.0	27	-		
		606.5	11	19		
		604.0	23	-	Sandy clay	
		601.5	82	-		
SW-2	634.0	617.0	36	-		Outside service water pump structure; does not affect stability of the structure
		612.5	10	-		
		607.5	11	18		
W-4	633.0	619.0	9	-		Outside service water pump structure; does not affect stability of the structure
		613.0	5	-	Sandy clay	
		609.0	12	17		
		606.5	23	-	Sandy clay	
		603.0	24	-	Sandy clay	

See Fig L-11

Table L-2 (continued)

Boring[3] Number	GSE[4] at Time of Drilling (feet)	SPT Information: Sample Elevation (feet)	Blowcounts: In-situ	Blowcounts: Required for M=6, a=0.19, FS=1.5	Soil Description Other Than Sand	Remarks
DG-28	629.0	610.5	15	-		Outside diesel generator building
		608.0	33	-		
		605.5	16	19		
		603.0	15	-	Sandy clay	
		600.5	9	-		
DG-29	630.0	618.5	64	-		Outside diesel generator building
		614.5	93	-		
		610.0	5	17		
		605.5	10	-	Sandy clay	
		601.5	26	-		

(1) This table excludes the areas directly below the diesel generator building and auxiliary building railroad bay. Blowcounts in these zones are shown in Figures L-6 through L-9.
(2) Standard penetration test
(3) Boring location shown in Figures L-3, L-4, and L-5
(4) Ground surface elevation

See Fig. L-12

TABLE L-3[1]

EVALUATION OF LOW SPT[2] BLOWCOUNTS IN THE PLANT AREA FILL

BETWEEN ELEVATIONS 610 AND 627 FEET

Boring[3] Number	GSE[4] At Time of Drilling (feet)	SPT Information Sample Elevation (feet)	Blowcounts In-situ	Blowcounts Required For M=6, a=0.19, FS=1.5	Soil Description Other Than Sand	Remarks
DF-1	633.0	628.0	30	-	Sandy clay	Zone of 3 foot sand fill layer with clay above and below
		623.0	10	11		
		621.5	3	12		
		620.0	12	-	Sandy clay	
		618.5	10	-	Sandy clay	
DF-2	634.0	629.0	47	-		This area has been excavated and later backfilled with sand. The tank foundation is resting on sandy clay with high blowcounts. These low blowcounts in sand occur around but not under tanks and do not affect tank stability.
		624.0	10	-	Sandy clay	
		622.5	3	12		
		621.0	8	13		
		619.5	11	14		
		618.0	16	-		
		616.5	9	16		
		615.0	13	17		
		612.5	6	-	Sandy clay	
		608.0	38	-	Sandy clay	
PD-19	634.0	630.0	9	-		Not near a structure
		627.5	4	-		
		623.5	3	12		
		620.0	21	-		
		617.5	23	-		
PD-20	634.0	631.5	7	-	Silty clay	Not near a structure
		629.0	6	-		
		626.5	7	9		
		624.0	16	-	Sandy clay	
		621.0	8	13		
		618.5	11	-	Clayey silt	
		616.0	3	-	Clayey silt	
		613.5	14	18		
		611.0	45	-		
		608.5	25	-		

See Fig. L-12

TABLE L-3 (continued)

Boring[3] Number	GSE[4] At Time of Drilling (feet)	SPT Information: Sample Elevation (feet)	Blowcounts: In-situ	Blowcounts: Required For M=6, a=0.19, FS=1.5	Soil Description Other Than Sand	Remarks
PD-20A	634.0	630.0	9	-	Silty clay	Not near a structure
		627.5	3	-		
		625.5	5	10		
		622.5	9	12		
		620.0	11	14		
		617.5	3	16		
		614.0	11	-	Clay & sand	
		611.5	24	-		
PD-20C	634.0	631.5	19	-		Not near a structure
		629.0	4	-		
		626.5	7	9		
		622.0	7	13		
		619.5	31	-		
		617.0	37	-		
SWL-1	634.0	616.0	14	-	Sandy clay	Zone of 2.5 foot sand fill layer with clay above and below
		613.5	9	-	Sandy clay	
		611.0	13	19		
		608.5	4	-	Sandy clay	
		606.0	29	-	Sandy clay	
PD-13	634.0	630.0	5	-		Above maximum ground water table
		627.5	1	-		Silty clay below
		625.0	6	11		
		622.5	5	-	Silty clay	
		620.0	10	-	Silty clay	
Q-9	634.0	629.0	5	-	Sandy clay	Within excavation zone
		624.0	9	-	Sandy clay	
		617.5	7	14		
		615.5	13	15		
		614.0	7	16		
		610.5	34	-		
		609.0	27	-		
SWL-8	634.0	630.0	6	-	Silty clay	Within dewatering zone
		627.5	5	-	Silty clay	
		625.0	4	11		
		622.5	16	-		
		620.0	7	14		
SWL-8A	634.0	622.5	2	12		Within dewatering zone
		620.0	9	14		
		617.5	7	16		

See Fig L-12

TABLE L-3 (continued)

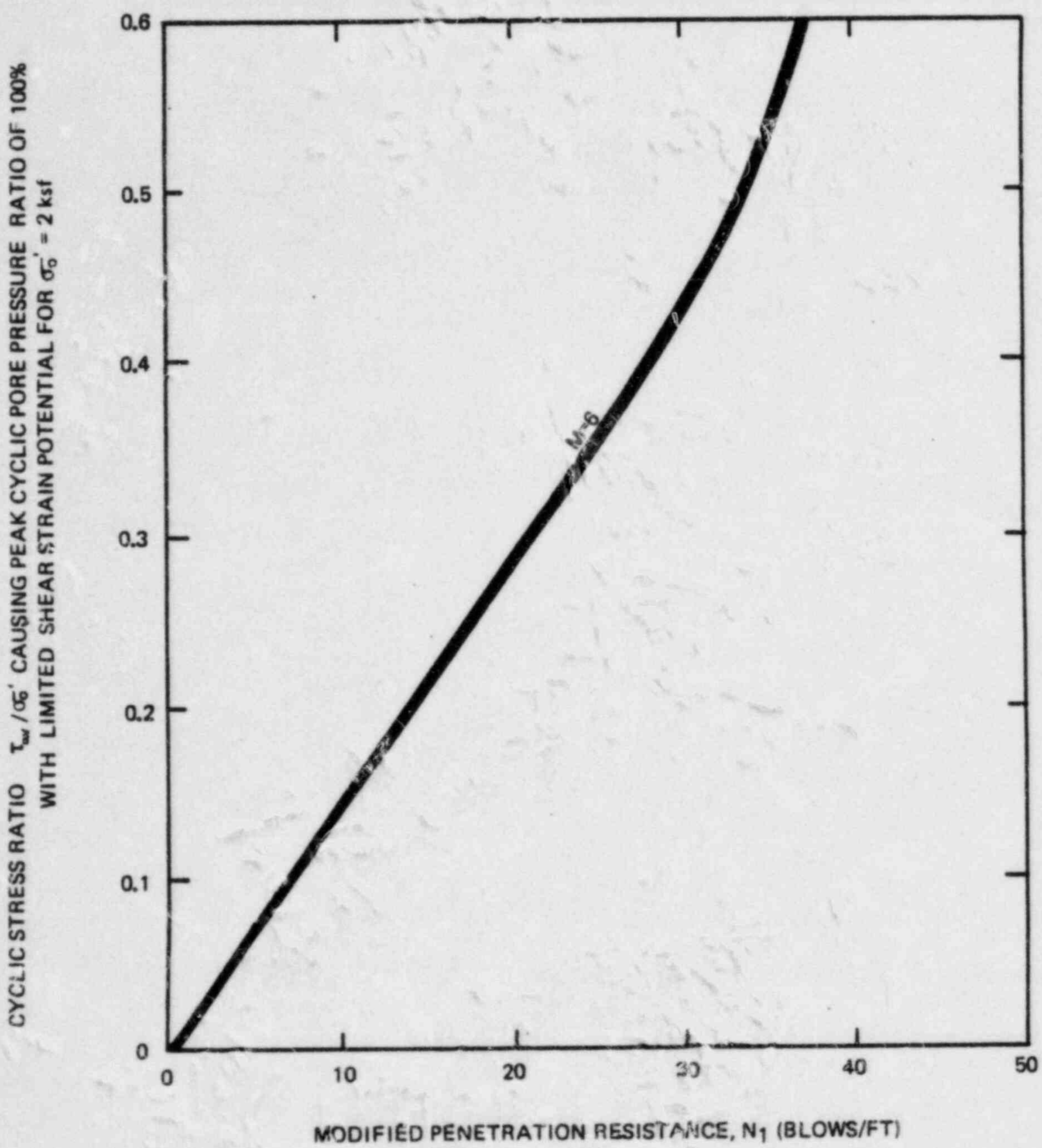
Boring[3] Number	GSE[4] At Time of Drilling (feet)	SPT Information Sample Elevation (feet)	Blowcounts In-situ	Blowcounts Required For M=6, a=0.19, FS=1.5	Soil Description Other Than Sand	Remarks
SWL-6	634.0	617.5	8	-	Silty clay	Zone of 2 foot sand fill layer with clay fill above and below
		615.0	14	-	Silty clay	
		612.5	15	18		
		610.0	33	-	Silty clay	
		607.5	12	-	Silty clay	
SW-7	635.0	626.0	21	-		Within excavation zone
		623.5	24	-		
		621.0	12	14		
		618.5	9	16		
		616.0	19	-		
		613.5	11	-	Silty clay	
CH-2	633.8	622.3	4	12		Within excavation zone
		617.3	4	16		
		612.3	13	-	Silty clay	
		607.3	11	-	Silty clay	
CH-4	634.6	623.1	4	12		Within excavation zone
		618.1	45	-		
		613.1	17	18		
		608.1	24	-		
		603.1	33	-	Sandy clay	
CH-5	633.8	622.3	20	-		Within excavation zone
		617.3	38	-		
		612.3	9	18		
CH-6	634.0	622.5	17	-		Within excavation zone
		617.5	5	16		
		612.5	6	18		
PD-27	634.0	625.0	31	-		Within excavation zone
		622.5	8	-		
		620.0	4	13		
		617.5	16	-		
		615.0	33	-		
SW-2	634.0	621.5	51	-		Outside the service water pump structure and does not affect the stability of the structure
		617.0	36	-		
		612.5	10	16		
		607.5	11	-		

See Fig. L-12

TABLE L-3 (continued)

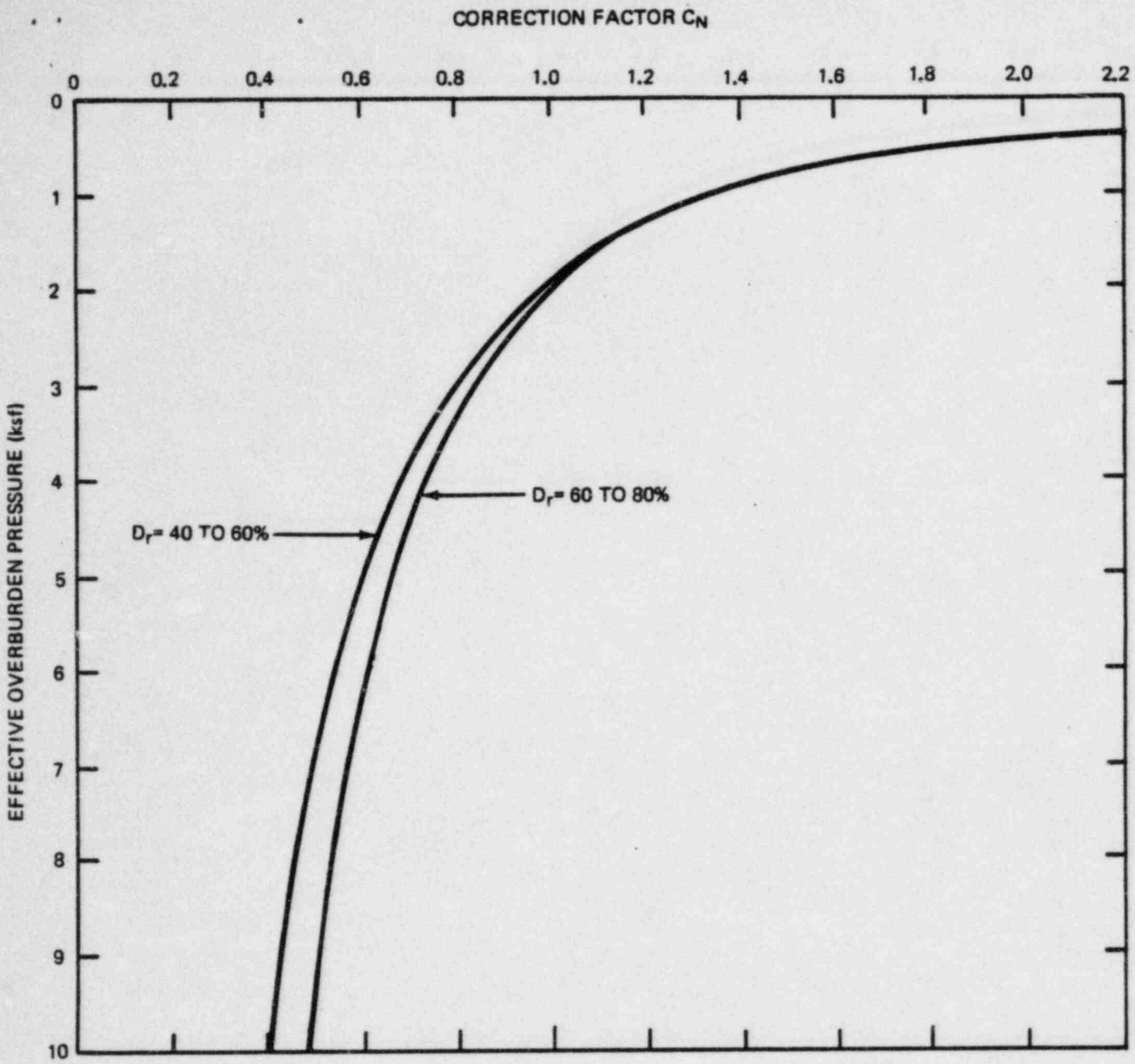
Boring[3] Number	GSE[4] At Time of Drilling (feet)	SPT Information: Sample Elevation (feet)	Blowcounts: In-situ	Blowcounts: Required For M=6, a=0.19, FS=1.5	Soil Description Other Than Sand	Remarks
SW-5	634.5	625.5	28	-		Outside the service water pump structure and does not affect the stability of the structure
		623.0	6	-	Silty clay	
		620.5	3	14		
		618.0	6	16		
		615.5	11	17		
		613.0	16	-	Silty clay	
		610.5	35	-		
DW-1	634.0	617.5	9	-	Sandy gravel	Excavated and backfilled during duct bank repair
		612.5	16	18		
		610.0	30	-	Silty clay	
DW-2	634.0	612.5	13	18		Isolated in clay fill
		609.5	31	-	Silty clay*	

(1) This table excludes the areas directly below the diesel generator building and auxiliary building railroad bay. Blowcounts in these zones are shown in Figures L-6 through L-9.
(2) Standard penetration test
(3) Boring location shown in Figures L-3, L-4, and L-5
(4) Ground surface elevation



SK-G-947

BECHTEL ANN ARBOR		
MIDLAND POWER PLANT		
LIQUEFACTION EVALUATION-CYCLIC STRESS RATIO VS "MODIFIED" PENETRATION RESISTANCE FOR EARTHQUAKE MAGNITUDE OF 6, AFTER SEED (2)		
JOB NO.	DRAWING NO.	REV.
7220	FIGURE L-1	0

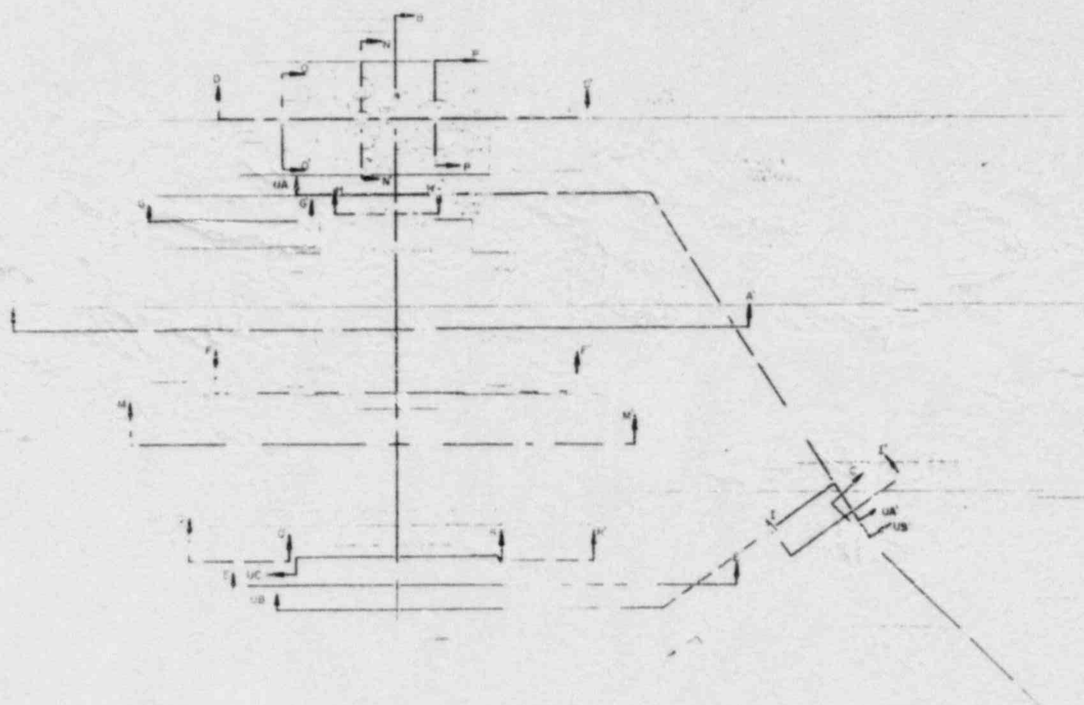


EXPLANATION

D_r– RELATIVE DENSITY

BECHTEL ANN ARBOR		
MIDLAND POWER PLANT		
LIQUEFACTION EVALUATION-CORRECTION FACTOR FOR BLOWCOUNT AS A FUNCTION OF OVERBURDEN PRESSURE, AFTER SEED (2)		
JOB NO.	DRAWING NO.	REV.
7220	FIGURE L-2	0

SK-G 948



ION

1961

EXPLORATION PROG... PERIOD

- WALTER ... BORINGS
- BECHTEL ...
- SOIL & MATE... ...EERS, INC BORINGS ...974
- BECHTEL BORINGS 1973 & 1974
- BECHTEL BORINGS 1977 & 1978
- BECHTEL BORINGS 1978, 1979, 1980 & 1981
- WOODWARD-CLYDE CONSULTANTS BORINGS, 1981 & 1982
- BECHTEL TEST PITS, 1978 & 1979
- BECHTEL PLATE LOAD TEST, 1979
- CROSS-SECTION LOCATION

NOTE:
For the location of additional cross-sections in Emergency Cooling Water Reservoir see SK-G-496.

N

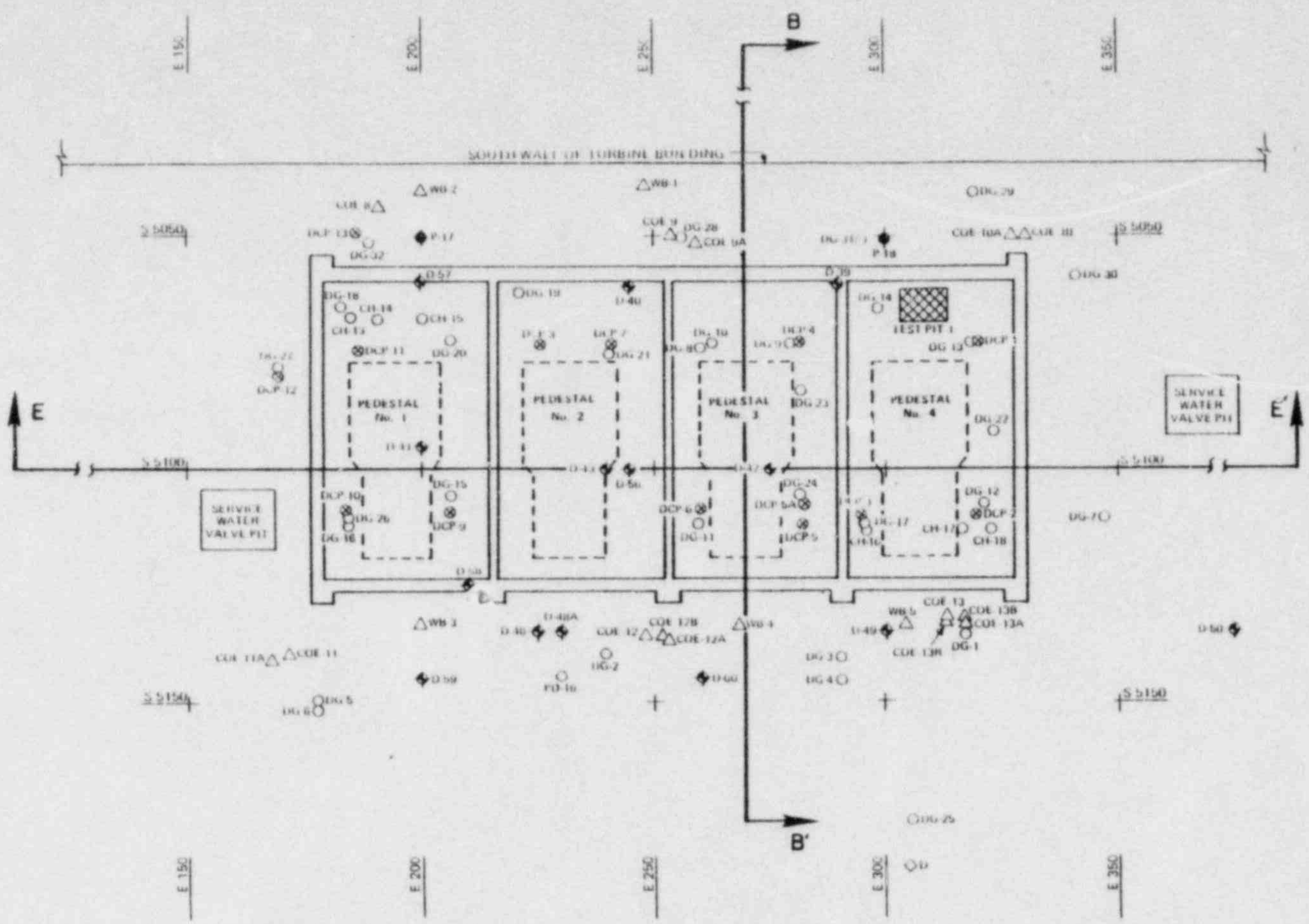
0 25 50 100 150 200
SCALE IN FEET

BECHTEL
ANN ARBOR

...ER PLANT

...AD BORING PLAN

...URE ...



EXPLANATION

- ● — DAMES & MOORE BORINGS, 1969
- BECHTEL BORINGS, 1970
- BECHTEL BORINGS, 1977
- ○ — BECHTEL BORINGS, 1978, 1979 and 1980
- ⊗ — DUTCH CONE PENETROMETER, 1978
- △ — WOODWARD CLYDE CONSULTANTS BORINGS, 1981 and 1982
- BECHTEL TEST PIT, 1978

NOTE
For the location of borings and ... profiles in adjacent areas, see SK G 443.

SCALE IN FEET

NO	DATE	REVISION	BY
		... FOR ... ISSUE	

SCALE ... DESIGNED ... DRAWN ...

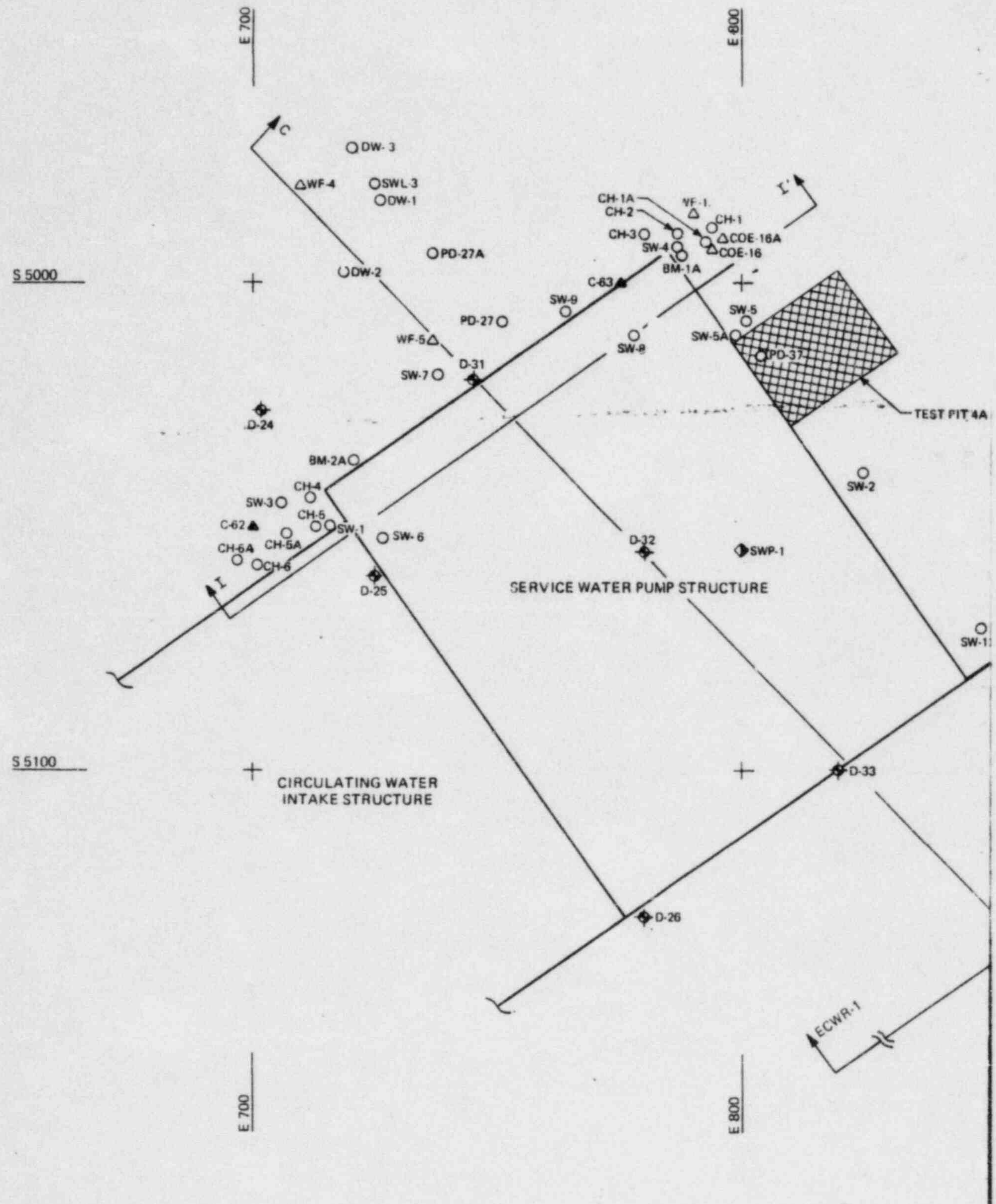
BECHTEL
ANN ARBOR

MIDLAND POWER PLANT

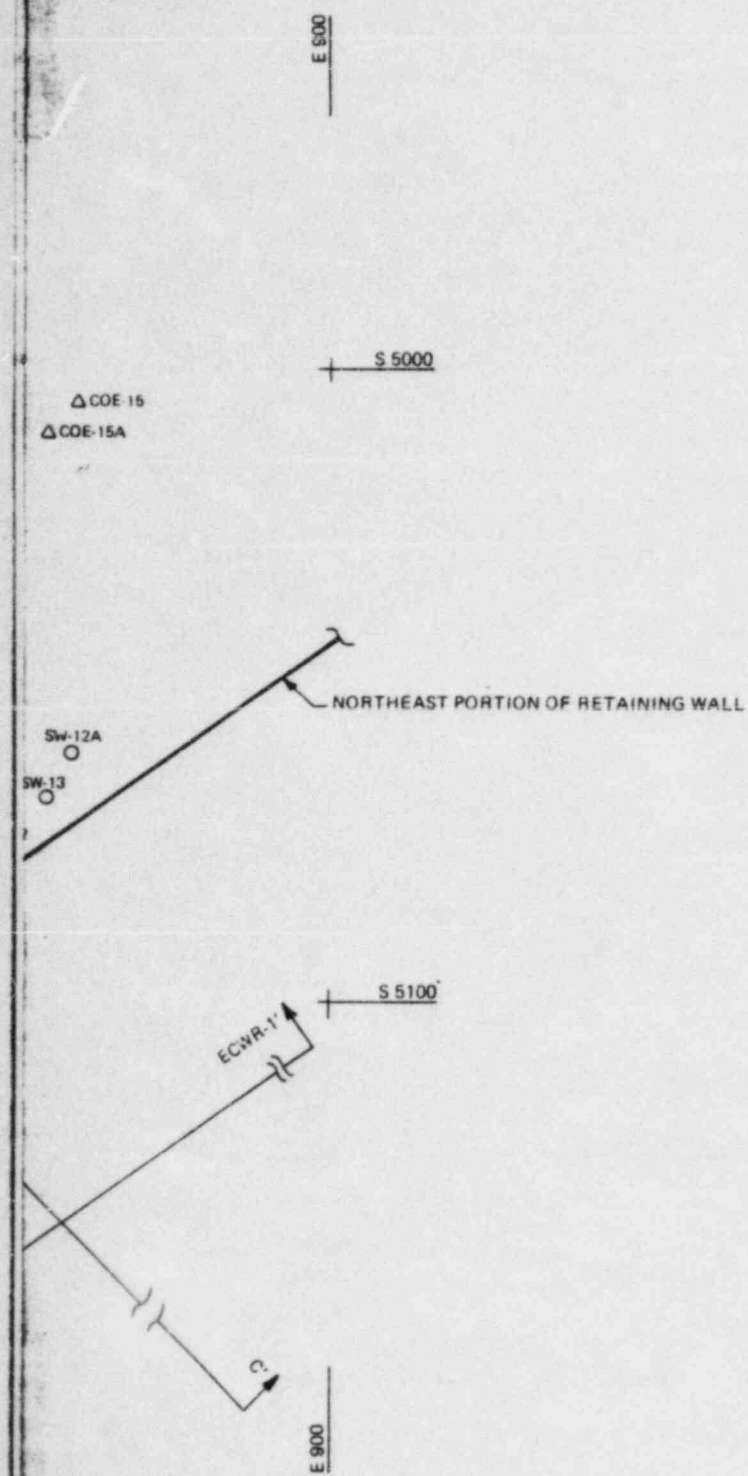
CROSS SECTIONS, BORINGS AND TEST PIT
LOCATION PLAN
DIESEL GENERATOR BUILDING

JOB NO.	DRAWING NO.	REV
7220	FIGURE ... 4	0

K-G-653



E 700
E 800
S 5000
S 5100
C
I'
I
DW-3
WF-4
SWL-3
DW-1
DW-2
PD-27A
PD-27
WF-5
SW-7
D-31
SW-9
C-83
SW-8
CH-1A
CH-2
CH-3
SW-4
WF-1
CH-1
COE-16A
COE-16
BM-1A
SW-5
SW-5A
PD-37
TEST PIT 4A
D-24
BM-2A
CH-4
SW-3
C-62
CH-5
SW-1
CH-5A
CH-6A
CH-6
SW-6
D-25
D-32
SWP-1
SW-2
SERVICE WATER PUMP STRUCTURE
CIRCULATING WATER INTAKE STRUCTURE
D-33
D-26
ECWR-1



EXPLANATION

- ▲ — WALTER FLOOD COMPANY BORINGS 1969 & 1970
- BECHTEL BORINGS, 1970
- BECHTEL BORINGS; 1973 & 1974
- ○ — BECHTEL BORINGS; 1978, 1979, 1980 & 1981
- △ — WOODWARD-CLYDE CONSULTANTS BORINGS; 1981
- BECHTEL TEST PIT; 1979
- LOCATION OF SUBSURFACE PROFILE

NOTE:

For the location of borings and subsurface profiles in adjacent areas, see SK-G-443 and SK-G-496.

N

0 10 20 40 60

SCALE IN FEET

NO.	DATE	REVISIONS	BY	CH'D

SCALE SHOWN | DESIGNED DAH | DRAWN BJR

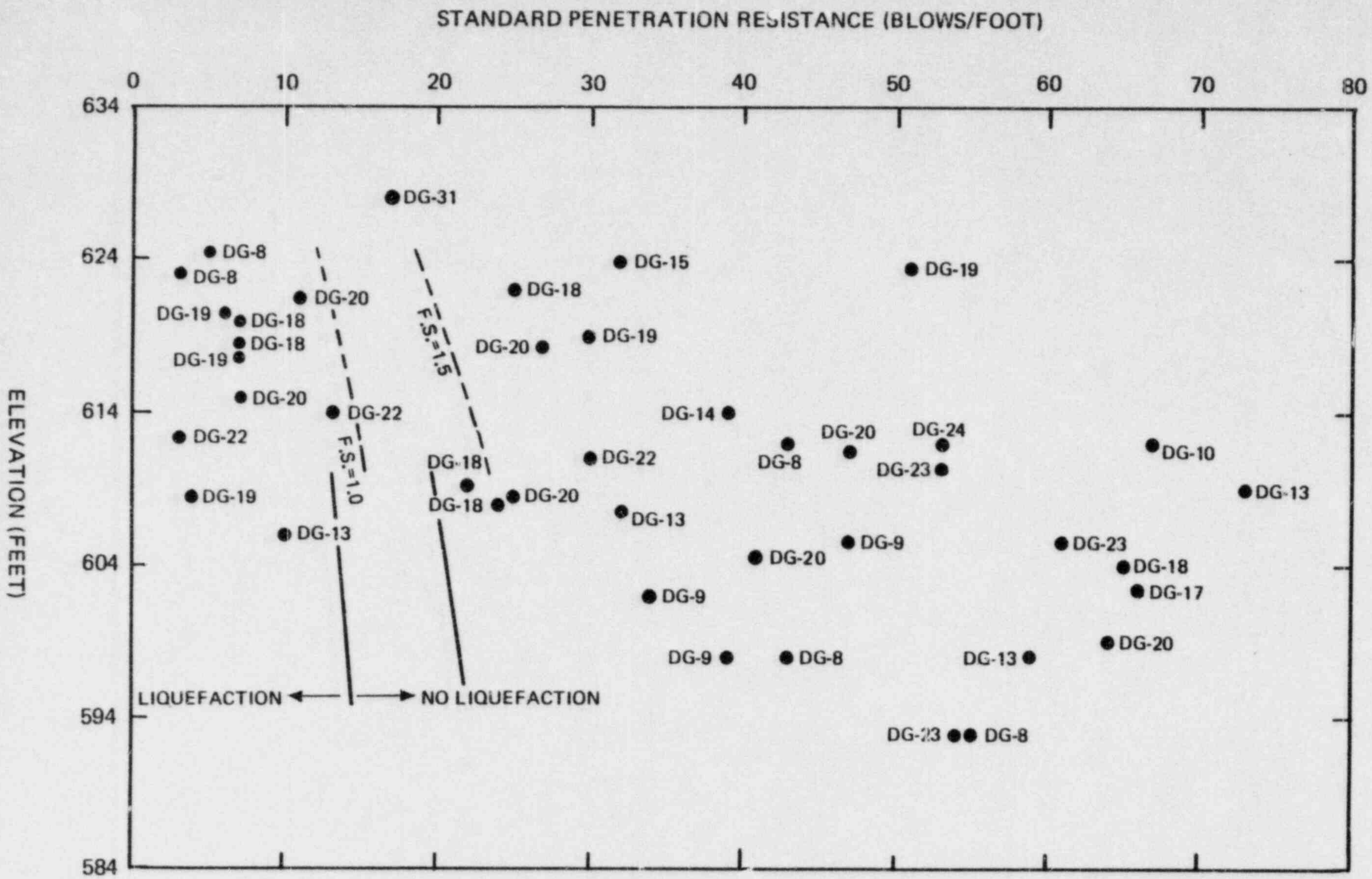
BECHTEL

ANN ARBOR

MIDLAND POWER PLANT

BORING AND TEST PIT LOCATION PLAN
SERVICE WATER PUMP STRUCTURE

JOB NO.	DRAWING NO.	REV.
7220	FIGURE I. 5	0



EXPLANATION

— — — — BOUNDARY OF LIQUEFACTION, GWT AT 627.0'

———— BOUNDARY OF LIQUEFACTION, GWT AT 610.0'

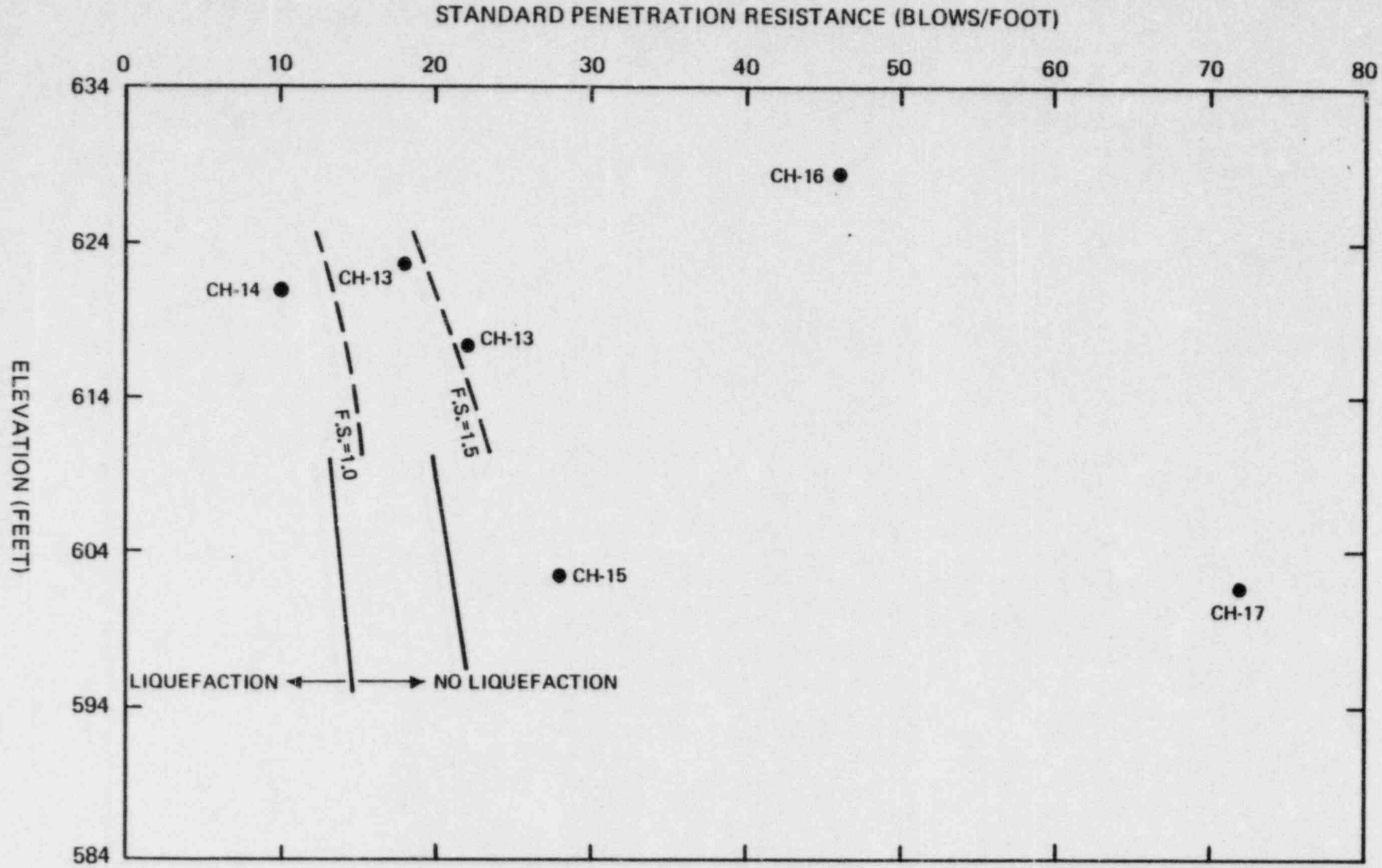
GWT – GROUND WATER TABLE

NOTES:

1. BLOWCOUNTS WERE CORRECTED TO ACCOUNT FOR ADDED SURCHARGE DUE TO THE BUILDING LOAD AND LOWERED WATER TABLE.
2. BORINGS PRESENTED ARE LOCATED WITHIN THE DIESEL GENERATOR BUILDING.

SK-G-702

BECHTEL ANN ARBOR		
MIDLAND POWER PLANT		
LIQUEFACTION EVALUATION BASED ON 1978 "DG" BORINGS – BOUNDARIES OF LIQUEFACTION AND NO LIQUEFACTION FOR DIESEL GENERATOR BUILDING		
JOB NO.	DRAWING NO.	REV.
7220	FIGURE [illegible]	2



EXPLANATION

— — — — BOUNDARY OF LIQUEFACTION, GWT AT 627.0'

———— BOUNDARY OF LIQUEFACTION, GWT AT 610.0'

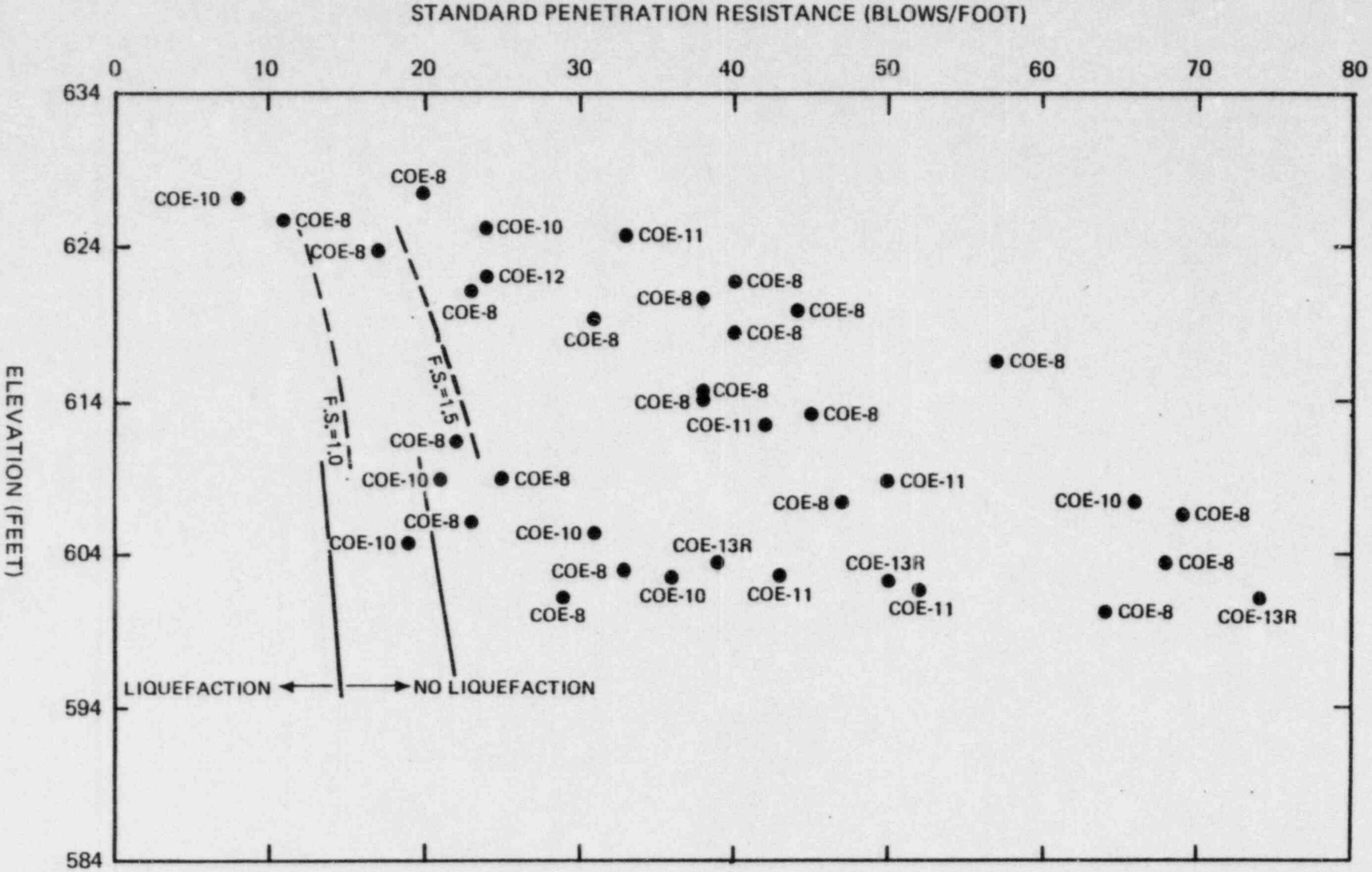
GWT – GROUND WATER TABLE

BECHTEL
ANN ARBOR

MIDLAND POWER PLANT

LIQUEFACTION EVALUATION BASED ON 1979 "CH" BORINGS – BOUNDARIES OF LIQUEFACTION AND NO LIQUEFACTION FOR DIESEL GENERATOR BUILDING

JOB NO.	DRAWING NO.	REV.
7220	FIGURE	0



EXPLANATION

— — — — — BOUNDARY OF LIQUEFACTION, GWT AT 627.0'

———————— BOUNDARY OF LIQUEFACTION, GWT AT 610.0'

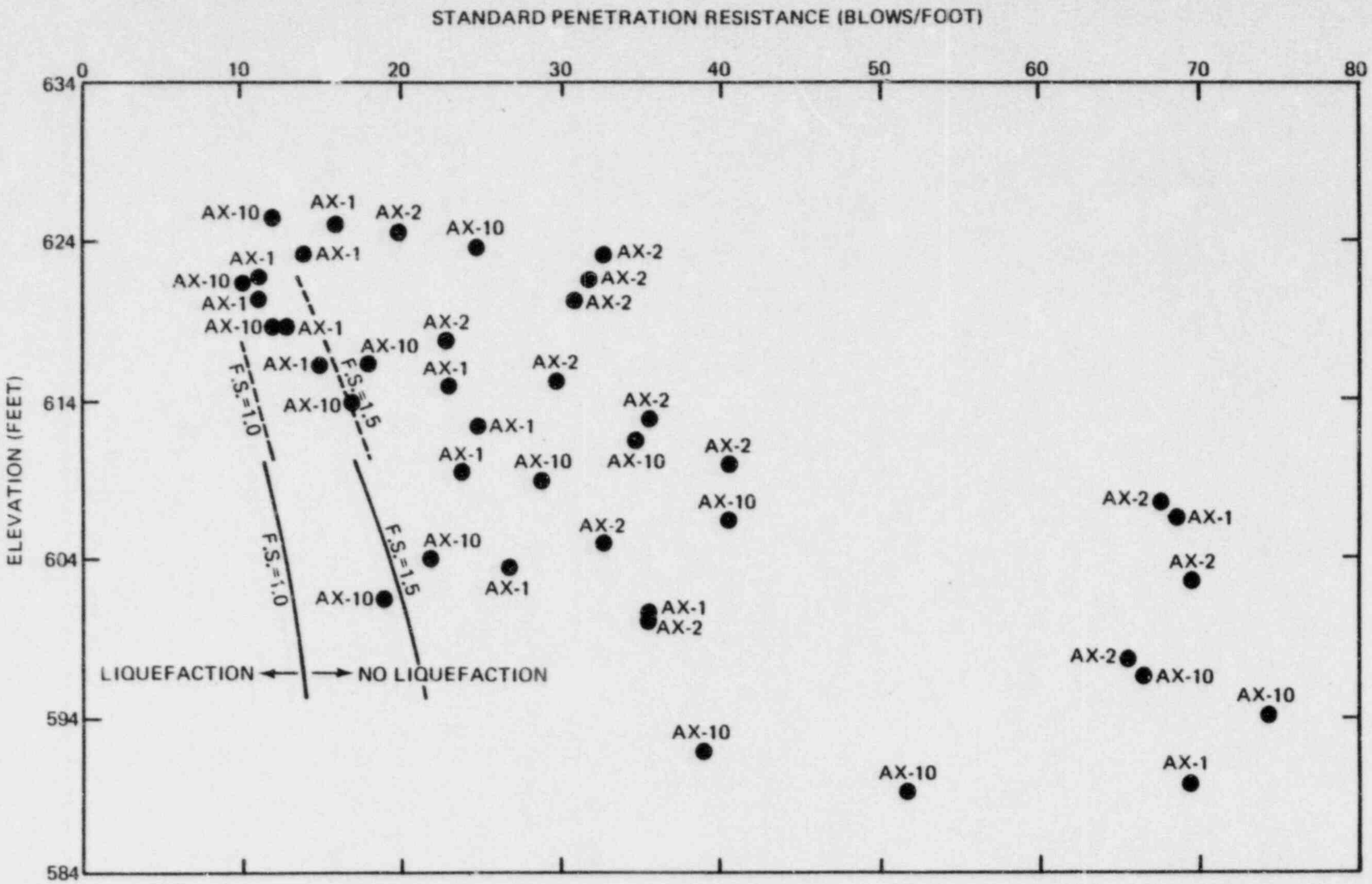
GWT — GROUND WATER TABLE

NOTE :

BLOWCOUNTS WERE CONVERTED FROM THE RELATIVE DENSITY VALUES OBTAINED FROM WOODWARD–CLYDE CONSULTANTS TEST DATA

BECHTEL ANN ARBOR		
MIDLAND POWER PLANT		
LIQUEFACTION EVALUATION BASED ON 1981 "COE" SERIES BORINGS – BOUNDARIES OF LIQUEFACTION AND NO LIQUEFACTION FOR DIESEL GENERATOR BUILDING		
JOB NO.	DRAWING NO.	REV.
7220	FIGURE L-8	1

SK-C-704



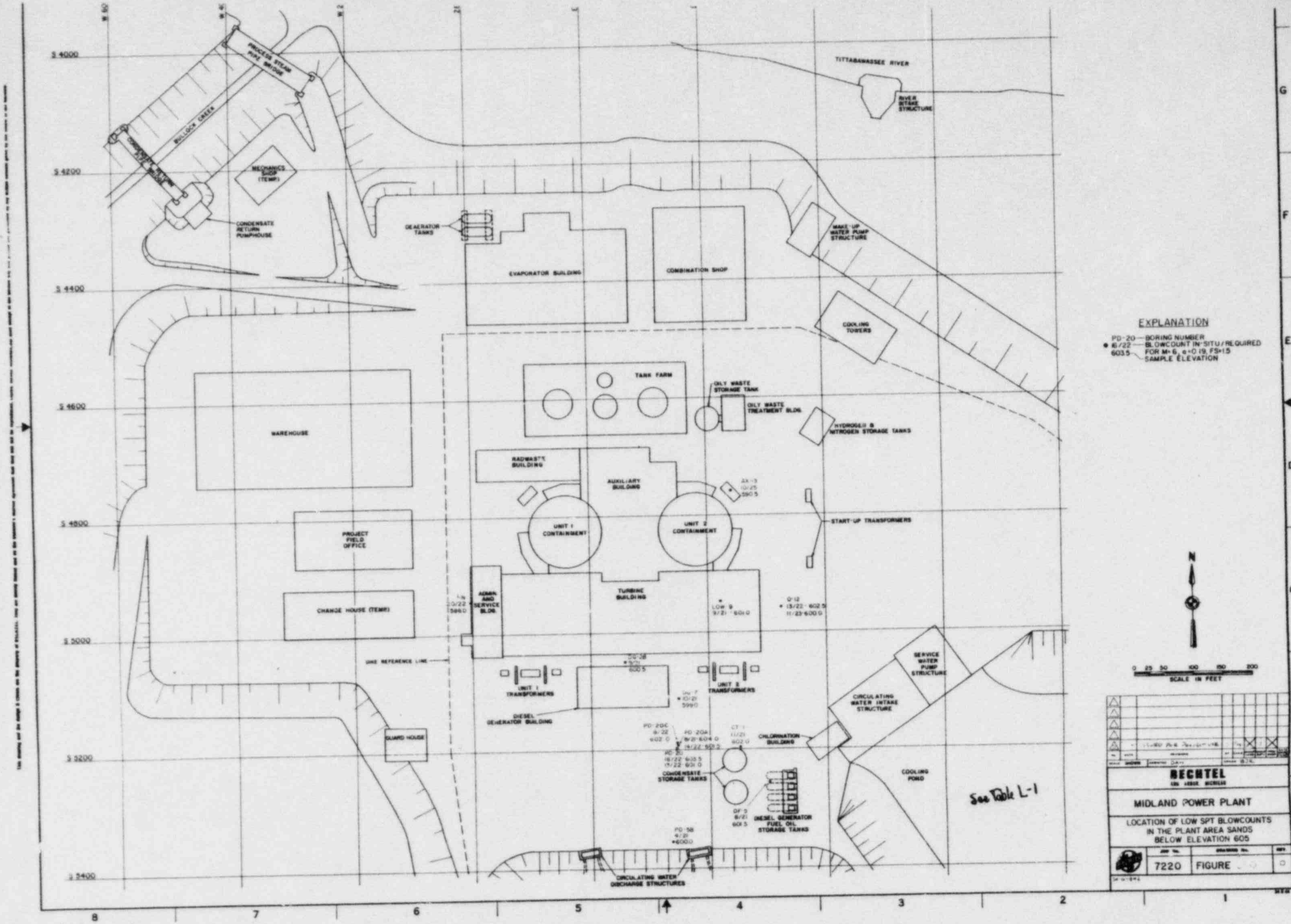
EXPLANATION

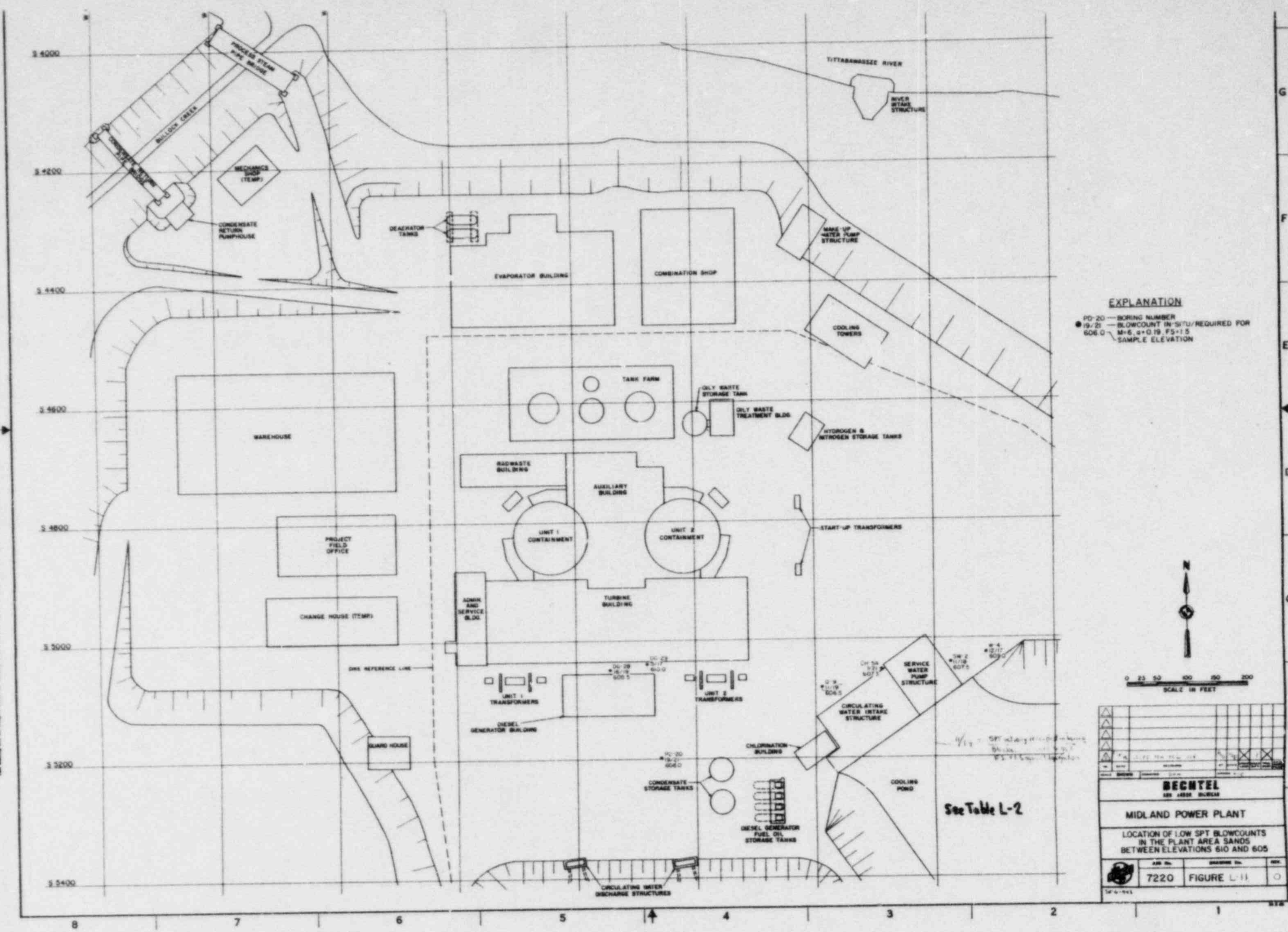
- - - - - BOUNDARY OF LIQUEFACTION, GWT AT 622.0'

——— BOUNDARY OF LIQUEFACTION, GWT AT 610.0'

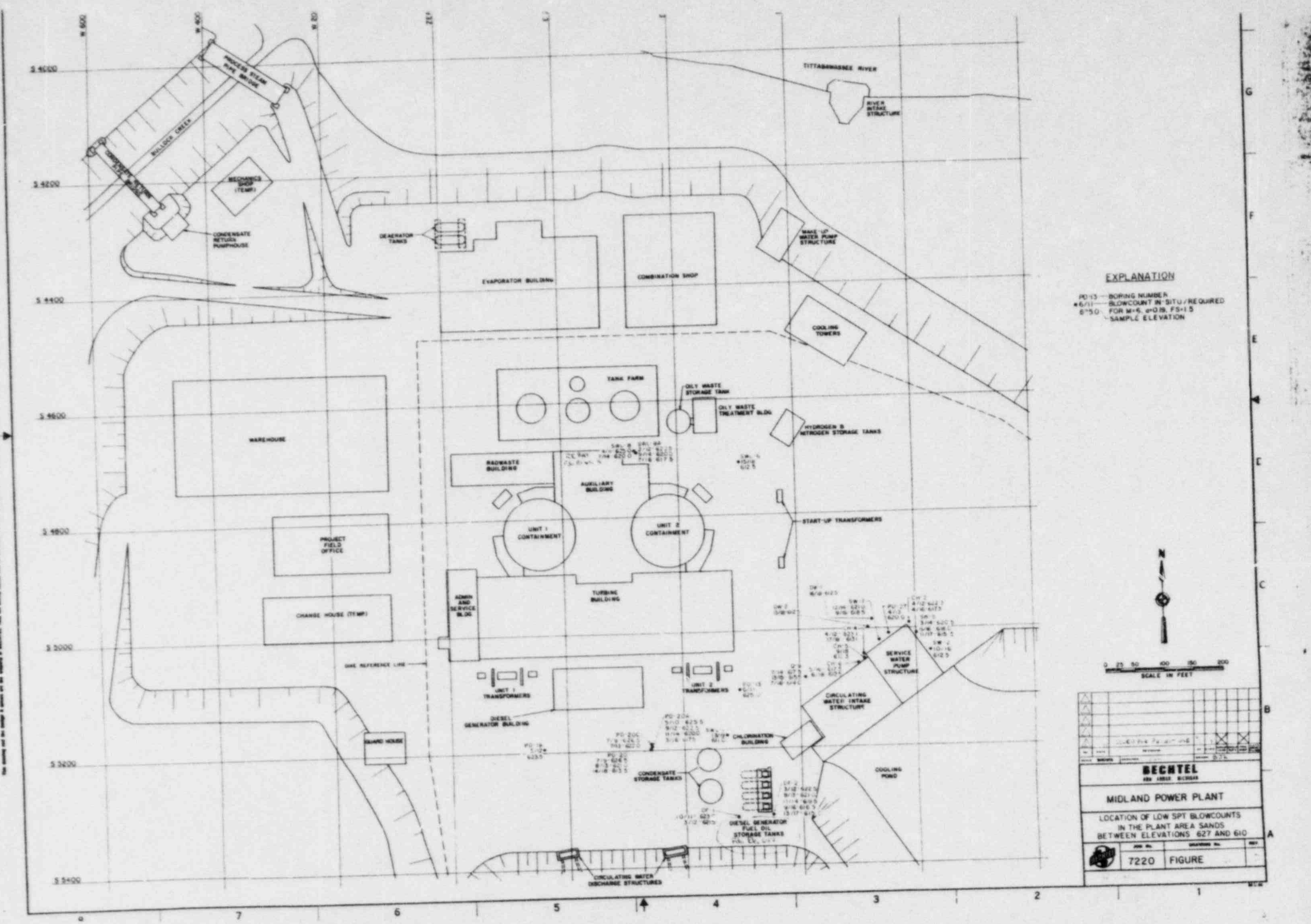
GWT – GROUND WATER TABLE

BECHTEL ANN ARBOR		
MIDLAND POWER PLANT		
LIQUEFACTION EVALUATION BASED ON 1979 BORINGS – BOUNDARIES OF LIQUEFACTION AND NO LIQUEFACTION FOR THE RAILROAD BAY AREA OF THE AUXILIARY BUILDING		
JOB NO.	DRAWING NO.	REV.
7220	FIGURE L-9	0


TITTABAWASSEE RIVER
RIVER INTAKE STRUCTURE
PROCESS STEAM PIPE BRIDGE
BULLOCK CREEK
MECHANICS SHOP (TEMP)
CONDENSATE RETURN PUMPHOUSE
DEAERATOR TANKS
EVAPORATOR BUILDING
COMBINATION SHOP
MAKE-UP WATER PUMP STRUCTURE
COOLING TOWERS
EXPLANATION
PD-20 — BORING NUMBER
6/22 — BLOWCOUNT IN-SITU/REQUIRED FOR M=6, a=0.19, FS=1.5
603.5 — SAMPLE ELEVATION
WAREHOUSE
TANK FARM
OILY WASTE STORAGE TANK
OILY WASTE TREATMENT BLDG.
HYDROGEN & NITROGEN STORAGE TANKS
RADWASTE BUILDING
AUXILIARY BUILDING
UNIT 1 CONTAINMENT
UNIT 2 CONTAINMENT
START-UP TRANSFORMERS
PROJECT FIELD OFFICE
CHANGE HOUSE (TEMP)
ADMIN. AND SERVICE BLDG.
TURBINE BUILDING
UNIT 1 TRANSFORMERS
UNIT 2 TRANSFORMERS
DIESEL GENERATOR BUILDING
GUARD HOUSE
CONDENSATE STORAGE TANKS
CHLORINATION BUILDING
DIESEL GENERATOR FUEL OIL STORAGE TANKS
CIRCULATING WATER DISCHARGE STRUCTURES
CIRCULATING WATER INTAKE STRUCTURE
SERVICE WATER PUMP STRUCTURE
COOLING POND
UNIT REFERENCE LINE
S 4000
S 4200
S 4400
S 4600
S 4800
S 5000
S 5200
S 5400
N
0 25 50 100 150 200
SCALE IN FEET
See Table L-1
BECHTEL
MIDLAND POWER PLANT
LOCATION OF LOW SPT BLOWCOUNTS
IN THE PLANT AREA SANDS
BELOW ELEVATION 605
7220
FIGURE
A
B
C
D
E
F
G
1
2
3
4
5
6
7
8



TITTABAWASSEE RIVER
RIVER INTAKE STRUCTURE
PROCESS STEAM PIPE BRIDGE
BULLOCK CREEK
MECHANICS SHOP (TEMP)
CONDENSATE RETURN PUMPHOUSE
DEAERATOR TANKS
EVAPORATOR BUILDING
COMBINATION SHOP
MAKE-UP WATER PUMP STRUCTURE
COOLING TOWERS
EXPLANATION
PD-20 — BORING NUMBER
19/21 — BLOWCOUNT IN-SITU/REQUIRED FOR M=6, a=0.19, FS=1.5
606.0 — SAMPLE ELEVATION
WAREHOUSE
TANK FARM
OILY WASTE STORAGE TANK
OILY WASTE TREATMENT BLDG.
HYDROGEN & NITROGEN STORAGE TANKS
RADWASTE BUILDING
AUXILIARY BUILDING
UNIT 1 CONTAINMENT
UNIT 2 CONTAINMENT
START-UP TRANSFORMERS
PROJECT FIELD OFFICE
CHANGE HOUSE (TEMP)
ADMIN AND SERVICE BLDG.
TURBINE BUILDING
DHS REFERENCE LINE
UNIT 1 TRANSFORMERS
UNIT 2 TRANSFORMERS
DIESEL GENERATOR BUILDING
SERVICE WATER PUMP STRUCTURE
CIRCULATING WATER INTAKE STRUCTURE
CHLORINATION BUILDING
GUARD HOUSE
PD-20
19/21
606.0
CONDENSATE STORAGE TANKS
DIESEL GENERATOR FUEL OIL STORAGE TANKS
COOLING POND
See Table L-2
CIRCULATING WATER DISCHARGE STRUCTURES
S 4000
S 4200
S 4400
S 4600
S 4800
S 5000
S 5200
S 5400
SCALE IN FEET
0 25 50 100 150 200
BECHTEL
MIDLAND POWER PLANT
LOCATION OF LOW SPT BLOWCOUNTS IN THE PLANT AREA SANDS BETWEEN ELEVATIONS 610 AND 605
7220
FIGURE L-11
0


EXPLANATION
PD-13 — BORING NUMBER
6/11 — BLOWCOUNT IN-SITU/REQUIRED FOR M=6, a=0.19, FS=1.5
6150 — SAMPLE ELEVATION
TITTABAWASSEE RIVER
RIVER INTAKE STRUCTURE
PROCESS STEAM PIPE BRIDGE
BULLOCK CREEK
MECHANICS SHOP (TEMP)
CONDENSATE RETURN PUMPHOUSE
DEAERATOR TANKS
EVAPORATOR BUILDING
COMBINATION SHOP
MAKE-UP WATER PUMP STRUCTURE
COOLING TOWERS
TANK FARM
OILY WASTE STORAGE TANK
OILY WASTE TREATMENT BLDG
HYDROGEN & NITROGEN STORAGE TANKS
WAREHOUSE
RADWASTE BUILDING
AUXILIARY BUILDING
UNIT 1 CONTAINMENT
UNIT 2 CONTAINMENT
START-UP TRANSFORMERS
PROJECT FIELD OFFICE
CHANGE HOUSE (TEMP)
ADMIN AND SERVICE BLDG
TURBINE BUILDING
SERVICE WATER PUMP STRUCTURE
CIRCULATING WATER INTAKE STRUCTURE
UNIT 1 TRANSFORMERS
UNIT 2 TRANSFORMERS
DIESEL GENERATOR BUILDING
CHLORINATION BUILDING
GUARD HOUSE
CONDENSATE STORAGE TANKS
DIESEL GENERATOR FUEL OIL STORAGE TANKS
COOLING POND
CIRCULATING WATER DISCHARGE STRUCTURES
SCALE IN FEET
0 25 50 100 150 200
N
BECHTEL
MIDLAND POWER PLANT
LOCATION OF LOW SPT BLOWCOUNTS IN THE PLANT AREA SANDS BETWEEN ELEVATIONS 627 AND 610
7220
FIGURE

6/11/82 J. Kane
8/133

A material false statement was made in section 2.5.4.5.3 of the FSAR which stated that "all fill and backfill were placed according to Table 2.5-9". Table 2.5-9, minimum compaction criteria contained the following:

"Function	Zone (1) Designation	Soil Type	Compaction Criteria Degree	ASTM Designation
Support of structures		Clay	95%	ASTM D 1557-66T (modified) (2)

(1) For zone designation see Table 2.5-10.
(2) The method was modified to get 20,000 foot-pounds of compactive energy per cubic foot of soil."

This statement is material in that sections 2.5.4.5.3 and the indicated portion of table 2.5-9 would have been found unacceptable without further staff analysis and questions if the staff had known that category I structures had been placed in fact on fill which did not meet the minimum compaction criteria set out in FSAR Table 2.5-9.

I propose to substitute the above for

① The 2 sentences on p 2 of the 12-6-79 order that begin with "In addition, as described → and

② App. B.

Please review carefully + give me a call.

Bill P.

Purpose of making change now?

Rec'd 6/10/82
from W. Paton

which did not meet the
minimum compaction criteria (shown on ~~set out~~
~~in~~ Table 2.5-9) as stated in FSAR paragraph 2.5.4.5.3.
(pg 2.5-51 orig. submitt
pg 2.5-52 later revision

Joe - I want to add ~~the~~ these words to ~~pg~~
pg 2 where the X is.
pls give me a call

relation to Keppler Report april 3, 79?

and compaction requirements were not followed; (2) there was a lack of clear direction and support between the contractor's engineering office and construction site as well as within the contractor's engineering office; (3) there was a lack of control and supervision of plant fill placement activities which contributed to inadequate compaction of foundation material; (4) corrective action regarding noncomformances related to plant fill was insufficient or inadequate as evidence by repeated deviations from specification requirements; and (5) the FSAR contains inconsistent, incorrect, and unsupported statements with respect to foundation type, soil properties and settlement values. The details of these findings are described in the inspection reports 50-329/78-12, 50-330/78-12 (November 14, 1978) and 50-329/78-20, 50-330/78-20 (March 19, 1979) which were sent to the Licensee on November 17, 1978 and March 22, 1979 respectively.

QA → admits + denies

The items of noncompliance resulting from the NRC investigation are described in Appendix A to this Order. In addition, as described in Appendix B to this Order, a material false statement was made in the FSAR in that the FSAR falsely stated that "All fill and backfill were placed according to Table 2.5-9." This statement is material in that this portion of the FSAR would have been found unacceptable without further Staff analysis and questions if the Staff had known that Category I structures had been placed in fact on random fill rather than controlled compacted cohesive fill as stated in the FSAR.

MF Statement

See C.P. answer to App A + B or in Appendix to Answer

Original Table 2.5-9 in FSAR indicated fill to be either clay or sand

or clay fill - PSAR refers to it both as clay fill (pg. 2-28) and ~~cohesive fill~~ (pg. 3 of Amend 1)

As a result of questions raised during the NRC investigation of the Diesel Generator Building settlement, additional information was necessary to evaluate

Ask W. Paton where in FSAR the term "compacted cohesive fill" term is used

See Table 2.5-14 of original FSAR submittal

This information is false, in that materials other than controlled compacted cohesive fill were used to support the diesel generator building and information presented concerning the supporting soils influenced the staff review of the FSAR.

last pg of
12-6-79
order
pls discuss

Discussed w/ Wm. Paton on 6/12/81

I need to review; CELD's comments & Harris testimony then get together to establish the following:

How this testimony fits in with the new established order of the hearing? What portions are need by July 20, 1981 for August sessions & what portions are needed by Sept 15th for the October sessions

How does Joe Kane fit into this - will

~~allows more structural than geotechnical~~ ~~write #1~~ say good... on history - 9/1/83

have all reviews to a quick review

NRC STAFF TESTIMONY OF H.N. SINGH, P.E. ON UNRESOLVED SAFETY ISSUES (GEOTECHNICAL ENGINEERING)

Q1. Please state your name and position with the Corps of Engineers.

A. My name is Hari Narain Singh. I am a Civil Engineer with the U.S. Army Corps of Engineers, Detroit District.

Q2. Have you prepared a statement of your professional qualifications?

A. Yes. A copy of this statement is attached.

Q3. Please state the nature of the responsibilities that you have had with the Corps of Engineers before assuming your assignment of reviewing the geotechnical aspects of the Midland Nuclear Power Plant.

A. I worked in the Design Section of the Technical Branch, and was responsible for designing and reviewing designs of structures involving soil structure interaction such as sheet piles, earth anchors, friction and bearing piles, machine foundations, foundations for buildings. I was also responsible for design and review of designs of dikes for dredged material disposal facilities.

Q4. Please state the purpose of this testimony. too general

to be done on or about 8-15-81

A. The purpose of this testimony is to apprise the Atomic Safety and Licensing Board (ASLB) of the safety related problems pertaining to geotechnical engineering, at the Midland Nuclear Power Plant Site.

needs a lot be much more specific

Kimball

Q5. When did the Corps of Engineers get involved and what were the areas of its review and the limits of their responsibilities?

A. According to Interagency Agreement No. NRC-03-79-167, which began on 25 September 1979, the U.S. Army Corps of Engineers is obligated to provide technical assistance to the U.S. Nuclear Regulatory Commission (NRC) as to Geotechnical Engineering concerns in reviewing and evaluating the Preliminary Safety Evaluation Report (PSAR) and the Final Safety Evaluation Report (FSAR) submitted by the applicant for a Construction Permit (CP) or Operating License (OL).

does Harry prove?

The reviews are to be conducted using the guidance contained in the NRC Regulatory Guides, industry standards, and the guidance and the acceptance criteria in the Standard Review Plan (SRP) in the areas of geotechnical responsibility. The approach outlined below was to be followed:

(1) Recommend requests for additional information or clarification based upon initial review and evaluation of the information provided by the applicant.

Joe - what is our response to expert comment that purpose of DG 8 is to keep rain off DG - Somebody should say precisely what the concern is.

the answer ignores the OL-OM hearing answer should say "with respect to the OL-OM hearing our responsibilities are →

(ii) Evaluation of the responses provided by the applicant.

(iii) Attendance at meetings with the staff and the applicant to discuss and resolve outstanding issues, and audit the implementation of the applicant commitments.

(iv) Preparation of a Safety Evaluation Report (SER) input which describes the evaluation of the design of the applicant's safety related (and some non-safety related) systems.

(v) Attend meeting with the Advisory Committee on Reactor Safeguards (ACRS) and public hearings to assist the staff in explaining bases for conclusions and positions reached in the SER.

(vi) Preparation of input to SER supplements which further clarify and document systems evaluations in the SER based upon review by the ACRS.

Discuss w/ other 2 questions

Q6. What is Geotehnical Engineering? Why is it necessary to review the geotechnical aspects of the Midland Nuclear Power Plant?

A. Geotechnical Engineering is a branch of Civil Engineering which deals with the foundation of structures and the soil supporting them. It includes soil exploration study of soil properties under various environmental and loading conditions, soil-structure interaction and then by utilizing these information, determination of adequate foundations for structures.

A foundation is the part of a structure which serves to transmit to the soil beneath it, its own weight, the weight of the superstructure above it and any force which might act upon it. A foundation is therefore, the connecting link between a superstructure and the soil. A foundation should be designed to support the loads and moments acting on it and distribute the loads in a satisfactory manner over the contact surface of the soil layer over which it rests. In order to be satisfactory, this distribution must not produce excessive stresses within the soil mass at any depth beneath the foundation. The term excessive stress implies a force per unit area which would cause a complete rupture within the supporting soil mass and result in noticeable tilting and/or sinking of the structure as a whole. Stresses are also to be rated as excessive, if they cause a settlement of the supporting soil surface so uneven that the structure above it would crack or be otherwise damaged while undergoing deformations resulting from this uneven settlement. Thus, the importance of a foundation is self evident, since no structure can endure without an adequate foundation.

Does Joe Kane agree with all this?

A foundation will naturally tend to follow any settlement of the soil on which it rests. In turn, the superstructure will follow the settlement of the foundation which supports it. Both will tend to equalize uneven settlements by resisting deformation and thereby transmitting more load to those parts of the soil surface which have settled least. No deformation of the soil surface beneath a structure can take place without a corresponding deformation of both the foundation and the superstructure above it. Undue deformation in a structure due to uneven settlement of the soil can occur if soil of variable

Joe agrees to clarify

density and physical properties is supporting the structure. The undue deformation might cause serious cracking which will reduce the load carrying capacity of the structure.

To ensure safety against sinking, tilting, cracking of the safety related structures at the Midland Nuclear Power Plant, particularly due to the inadequate compaction of fill material, it is imperative to review the geotechnical aspects of all the Category-I structures deriving support from the plant fill.

Q7. State specifically, the names of the safety related structures which the Corps of Engineers were requested by the NRC to review. Also state specifically the geotechnical aspects reviewed to insure the safety of these structures, and the sources which furnished the Corps the review materials.

A. According to the interagency agreement between the Corps of Engineers and the NRC, the Corps of Engineers is obligated to review the geotechnical aspects of all safety related, Category-I structures under both static and dynamic conditions to the safe shut down and operating basis earthquakes. These structures include:

Joe check list

(i) Reactor Buildings
(ii) Auxiliary Building
(iii) Diesel Generator Building
(iv) Service Water Structure
(v) Diesel Fuel Storage Tanks
(vi) Borated Water Storage Tanks
(vii) Category-I Underground Piping System
(viii) Emergency Cooling Pond (enclosing dikes)

The geotechnical aspects reviewed included:

(a) A review of the site investigation program, both field and laboratory, to assure that an adequate determination of all surface conditions has been achieved including consideration of borrow sources. This may require recommendation for additional investigations to obtain the required data.

(b) Evaluations and recommendations pertaining to proposed design criteria.

(c) A review of the bearing capacity and settlement analyses performed by the applicant and, in many cases, the performance of independent bearing capacity analyses. A review of the slope stability of the Category-I dikes. A determination that the applicant has presented adequate bases to support design parameters used in its analyses.

(d) An evaluation of the stabilization technique proposed by the applicant to solve site foundation problems. Recommendations for stabilization.

make sure its clear that all this is related to OL-OM

- Should we say how many trips to site by how many employees of COE

(e) In regard to most cases, field trips were necessary to inspect the site, to observe sampling and testing of soil, and to evaluate the adequacy of the techniques and equipment.

The information to be reviewed was included in the Final Safety Analysis Report (FSAR) and the pertinent amendments to it, and in the responses to 10CFR 50.54(f) requests regarding the plant fill, which all were forwarded by the applicant to the Corps of Engineers. The review included an evaluation of information included in Sections 2.5, 3.7 and 3.8 of the FSAR and 10CFR 50.54(f) documents which addresses the adequacy of soil mechanics, earthquake engineering and the foundation engineering in order to assure the safe siting and operation of all the seismic safety related Category-I structures and conduits. The review was conducted in accordance with the NRC Standard Review Plans Section 2.5.1, 2.5.2 and 2.5.4. Specific guidance in review was obtained from the NRC Regulatory Guides 1.132, 1.138 and 1.70.

Assure ~~Does~~ COE knows

Q8. What were the results of your review of the materials pertinent to geotechnical engineering provided in the FSAR and in the applicant's responses to 10CFR 50.54(f) requests?

attach

A. The geotechnical information pertaining to each of the Category I structure and conduit provided by the applicant in the FSAR and responses to 10CFR 50.54(f) requests were reviewed by the Detroit District Corps of Engineers. The details of the review comments are provided in the Corps of Engineers' Letter Report of 7 July 1980, and in the Corps of Engineers' review comments of 17 April 1981 on the applicant's Amendment 85 to the operating license requests and on Revision 10 to the 10CFR 50.54(f) requests. A brief description of the descrepancies noted for each structure is given below.

(a) Reactor Building Foundation.

NOW?

The soils and foundation information pertaining to the Reactor Building provided in the FASR are based on the original design which assumes no site dewatering. Site dewatering is now proposed. The Corps' report of 7 July 1980 pointed out this descrepancy and requested the applicant (Question 39, 10CFR 50.54(f)) to discuss and provide analyses for settlements and bearing capacity for the foundation soils considering the effect of permanent dewatering proposed by the applicant to preclude liquefaction under the plant area. The applicant's response to question 39, 10CFR 50.54(f) is not acceptable. The Corps of Engineers' comments of 17 April 1981 on Amendment 85 (attached) provide the details.

Say generally why?

(b) Diesel Generator Building.

update to latest status

The Diesel Generator Building was reported to have settled. The magnitude of the settlements varied from one end to another end along the length and the width of the building with maximum settlement at the southeast corner and the minimum at the northwest corner. The settlements measured in the time interval between 28 March 1978 and 19 January 1979 indicated a maximum settlement of 4.25 inches at the southeast corner and a minimum settlement of 2.09 inches (Fig 27-10 of 10CFR 50.54(f) responses). The settlements would

- middle of next page shows existing warping

only would? is there no observed warping cause a warping of the structure's foundation. The settlements which occurred prior to 28 March 1978 were not reported in the responses to 10CFR 50.54(f) requests.

what happened - did they stop?

where are the details of this set out?

In an effort to determine the cause of the excessive differential settlements, the applicant began a soil exploration program which indicated soil fill of very substandard compaction. As indicated by the blowcounts of the standard penetration test, the quality of the fill material varied from loose sand to dense sand and from soft clay to stiff clay, indicating very poorly compacted soil.

who is to address the cause of the soils problem

The applicant preloaded the area inside the building and a 20' wide area immediately outside the outer walls of the building with a 20' high sand pile (2.2 kips per square foot) to accelerate the settlements and to achieve a stable foundation prior to making connection to the building with outside pipe lines. As a result of this preloading, the building settled further with a total maximum settlement of 7.45" (4.25"+3.2") at southeast corner and a total minimum settlement of 3.49" (2.09+1.5) at northwest corner. The settlement data at the corners obtained after the surcharge indicated warping of the foundation still existed.

WARP

Joe Kane check sentence

With the changed density of the fill material due to preloading on which the Diesel Generater Building is founded, the soils and foundation information pertaining to this building provided in the FSAR are no longer valid. The bearing capacity, settlement predictions for the 40 year plant lifespan must be reevaluated on the basis of the soil paramenters obtained from the test results on representative soil samples taken from the actual fill material. Has this been done - when will it be done

In response to 10CFR 50.54(f) requests, the applicant has furnished information regarding settlements and bearing capacity of soils under the footings of the Diesel Generator Building. The Corps of Engineers in ~~these~~ their report of 7 July 1980 requested additional information needed to evaluate the adequacy of the foundation of the Diesel Generator Building and others. The information needed was explicitly spelled out in the 7 July 1980 report which was transmitted to the applicant on 4 August 1980 by the NRC. The applicant responded to the request through its Amendment 85 to the operating license request and Revision 10 to 10CFR 50.54(f). The details provided in the applicant response were not adquate to evaluate the stabitily of the structure. The Corps of Engineers comments of 16 April 1981 on Amendment 85 and Revision 10 to 10CFR 50.54(f) shows the reasons for the applicant's response not being adequate. (attached) s

(p. _)

Say generally why if possible

~~In addition, I would like to inform the Board that~~ Severe damage to the integrity of the structure has already been done due to the settlements caused by the weight of the structure and the additional settlements caused by the preloading. Many diagonal tension cracks have appeared on the east wall of the structure indicating the structure has been subjected to severe stresses and strains due to differential settlements. There is no guarantee that these cracks have stabilized and would not propagate when the structure will be subject to enviromental loads (earthquake, tornado, servere termperature variations, wind load etc.) in future.

What does Rinaldi have to say about this?

attach a simple diagram!

what happened

(copy attached)

what?

(c) Service Water Building Foundation.

The Sevice Water Building is founded partly on the original ground and partly on the fill material. The foundation elevation for the portion of the structure founded on original ground is 587.00 and that for the portion on fill material is 617.00. The walls of the portion founded on fill cracked indicating settlement of the building. The applicant, as in case of the Diesel Generator Building began a soil investigation program which indicated some poorly compacted soil underneath the foundation. As per applicant's MCAR 24 Interim Report 6, June 11, 1979, the fill material was summarized as soft to very stiff clay and loose to very dense sand backfill. Some areas of the fill material under the northern part of the structure have not been sufficiently compacted.

doesn't discuss the new proposed fix

As a corrective action, the applicant proposed to support the north wall on 16 underpinning piles driven into the glacial till through predrilled holes in the fill material. The design capacity of each piles was to be 100 tons. The piles were to be placed a few inches away from the outside face of the north wall and was to be connected with the wall with shear connection or other mode dowels. Figure 83 of the applicant's MCAR 24 Interim Report 6 shows the preliminary arrangement of the underpinning system. (attached)

(copy attached)

The Corps of Engineers performed the preliminary review of the applicant's proposal and wanted more information to check the adequacy of the proposal to carry the loads under the static and seismic conditions. The information required to complete the review was included in the Corps of Engineers' letter report of 7 July 1980 (Question 40, 10CFR 50.54(f)). A copy of the report was trasmitted to the applicant by the NRC on 4 August 1980 for its response. The applicant's response to question 40, Amendment 85 to the operating license request, and revision 10 to 10CFR 50.54(f) was reviewed. The information provided by the applicant was found to be inadquate. The Corps of Engineers review comments of 16 April 1981 on Amendment 85 shows the details of the information still required. (copy attached)

(d) Auxiliary Building Electrical Penetration Areas Feedwater Isolation Value Pits.

The Electrical Penetration Areas (EPA) and the Feedwater Isolation Valve Pits (FIVP) for the Reactor Units 1 and 2 are founded on the plant fill area. The Reactor Buildings and the main body of the Auxiliary Building are founded on glacial till. A soil investigation by the applicant for all Category-I Structures founded on fill material, after the discovery of the excessive settlements of the Diesel Generator Building, indicated layers of loose sand and soft clay (MCAR 24, Interim Report 6, page 3) in the soil mass under the Electrical Penetararion Area and the Feedwater Isolation Value Pits. The applicant, on page 4 of MCAR 24, Interim Report 6, concluded that approximately 15 feet of the backfill material under the Electrical Penetration Areas and the Feedwater Isolation Valve Pits has not been sufficently compacted. we agree?

attached?

use all this stuff + then say on Feb 23, 1981 CPC told us about 6 on entirely new fix — + how we see that

attach a simple diagram

Because of the poor soil conditions (loose sand and soft clay) attributed to inadequate compaction, the actual soil parameters (shear strength parameters, compressibility coefficients) of the soil are not the same or better than the assumed design soil parameters provided in the FSAR. The values of ultimate bearing capacity provided in Table 2.5-14 of the FSAR for the EPA and FIVP are not valid. Also the settlement values for these structures provided in the FSAR would change. As a matter of fact, the effects of the poor soil conditions under the foundations have already become visible in the form of cracks in the walls of the structures, and the structures have partially lost their structural integrity. The capability of these structures to withstand environmental loads (earthquake, tornado, etc.) is questionable.

Does Harry say this or is this S.E.B.?

As a corrective action, the applicant has proposed the following actions:

The unsuitable backfill materials (inadequately compacted materials) under the Feedwater Isolation Valve Pits of both Units 1 and 2 will be removed and be replaced by lean concrete (fc'-2000 p.s.i.). The Electrical Penetration Areas will be supported on caissons. The caissons will be provided under the structures at their free ends (near their junctions with the FIVP), and at the other ends, supports to the EPA will be provided by the control tower with which they are built monolithicly.

show this is needed at CP stage

The Corps of Engineers found the applicant proposal at a conceptual stage and requested the applicant to furnish analyses for capacity of caissons, soil parameters used in the analyses, constructuion plans and specifications etc. for a ~~complete~~ review to determine the adequacy of the proposal. The details of the information requested are given in the Corps of Engineers' Letter Report of 7 July 1980. The NRC transmitted this report to the applicant on 4 August 1980 for its response. The applicant's response to the Corps request regarding the Auxiliary Building EPA and FIVP (Question 42 of the letter report) was reviewed and the information furnished by the applicant was not adequate to evaluate the adequacy of the applicant's proposal. The Corps of Engineers review comments of 15 April 1981 on Amendment 85 shows the needed information, and the analyses to complete evaluation of the proposal.

does this mean we are happy with Am 85?

Does this include latest fix — I don't think so —

do as before — on 2-15-81 they gave us a whole new fix-

(e) Borated Water Tanks,

The Borated Water Tanks were built on the fill material despite the numerous evidences that compaction of fill material was questionable (settlements of the Diesel Generator Building, cracking of the Service Water Building and portions of the Auxiliary Building founded on the fill materials). Prior to their construction, the NRC through Question No.6, 10CFR 50.54(f) requested the applicant to provide justification for constructing the safety-related tanks on the questionable fill material.

If NRC told them in 78, say so

Based on some preliminary soil investigation, the applicant concluded that the soil conditions in the area where the tanks were founded would be adequate, and it completed the construction of the tanks. The Corps of Engineers reviewed the applicant's response to Question 6 and 31, 10CFR 50.54(f) which pertain to foundations of the two Borated Water Tanks, and requested soil information needed to evaluate the adequacy of the tanks foundation. The tell what this is — very few ~~board~~ members could know this

details of the requests are included in the Corps of Engineers Letter Report of 7 July 1980. The NRC transmitted the Corps' requests to the applicant on 4 August 1981 for its response. The applicant's response to the requested information as to the tanks (Question 43) was reviewed by the Corps of Engineers and was found to be inadequate to complete the review. The soil modulus of subgrade reactions used by the applicant to analyze the ring beam foundations of the tanks was not compatible with the type of soil conditions prevailing under the Borated Water Tanks. It appears that the applicant has performed no test to evaluate the variation in the modulus of subgrade reaction because of the varying density of the soils along the depth as well as across the diameters of the tanks as indicated by the borings. The details of the discrepancies noticed in the applicant's response to the Corps of Engineers' request of 7 July are included in the Corps review comments of 16 April 1981 on Amendment 85. It has been reported recently that the ring beams of both the tanks have cracked severely when the tanks were filled with water to perform load tests of the foundation soil.

? generally Why.

heavy

tell sign of this - whats the status of the fix

(f) Underground Diesel Fuel Tank Foundation Design.

The Underground Diesel Fuel Tanks are buried in the questionable fill materials, and are anchored to concrete pads with their bottom elevation at 612.00. The tanks are covered with fill material. The Corps of Engineers has reviewed the information submitted by the applicant in response to NRC Question 31, 10CFR 50.54(f) and to the Corps of Engineers' requests forwarded to the applicant on 4 August 1980. The applicant's response was not satisfactory. The applicant must demonstrate by analysis that the tanks are safe against uplift pressure. Also, a settlement analysis of the tanks due to seismic events is necessary because some of the boring logs indicate a layer of loose sand below the pads. The details of the information required to complete the review are given in the Corps of Engineers comments of 16 April 1981 on Amendment 85.

generally why

(g) Underground Utilities Joe help me on this one —

Because of the questionable plant area fill discovered after the excessive settlements of the Diesel Generator Building, it became necessary to investigate for the additional stresses developed in the Seismic Category I pipings due to the settlements of the fill material. Because of the natural soil structure interaction between the piping and the surrounding soils, the pipes conformed to the configuration of the settling soil mass resulting in bending of the pipes, introducing bending stresses in the pipes beyond the permissible limits.

The Corps of Engineers evaluated the stresses in one of the pipes (26" dia OHBC-54) using the information furnished by the applicant in response to the 10CFR 50.54(f) requests. As shown in the Corps of Engineers Letter Report of 7 July 1980, the stresses developed due to curvature caused by the settlements was found to be 130 KSI exceeding the permissible limit by more than 100%. A copy of the Corps of Engineers Letter Report was forwarded to the applicant by the NRC on 4 August 1980. But the applicant has not yet responded to the Corps of Engineers evaluation of the underground piping stresses.

'this Harrejo'?

make sure cappucci sees + responds to all this.

The plant fill around the Diesel Generator Building was consolidated under the preload, therefore, the Category-I water circulating piping within this area were subjected to additional settlements. The Corps of Engineers requested the applicant to perform a thorough inspection of these piping with video cameras and sensing divices for possible areas of crackings and openings. The applicant's response to this request (Amendment 85 and Revision 10 to 10CFR 50.54(f)) was not satisfactory. As stated in the Corps of Engineers' review comments of 16 April 1981 on Amendment 85, it not possible to evaluate the adequacy of the piping in absence of the requested information.

During the site visit on 19 February 1980, the Corps of Engineers representatives observed three instances of what appeared to be degradation of rattlespace at the penetrations of Category-I piping through concrete walls. The Corps of Engineers Letter Report of 7 July 1980 explains these descrepancies in detail and requests information from the applicant to evaluate the adequacy of the rattlespaces.

The applicant's response received through Amendment 85 to the operating license request, and Revision 10 to 10CFR 50.54(f) was reviewed by the Corps of Engineers and some discrepancies in the applicant's information were noticed. The Corps of Engineers' comments of 16 April 1981 show the discrepancies noticed and the clarifications required from the applicant.

The stability of the two reinforced concrete discharge pipes which exit the Service Water Pump Structure, run along either side of the Emergency Cooling Water Reservior, and ultimately enter into the reservior, have not been demonstrated by the applicant to be adequate. The Corps of Engineers' Letter Report of 7 July shows the information required by the Corps to complete review of the stability of these pipes. The applicant's response to this request was very unsatisfactory. The applicant has not used the proper soil parameters to analyze the stability of dike's bases from which these pipes derive their support. The Corps of engineers review comments of 16 April 1981 on Amendment 85 shows the details of information still needed to complete the review.

why

(h) Cooling Pond. tell what portion is Cat I + why

A detailed review of the FSAR has indicated that the applicant has taken no record sampling during construction of the dikes to verify the design assumptions as to the soil shear strength parameters. It has performed no field control tests for compacted soil in the dikes above elevation 620+. Thus, the applicant has not demonstrated that the required compaction of the fill material in the dikes has been achieved. In recognition that the type of the embankment fill and the compaction control used to construct the dikes for the cooling pond were the same as for the problem plant fill, the Corps of Engineers requested reasonable assurance that slopes of the Category-I Emergency Cooling Pond (baffle dike and main dike) are stable under both the static and the dynamic loads. The details of the information required to evaluate the stability of the dikes, slopes and the Category-I pipes buried under the slopes are given in the Corps of Engineers' Letter Report of 7 July

?

Harry should use ~~better~~ his own words

1980, which was transmitted to the applicant by the NRC on 4 August 1980. The applicant's response was received through Amerndment 85 to the operating licence request and Revision 10 to 10CFR 50.54(f) requests. The Corps of Engineers reviewed the response and found the information provided in the response inadequate for the review. The Corps of Engineers' review comments of 16 April 1981 on Amendment 85 show the discrepancies and the information needed by the Corps to complete the evaluation of the stability of the slopes and the concrete discharge pipes.

2 → The operating Cooling Pond Dikes are not Category I Structures. However, a high level of safety should be requred for these dikes unless it can be assured that a failure will not: (a) endanger public health and properties, (b) result in an assault on the environment (c) impair needed emergency access to the plant power block.

(i) Site Dewatering. let Harry + Joe do this?

get Joes comments

The applicant's soil exploration of the plant fill indicated layers of loose sand under several Category-I Structures, which are subject to liquefaction under siesmic events. To eliminate the possiblity of liquefaction, the applicant proposed to lower the water table to an elevation of 595 by a permanent dewatering device. Most of the loose sand layers were above elevation 610.

6

The Corps of Engineers reviewed the materials furnished by the applicant as to the permanent dewatering and requested additional information as outlined in its Letter Report of 7 July 1980. The information furnished by the applicant in response to the Corps request was mostly satisfactory. However, some minor discrepancies still exist. The Corps' review comments of 16 April 1981 Amendment 85 show the discrepancies noticed. It is emperative to resolve the discrepancies to assure adequate dewatering. i?

why is Harry doing this? → (j) Seismic Analysis of the Structures on Plant Fill Materials.

The applicant's seismic analyses were reviewed by the Corps of Engineers. The methods of analysis followed appeared satisfactory, however, certain parameters such as damping ratio (actual damping as a percent of critical damping) and shear modulus of the soil used in the analyses were not known to the reviewers. The shear modulus computed using the shear wave velocity provides a very low strain shear modulus and is not applicable to seismic events. The applicant has to clarity these points.

explain

(9) Did Corps of Engineers request soil exploration and testing? If so what were the reasons for the request? stop

The soil exploration and testing were initially requested by the Corps of Engineers in its letter of 27 March 1980 to Dr. Robert E. Jackson of the NRC and were later revised in its letter of 16 April 1980.

Because of the inadequately compacted plant fill materials, the physical properties (shear strength parameters, compressibility coefficients, etc.) of

What is the purpose - what are we trying to communicate?

the fill materials have degraded from those used in the design of the foundations of the several Category I structures and the piping deriving its support from the plant fill. Also, the load on the soil mass below the footings would be considerably increased due to proposed permanent dewatering of the site. The effects of degraded physical properties of the soil are apparent from the excessive settlements of the Diesel Generator Building and the crackings of the walls of the several Category-I Structures (Service Water Structure, Auxiliary Building, Diesel Generator Building) founded on the inadequately compacted fill.

In view of these facts, it was imperative to determine the actual soil properties of the plant fill and reevaluate the bearing capacity of the foundation soils and the predicted settlements of the structures, using the actual soil parameters. The bearing capacity and settlement information provided in FSAR no longer valid because of the changes in the soil physical properties and the increased load on the soil mass due to dewatering. The Corps of Engineers requested the applicant to perform consolidation tests and triaxial shear tests on undisturbed samples taken form the plant fill area where Category-I structures are located.

(10) What is an undisturbed sample and why is it necessary to test undisturbed samples?

Preconstruction site investigations are required to determine geotechnical conditions that affect the feasibility of a project, design, cost, performance, and ultimate safety of the structure. It is necessary that the investigations be adequate in terms of thoroughness, suitability of methods used, and quality of execution of the work to assure that all important conditions have been detected and reliably evaluated. An important phase of any site investigation is obtaining high quality, undisturbed samples of subsurface materials. In the case of the Midland Nuclear Power Plant, because of the changed soil conditions due to inadequate compaction, testing of X undistrubed samples is imperative to ascertain the actual soil design parameters.

is this ~~Joe~~ clear to Joe?

In the current state of the art of soil sampling, the term undisturbed sample means a sample that is obtained and handled by methods designed to minimize the disturbance to the sample that might occur during the sampling, handling, shipping, storage, extrusion, specimen preparation for testing and the laboratory setup processes. In fact, there is no such thing as truly X undistrubed sample, primarily for two reasons: (1) a sampling tube displaces a certain amount of soil which inevitably produces strain and some disturbance to the sample; and (2) even in perfect sampling, and imaginary process that eliminates disturbance due to soil displacement, the state of the stress into the soil sample undergoes a complex, and of some degree indeterminate history of change during sampling and handling .

The purpose of obtaining soil samples and testing them, is to determine the physical properties of the soils which are going to provide support for the structures to be built. The importance of the structure dictate the quality of the soil information to be obtained from the test results. For ordinary structures where public safety is not threatened in case of any failure, a very high quality undisturbed soil sample may not be necessary. But in the case of a Nuclear Power Plant where the failure of the structures involved in the plant must be guarded at all costs, it is imperative to have the highest quality undisturbed soil samples for testing to obtain the physical properties the soils possesses in its natural state under the foundation.