# MIDLAND - PERMANENT DEWATERING

2/138

Subject: Summary of CPCo Response to NRC Questions 24, 45 & 47 on permanent dewatering relative to geotechnical engineering issues.

1. Refer to P. Hadala's letter of Jun 4, 1980 to District Engineer, Detroit. P. Hadala's study on potential liquefaction covered:
   a. Adopted Magnitude 6 earthquake & peak ground acceleration of 0.19g.
   b. Calculated the minimum SPT value required to have a F.S. = 1.5 against liquefaction type failure, assuming GWT @ El. 610 and using the Seed-Idriss Simplified analysis approach (w/M=6). Results indicated a required N = 14 w/GWT @ El. 610 & N = 19 w/GWT @ El. 595.
   c. Reviewed 250 SPT values on cohesionless soils (both natural till soils and plant fill - included sands and silty & clayey sands) BELOW El. 610. Determined that only 27 SPT values out of the total 250 values, (See Fig. on Incl. 4) below El. 610 failed to meet or exceed the minimum SPT values (which indicated a F.S. = 1.5+). 23 of these tests were on plant fill material and 4 were on natural soils. (Actually 38 tests failed to meet minimum SPT criteria but 11 of these 38 tests were in natural soils which had plant fill eventually placed overtop the natural soils. P. Hadala indicates these 11 blow counts, when increased by a factor of 2.3 to compensate for the increased effective overburden pressure due to the fill, would reach SPT values where liquefaction would not be indicated)
   d. Of the 23 SPT test values which fail to satisfy minimum SPT criteria, 9 are located where remedial measures ARE NOT PLANNED & 7 of the 9 are near the DGB. Since remedial measures will eliminate the liquefaction concern @ 23-9 = 14 locations, only the 9 are a concern. Hadala concludes that these 9 locations

indicate non-continuous layers (believes are localized pockets of loose cohesionless soils) and that <u>LIQUEFACTION of SOILS BELOW ELEVATION 610 IS NOT A PLANT SAFETY PROBLEM IF THE GWT IS MAINTAINED AT OR BELOW ELEVATION 610</u>. (This does not address the soils above El. 610 which are considered liquefiable)

## SUMMARY OF CPCo Response to Q47

2. In response to Hadala's questions (Jun 4, 1980), CPCo responds to Q47.

Q47.4) a. During NORMAL OPERATION of permanent dewatering system, the GWT will be <u>maintained</u> AT OR BELOW EL. 595. (Anticipate lower elev. @ pumps in order to maintain El. 595)

b. <u>What action will CPCo take</u> if GWT were to rise above El. 595. in ANY permanent observation well, piezometer or monitoring well? An <u>Operational Tech SPEC will require</u>:

(1) Immediate remeasuring of GWT @ involved well or piezometer & in vicinity.

(2) If GWT > <u>El. 595</u> is verified, the pumping rate of system WILL BE INCREASED [can be increased by raising rate of already operating wells or putting into service backup interceptor wells and/or standy area wells (See Fig 47-11 which is slightly revised by Fig 1 of Sept. 16, 1981 submittal) or installing pumps in six monitoring wells.

(3) An examination of wells closest to high GWT level will be made to identify any malfunction (pump failure, power outage, pipe failure or leak, high level switch or timer failure, etc.). Flow rates will be checked. If electric supply power were to fail, a BACK-UP DIESEL GENERATOR would be started (can be initiated in 24 hrs.

(4) Storing of replacement parts ON SITE will be required (for system repairs)

2.c (cont.)

ALARM SYSTEM Fig. 47-6

Where are observation wells located that will be used to monitor GWT during plant life? (See Fig. 47-1 which in addition to showing location of observation wells also shows permanent piezometers & monitoring wells)

Basis for selecting location of permanent observation well & piezometer

1. Relation to critical structure
2. Areal location to check effectiveness of dewatering system
3. Position within backfill or natural sands
4. Timely response to changes in cooling pond level
5. Drawdown information during aquifer pumping tests
6. Response to construction dewatering

PERMANENT MONITORING WELLS

There are six wells for permanent monitoring (See Fig. 47-11). Two are near DGB, Two are near SW Structure & Chlorination Bldg. and Two are near Auxiliary Bldg (Railroad Bay end). The six Perm. monitoring wells can be converted to dewatering wells. if need were to arise.

Each PERMANENT MONITORING WELL will contain ULTRASONIC LEVEL TRANSMITTER that WILL CONTINUOUSLY record GWT levels. If GWT El. 595 is reached @ any of the six permanent monitoring, a RED LIGHT WILL FLASH on control panel of guard house. Each Perm. Monitoring Well also contains 1½"Φ Casagrande open piezometer. to check on ultrasonic level transmitter.

Oct. 12, 1981
4 of 4
J. Kane

Recommendations by P. Hadala
FIELD TEST ON PERMANENT DEWATERING SYSTEM

1. With pond @ El. 627, determine FOR EACH OBSERVATION WELL a plot of water level versus time following a shutoff of the dewatering system
2. Continue shutoff test until water reaches El. 610 in each well OR the sum of the time intervals allotted for repair and the time interval needed to accomplish shutdown (should repair be unsuccessful) be exceeded - whichever occurs first.

---

CPCo response to Field Test (See pg. 47-6 & 47-7)

- CPCo will conduct full scale field test by shutting off entire dewatering system after GWT has been lowered to el. 595 & cooling pond is at El. 627.

During test, GWT vs. time curves will be plotted to evaluate recharge time at DGB and Railroad Bay. (Believe test will demonstrate that there is more than 90 days recharge time before GWT reaches El. 610 - Refer to pg. 24-1 response,

4/2/81
1 of 1
J. Kane

# SUMMARY OF RESPONSE by CPCo to Q24

~~Request 47 Site Dewatering~~
~~Response to 47(1), Part b (Pg. 47-4 of Amendment 5)~~

— Rev. 5 2/80

Previous response to Question 24(c) gave the following information on monitoring, and operation of dewatering system

- Only 44 wells are required to <u>INITIALLY</u> lower the GWT (20 interceptor wells pumping @ 7.5 gpm each near cooling pond & 24 area wells pumping @ 5 gpm)
- Once GWT is lowered - only 22 wells anticipated to be needed. That leaves 22 of the 44 wells for back-up plus an additional 20 wells for backup (near intake & pump structure) TOTAL of 64 WELLS
- Each submersible pump has capacity of 8 to 10 GPM
- In addition to the 64 wells - have additional wells from temp. dewatering during construction
- Have calculated that it would take 90 Days for GWT to reach El. 610 if the ENTIRE dewatering system went down

pg 24-20 — PROPOSED MONITORING to detect system failure

a. Monitoring of groundwater levels
b. Periodic testing of groundwater quality
c. Testing discharge water to determine quantity of sand being removed from wells
d. Install 6 observation wells w/ continuous recorders & warning devices (2 @ DGB, 2 @ AUX BLDG, 2 @ SW - Circ. water Structures) & monitor WEEKLY
e. A <u>number</u> of the 80 observation wells already installed will be selected for PERMANENT monitoring

pg. 24-21 f. To check decrease in well efficiency - EACH WELL will have a 1" ϕ observation pipe w/ filter pack where GWT can be measured on ROUTINE BASIS and compared w/ recorded pumping rates

<u>g. Monitoring will be performed UNDER A Q-A program</u>

h. Remedial measures if WELL or GROUP of WELLS become INOPERABLE - cleaning of well - repair or replace screen - install new dewatering well

i. Will store replacement parts

# SUMMARY OF RESPONSE ~~TO~~ Q 42

4/3/81
1 of 1
J Kane

also see pg.

also see pg. 47-5 for procedure to be followed if any perm obserr. well, piez. or monitoring well indicates level higher than PI-597 Groundwater

Response to 42 (Part 2a)

Is this same frequency for measuring drawdown of GWT to be used in developing permanent wells?

See Table 42-2 pg. 42-17 for Frequency of Measurements on Construction Dewatering System (Measurement of Groundwater Level)

CPCo Procedure for Testing for Pumping Soil Particles from Wells

pg. 42-7 Prior to starting a dewatering subsystem, EACH WELL IS

What test? Standard?

Over what period of time are 3 tests made. Impt. if significant erosion is occurring

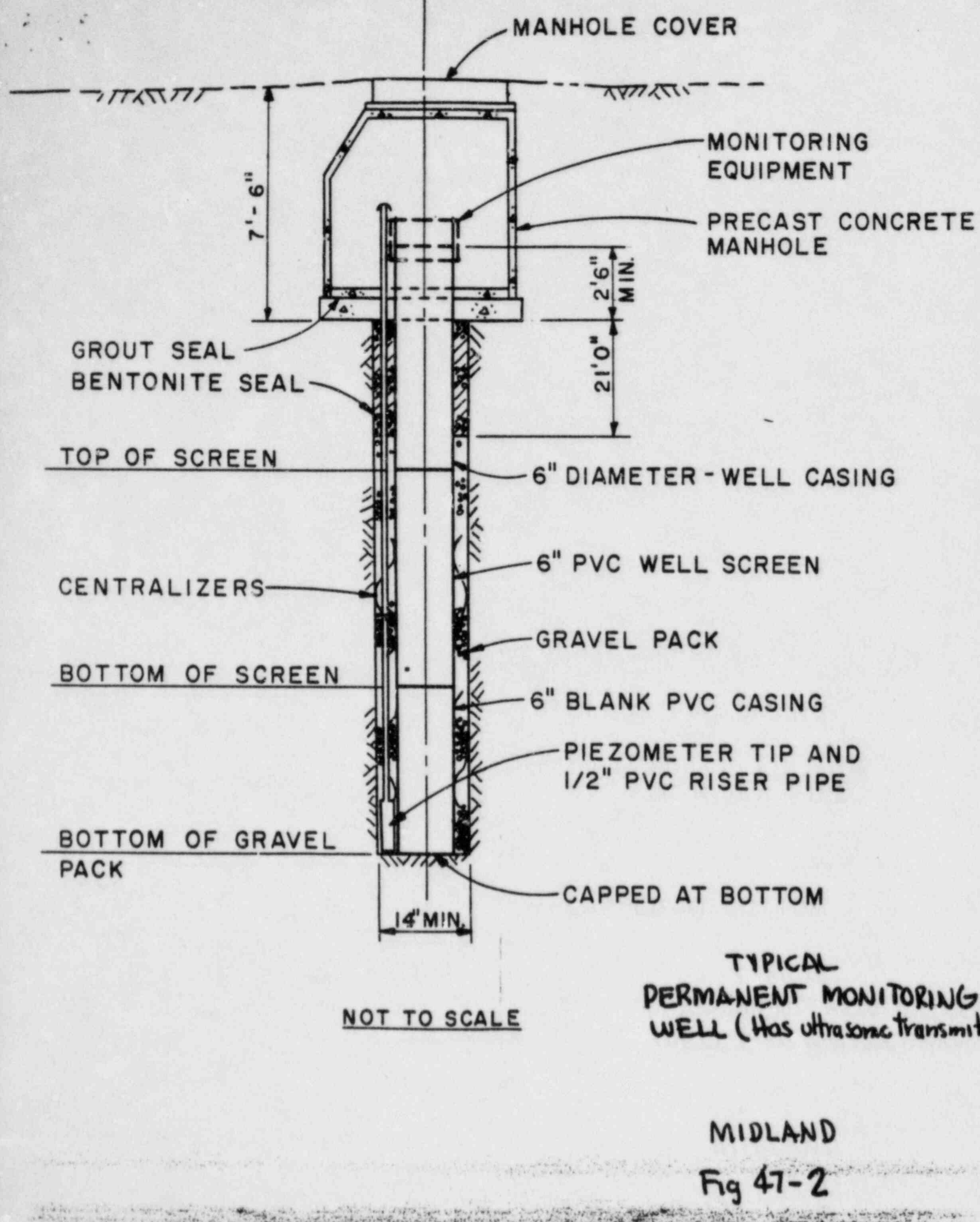
1. Sampled & tested to determine concentration of material coarser than 0.05mm (is silt size & smaller than 200 sieve)
2. If after THREE TESTS the concentration of soil EXCEEDS 10 ppm by weight - the well is REJECTED & DISCONNECTED from system.
3. EACH, individual well is TESTED MONTHLY for loss of soil fines during OPERATION

4. Water SAMPLES from the COMBINED SYSTEM overflow are TESTED TWICE a week @ onsite lab. Calculated (avg. concentration) each Monday & Thursday

Problems with using average. Development of problem may go unattended because of averaging effect

If AVG. concentration since pumping began EXCEEDS 10 ppm by weight the subcontractor has 24hrs to correct situation. If uncorrected after 24hrs, each individual would then be tested until the well or wells producing the material are located & removed from the system

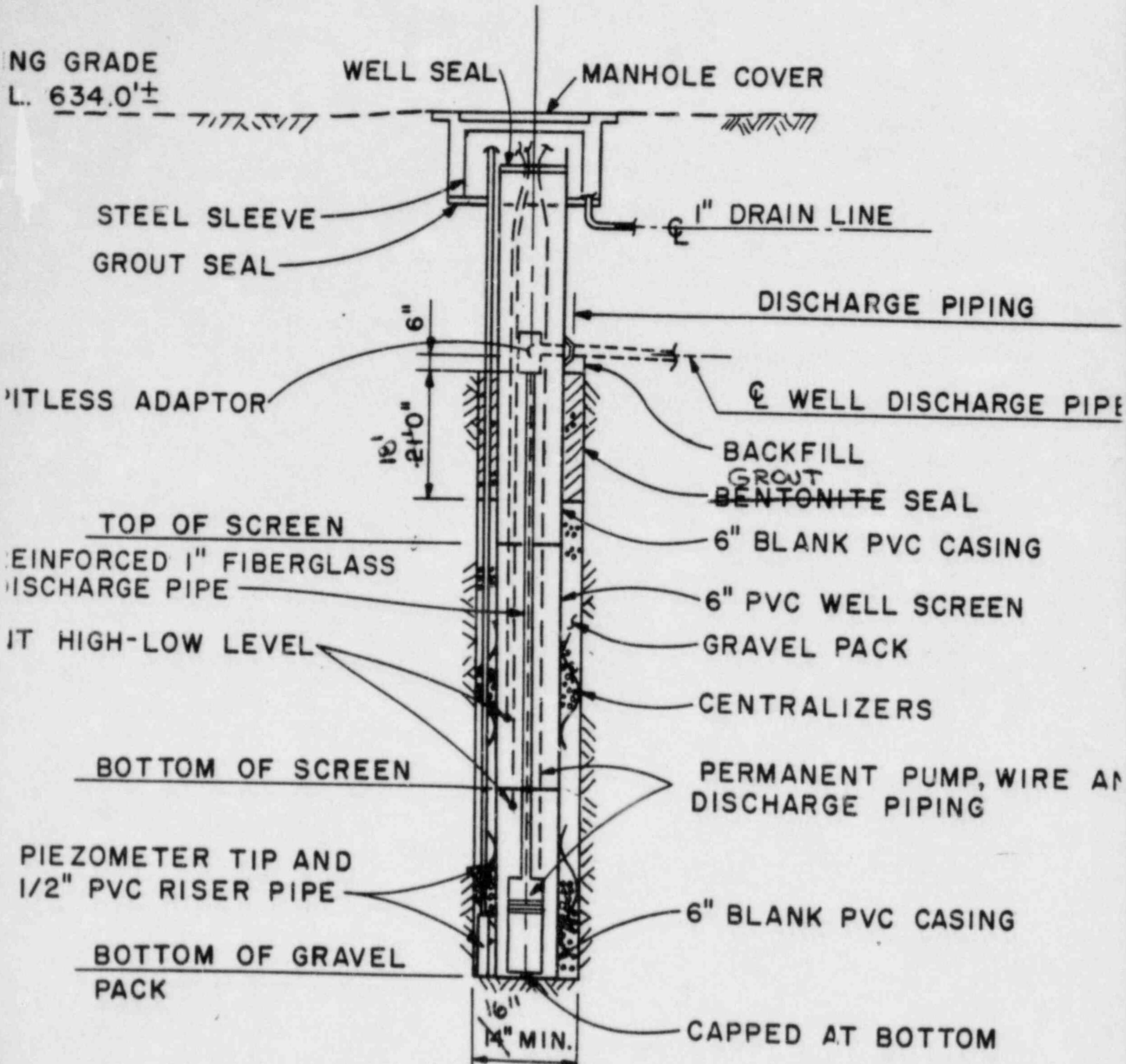
pg. 47-14 AFTER the step drawdown test, the sand concentration will be MEASURED THREE TIMES a WEEK for the FIRST WEEK
TWICE A WEEK thereafter during initial operation
? then monthly




TYPICAL PERMANENT MONITORING WELL (Has ultrasonic transmitter)

MIDLAND

Fig 47-2



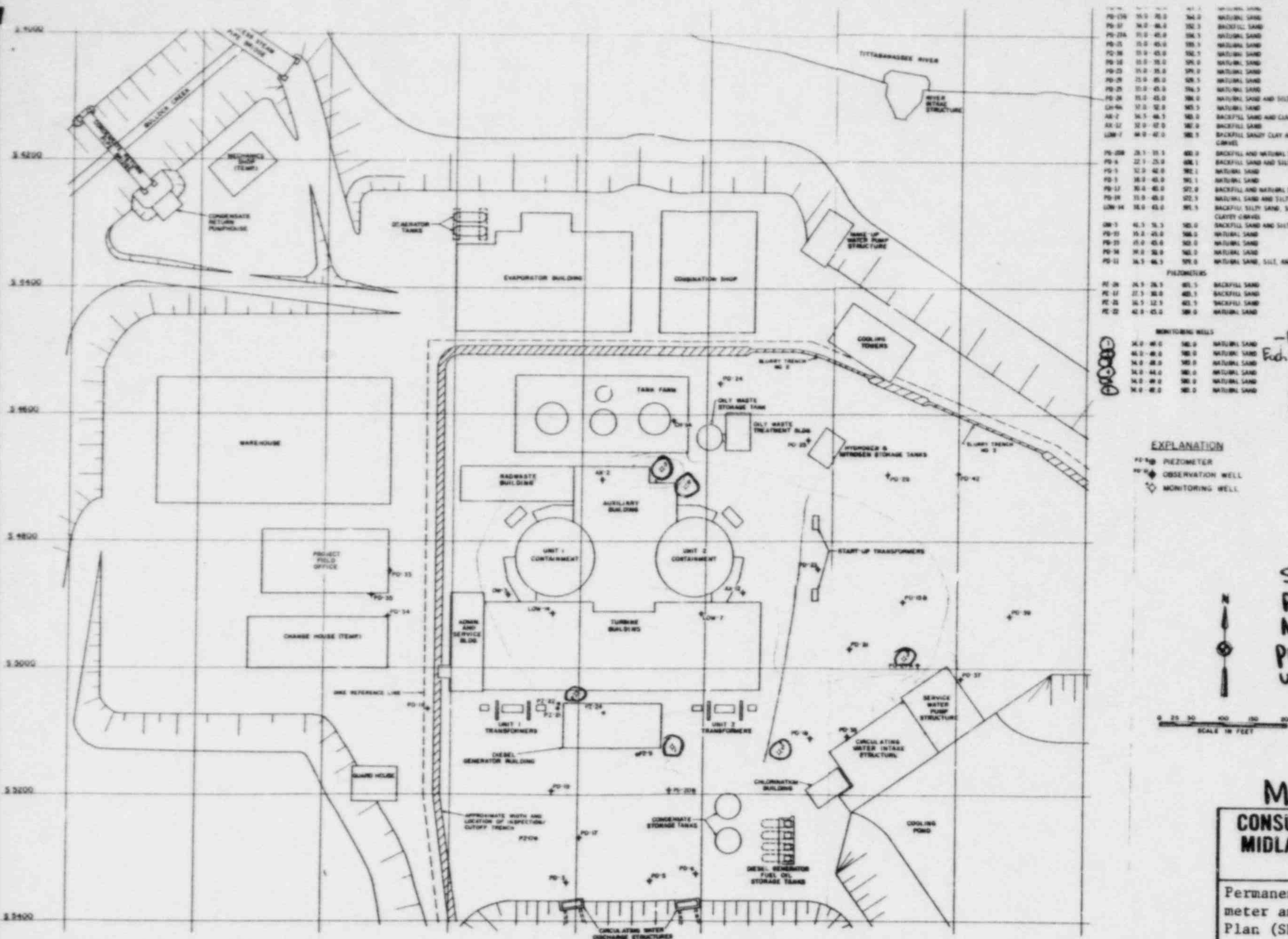

NOT TO SCALE

TYPICAL
PERMANENT DEWATERING
WELL

MIDLAND

Fig. 47-3

Changes in purple result from
the Sept 16, 1981 letter from J.W. Cook
to H. Denton on "Permanent Dewatering




-pg. 47-6
Each will contain an ultrasonic level transmitter that will continuously record groundwater level & also have on ALARm

SHOWS LOCATIONS OF PLANNED PERMANENT MONITORING (Obs. wells, piezometers & monitoring wells)

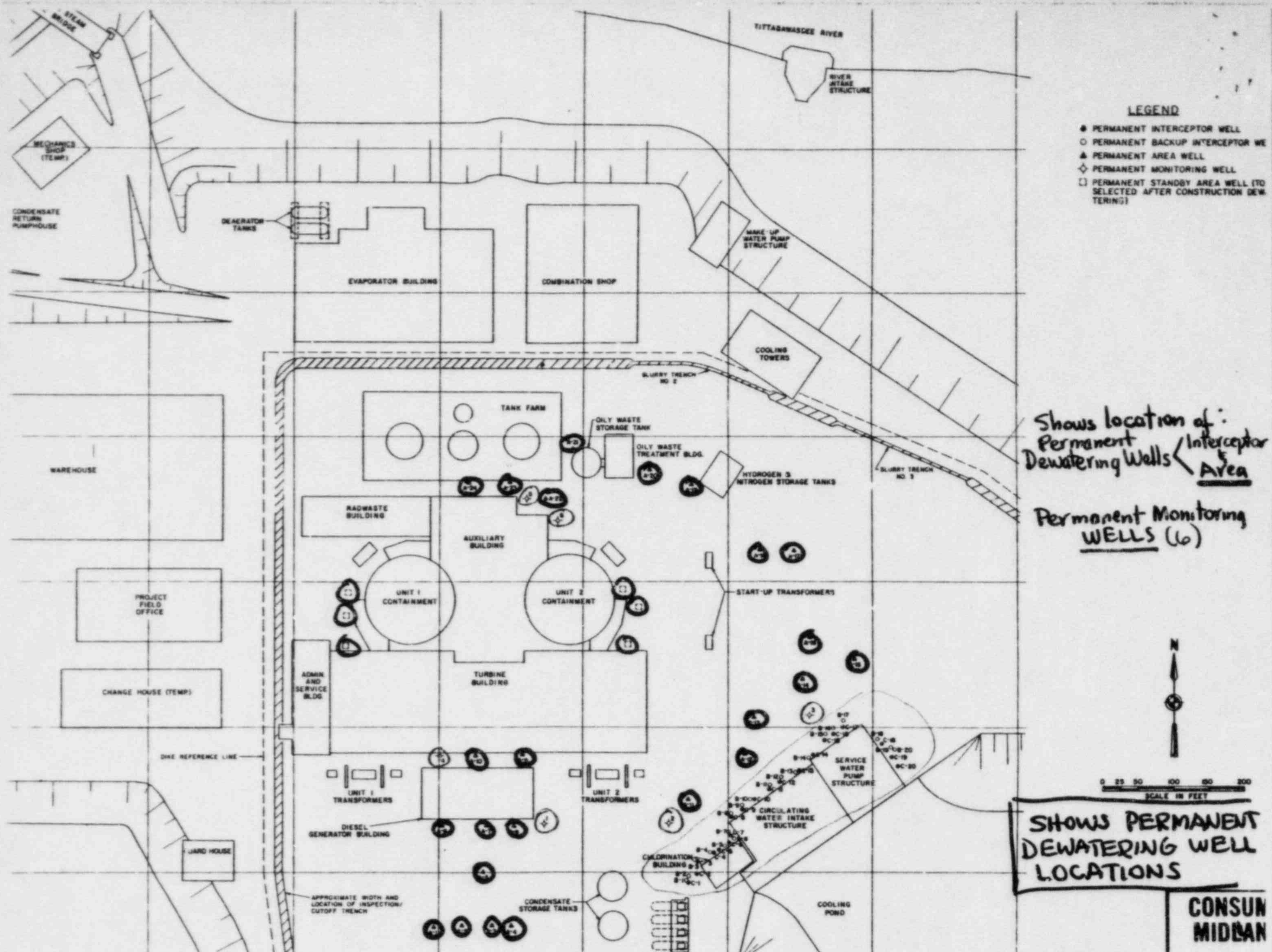
MIDLAND

**CONSUMERS POWER COMPANY**
**MIDLAND PLANT UNITS 1 & 2**

Permanent Observation Well, Piezometer and Monitoring Well Location Plan (SK-G-456, Rev 0)

Figure 47-1

11/80 Revision 10




## LEGEND

- ● PERMANENT INTERCEPTOR WELL
- ○ PERMANENT BACKUP INTERCEPTOR WE
- ▲ PERMANENT AREA WELL
- ◇ PERMANENT MONITORING WELL
- ☐ PERMANENT STANDBY AREA WELL (TO SELECTED AFTER CONSTRUCTION DEW. TERING)

Shows location of: Permanent Dewatering Wells < Interceptor & Area

Permanent Monitoring WELLS (6)

SHOWS PERMANENT DEWATERING WELL LOCATIONS

CONSUM
MIDLAN

MIDLAND
Fig 47-11

Rev. 10

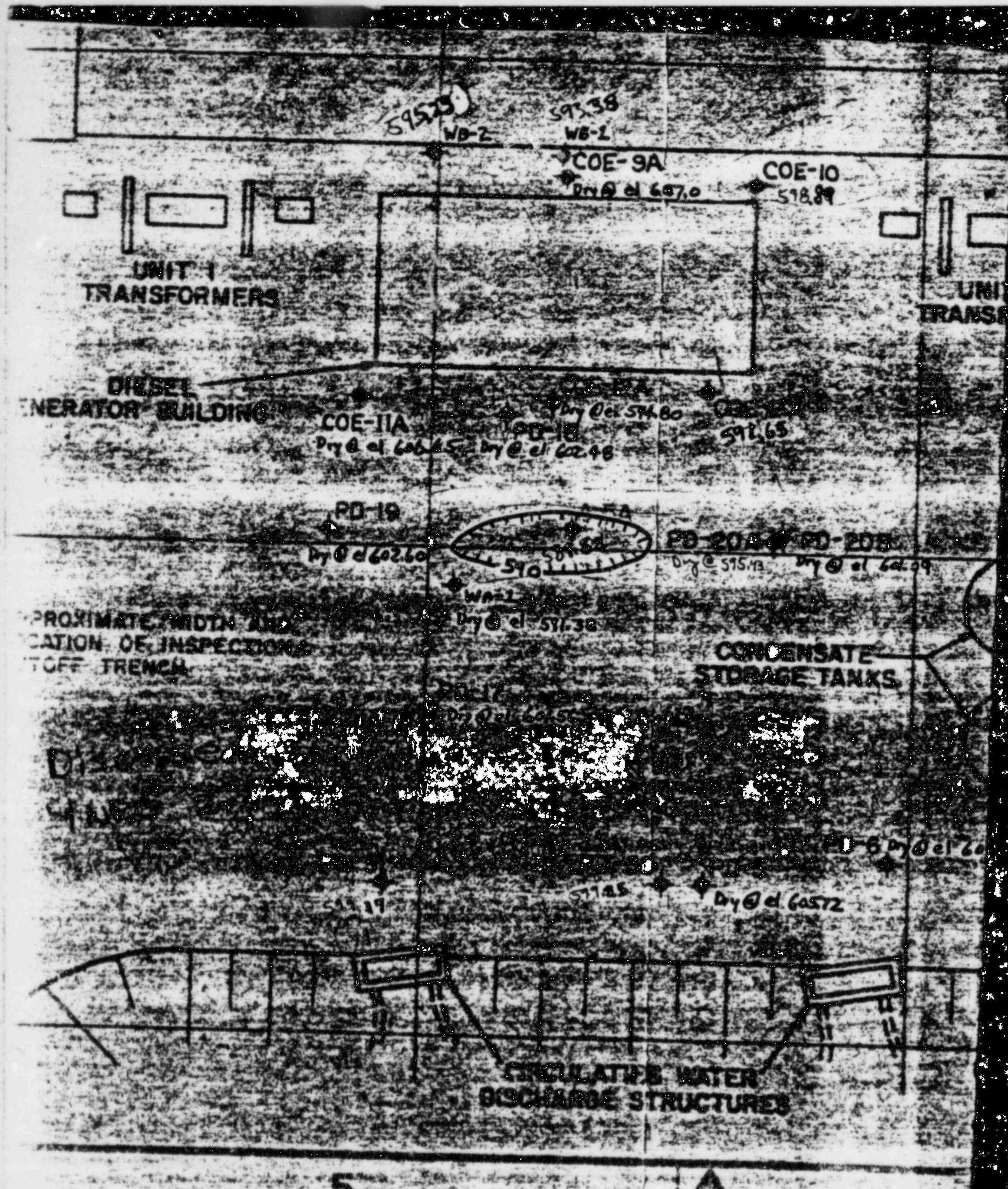
This Figured is revised by J.W. Cook's letter of Sept 26 to H. Denton (Fig. 1)

Perm Dewatering File

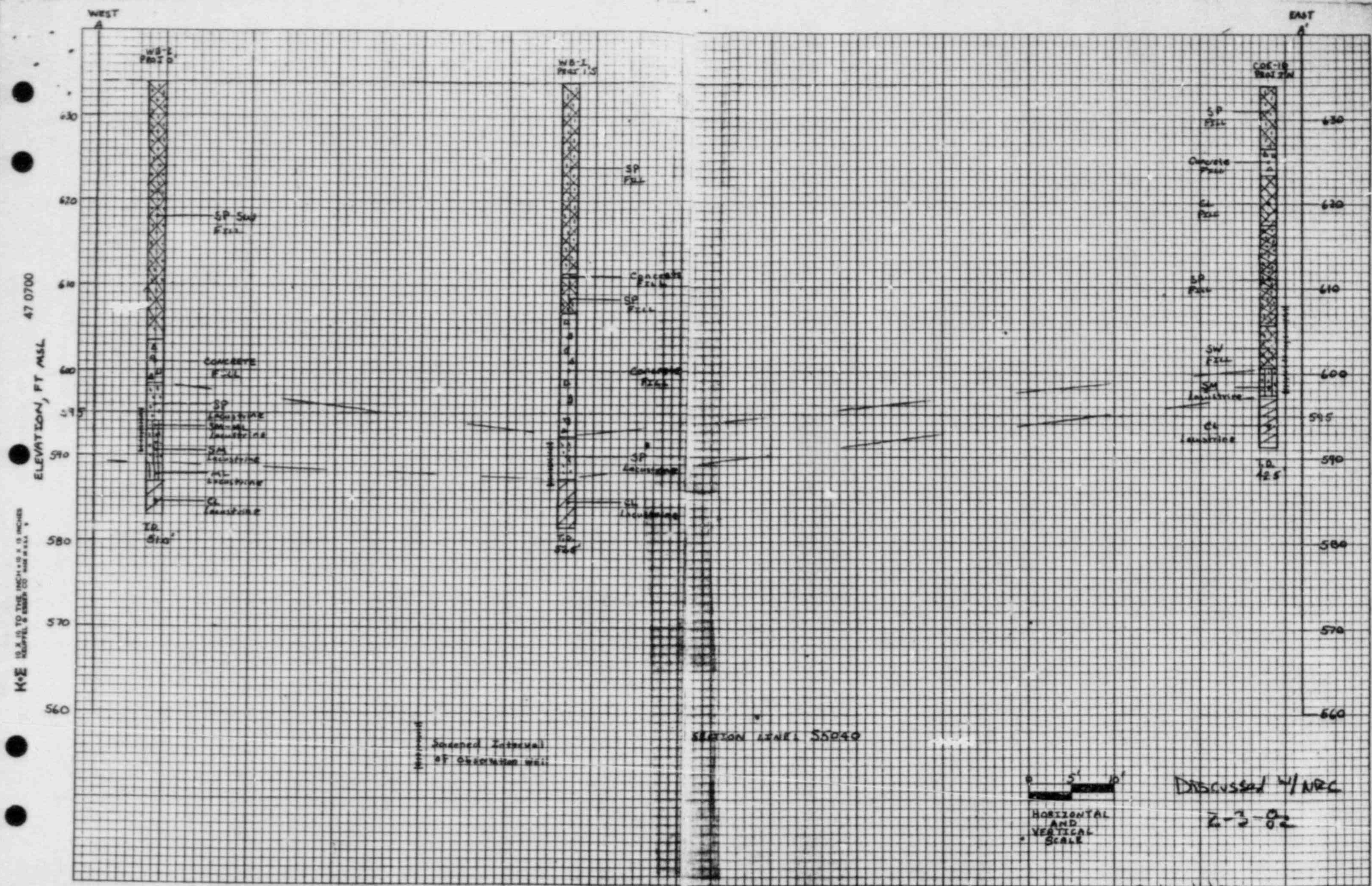
2/23/82
Bethesda Meeting
w CPC

## WELL FAILURE MECHANISMS AND RESPONSES

| Event | 50.54F Reference | Repair Time |
|---|---|---|
| 1. Electrical Failure | 24.C | |
| A. Single Well (Wired in Parallel) | | Less than 1 day |
| B. Multiple Wells Due to power outage | | One day to initiate operation of back-up diesel power to interceptor wells. Operate until normal power can be restored. |
| 2. Failure of Timers/Pumps/ Check Valves | 47.6 | Less than 1 day, replacement parts on site. |
| 3. Header Pipe Break | 24.C | One day to attach flexible hose to each well affected and pump water to storm drains. In case of interceptor well header failure, initiate back-up wells (on separate header system). |
| 4. Well Screen Installation | 47.6 | Two days to acidize well. |
| 5. Complete Loss of Well | 24.C | Four days to replace one well using cable tool rig. One day if other drilling method used. |



593.38
WB-2
WB-1
COE-9A
Dry @ el 607.0
COE-10
598.89
UNIT 1
TRANSFORMERS
DIESEL
ENERATOR BUILDING
COE-11A
Dry @ el 602.48
592.65
PD-19
Dry @ el 602.60
PD-20A
PD-20B
Dry @ el 602.09
Dry @ el 597.38
PROXIMATE WIDTH AND
CATION OF INSPECTION
TOFF TRENCH
CONDENSATE
STORAGE TANKS
Dry @ el 605.72
CIRCULATING WATER
DISCHARGE STRUCTURES




Rec'd 2/3/82
from W. Paris

Copy given in deposition documents

J. Kane
Rec'd. 6/9/80
3/B8

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

IN REPLY REFER TO: WESGA

4 JUN '80

SUBJECT: Report on Review of Geotechnical Aspects of the Seismic Safety of Midland Nuclear Power Plant

District Engineer
U. S. Army Engineer District, Detroit
ATTN: NCEED-T/Mr. Neil Gehring
477 Michigan Avenue
Detroit, MI 48226

1. Inclosed is a Memorandum for Record dated 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) by P. F. Hadala (Incl 1). This memorandum is an interim report on our work under your IAO No. NCE-IA-80-047.

2. If you have any questions, please feel free to contact Dr. Hadala at FTS 542-3475.

FOR THE COMMANDER AND DIRECTOR:

1 Incl
as

F. R. BROWN
Engineer
Technical Director

CF w/incl:
Mr. Jim Simpson, NCDED-G
Dr. Lyman Heller, NRC
Mr. Joe Kane, NRC

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

p. 13 of Detroit Report

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.

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.

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.

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.

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. 10 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:

Pg. 11 of Detroit Rpt

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

B. In the event the water levels in observation wells near Category I structures or pipelines supported on plant fill exceed those for normal operating conditions as defined by your 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 Report

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

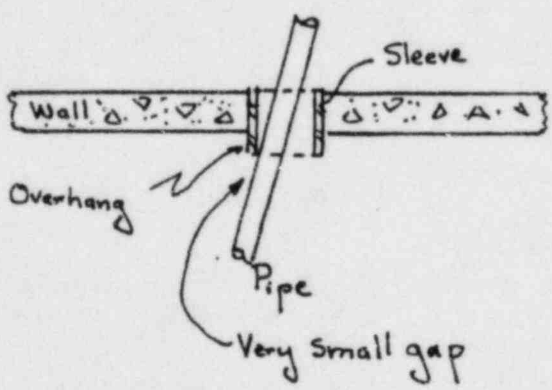
F(b). Provide after completion of the new pile foundation, in accordance with commitment No. 6, item 125, Consumers Power Company memorandum dated 13 March 1980, the results of measurements of vertical applied load and absolute pile head vertical deformation which will be made when the structural load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

<u>Comments regarding rattlespace at Category I pipe penetrations of structure walls</u>

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

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.

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:

Detroit Report pg. 3 & 9

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>

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:

Detroit Report pg. 14

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?

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)

J. Tabulate for each old analysis and each reanalysis, the foundation parameters ($V_s$, $\nu$ and $\rho$) used and the equivalent spring and damping constants derived therefrom so the reviewer can gain an appreciation of the extent of parametric variation performed.

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

<u>Category I retaining wall near the southeast of the service water pump structure</u>

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.

<u>Status of review of geotechnical earthquake considerations</u>

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.

6 Incl
as

Paul F. Hadala

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

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

Wanzeck

Feb 27, 1980

| | |
|---|---|
| G. Keeley | CPCo |
| J.O WANZECK | BECHTEL - GEOTECH |
| Darl S Hood | NRR/DPM/ |
| Pao HUANG | NSWC |
| D.S. Rist | Bechtel |
| W.C. Paris, Jr | Bechtel - Geotech |
| KARL SHEDNER | BECHTEL - ENGR |
| Harris H. Burke | Bechtel - SF |
| WALTER R. FERRIS | Bechtel - SF |
| Joseph D. Kane | NRC/DSS, Geotech. Engr. |
| B. DHAR | BECHTEL - ENGG. |
| J.V. Rotz | " Civil-Structural |
| Lyman Heller | NRC - Geotech |
| Wm C Otto | Corps Engrs Detroit |
| James W. Simpson | " " NCD Chief Engr |
| PAUL F. HADALA | Corps of Engineers, WES |
| JOHN F. NORTON | Corps of Eng North Cen Div. Chicago IL |
| John Brammer | ETEC |
| A.J. CAPPUCCI | NRC/DSS, MEB |
| W. PAUL CHEN | ETEC |
| Neal A. Gehring | Corps of Engineers, Detroit Dist. |
| JOHN GRUNDSTROM | " " |
| Ronald Erickson | " " |
| THIRU R THIRUVENGADAM | CONSUMERS POWER |
| L. H. Curtis | Bechtel |
| JOHN RUTGERS | BECHTEL |
| D.E. HORN | CONSUMERS POWER P.E.&C - QA |
| William Lawhead | C of E, Detroit |
| Gil Keeley | Consumers Power Proj. |
| TC Cooke | " " " |

Start 8 AM

Guides Hydr / Mach / Str / Geot. - less than ½ day

10AM

5.5 @ 24 Km 0.12 to 0.19 g

write communication via telecopier

Joe Kane is tech monitor for COE work

two phase ① are you satisfied with the fix

② are you satisfied overall???

Encl 1

Meeting 2/27/80 NRC / CPCo / Bechtel
Bechtel Consultants / US Corp of Eng
E TEC / US Navy Weapons Center

| | |
|---|---|
| B.C. McConnel | Bechtel - Ann Arbor |
| Ray Gonzales | NRC |
| Gene Gallagher | NRC RIII I:E |
| Frank Rinaldi | NRC NRR/DSS/SEB |
| John P. Matra Jr | NSWC |

On 2/28/80
Drs Ralph B. Peck, A.J. Hendron Jr, Tom Davisson & Chuck Gould, Consultants to Bechtel were also present

# MEETING WITH NRC ON MIDLAND PLANT FILL STATUS AND RESOLUTION

February 27 & 28, 1980
Midland Site

| | | |
|---|---|---|
| 1.0 | INTRODUCTION | G. Keeley (CP) |
| 2.0 | PRESENT STATUS OF SITE INVESTIGATIONS | T. Cooke (CP) |

2.1 Meetings with Consultants and Options Discussed (Historical)

2.2 Investigative Program

- A. Boring Program
- B. Test Pits
- C. Crack Monitoring and Strain Gauges
- D. Utilities

2.3 Settlement

- A. Area Noted
- B. Preload
- C. Instrumentation

| | | |
|---|---|---|
| 3.0 | WORK ACTIVITY UPDATE | J. Wanzeck |

3.1 Summary of work activities and settlement surveys for all Category I structures and facilities founded partially or totally on fill

| | | |
|---|---|---|
| 4.0 | REMEDIAL WORK IN PROGRESS OR PLANNED (Q4, 12, 27, 31, 33 & 35) | S. Afifi (Ann Arbor Office Bechtel) |

- 4.1 Diesel Generator Structures
- 4.2 Service Water Pump Structures
- 4.3 Tank Farm
- 4.4 Diesel Oil Tanks
- 4.5 Underground Facilities
- 4.6 Auxiliary Building and FW Isolation Valve Pits
- 4.7 Liquefaction Potential

| | | |
|---|---|---|
| 5.0 | EVALUATION OF PIPING (Q16, 17, 18, 19 & 20) | D. Riat |
| 6.0 | DEWATERING (Q24) | B. Paris |
| 7.0 | ANALYTICAL INVESTIGATION | B. Dhar |

- 7.1 Structural Investigation (Q14, 26, 28, 29, 30 & 34)
- 7.2 Seismic Analysis (Q25) Chuck McConnel
- 7.3 Structural Adequacy with Respect to PSAR, FSAR, etc.

| | | |
|---|---|---|
| 8.0 | SITE TOUR | All |
| 9.0 | CONSULTANTS SUMMARY (there in the am) | Peck/Hendron/Gould/Davisson |
| 10.0 | DISCUSSION | All |

Incl 2

# ATTENDEES

**Consumers Power**

G. S. Keeley
T. C. Cooke
T. Thiruvengadam

(1) Dec - order stopped all work on subcontracts for fixes - on hold

(2) Answers to questions 24 → 35

**Bechtel**

Harris Burke
Sherif Afifi
Don Riat
Bimal Dhar
Bill Paris
Julius Rotc
Jim Wanzeck
Karl Wiedner
John Rutgers
Lynn Curtis
Al Boos
Chuck McConnel

**Consultants**

R. B. Peck
A. J. Hendron, Jr.
C. H. Gould
H. T. Davisson

**NRC**

L. Heller
R. Jackson
J. Kane
Tony T. Cappucci → E-TEC
F. Rinaldi → NSWC
R. Gonzalis
F. Schauer
D. Hood — NRC Site Rep.
G. Gallagher
R. Cook

**US Corp Of Engineers**

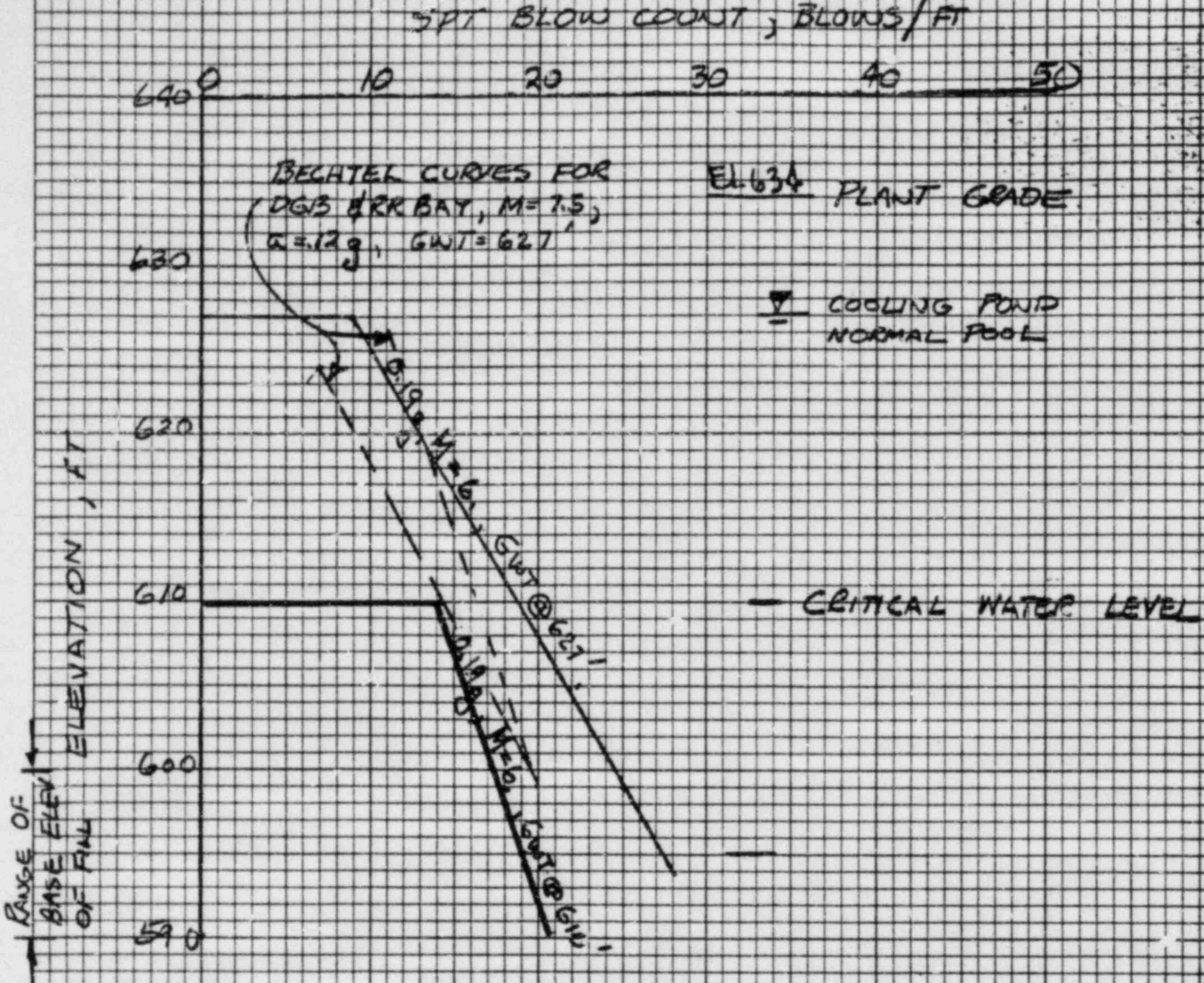
N. Gehring
J. Grundstrom
B. Otto
W. Lawhead
P. Hadala
J. Simpson

**E-TEC**

P. Chen
J. Brammer

**US Navy Weapons Center** ✓

P. Huany
J. Matra




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

Encl 3

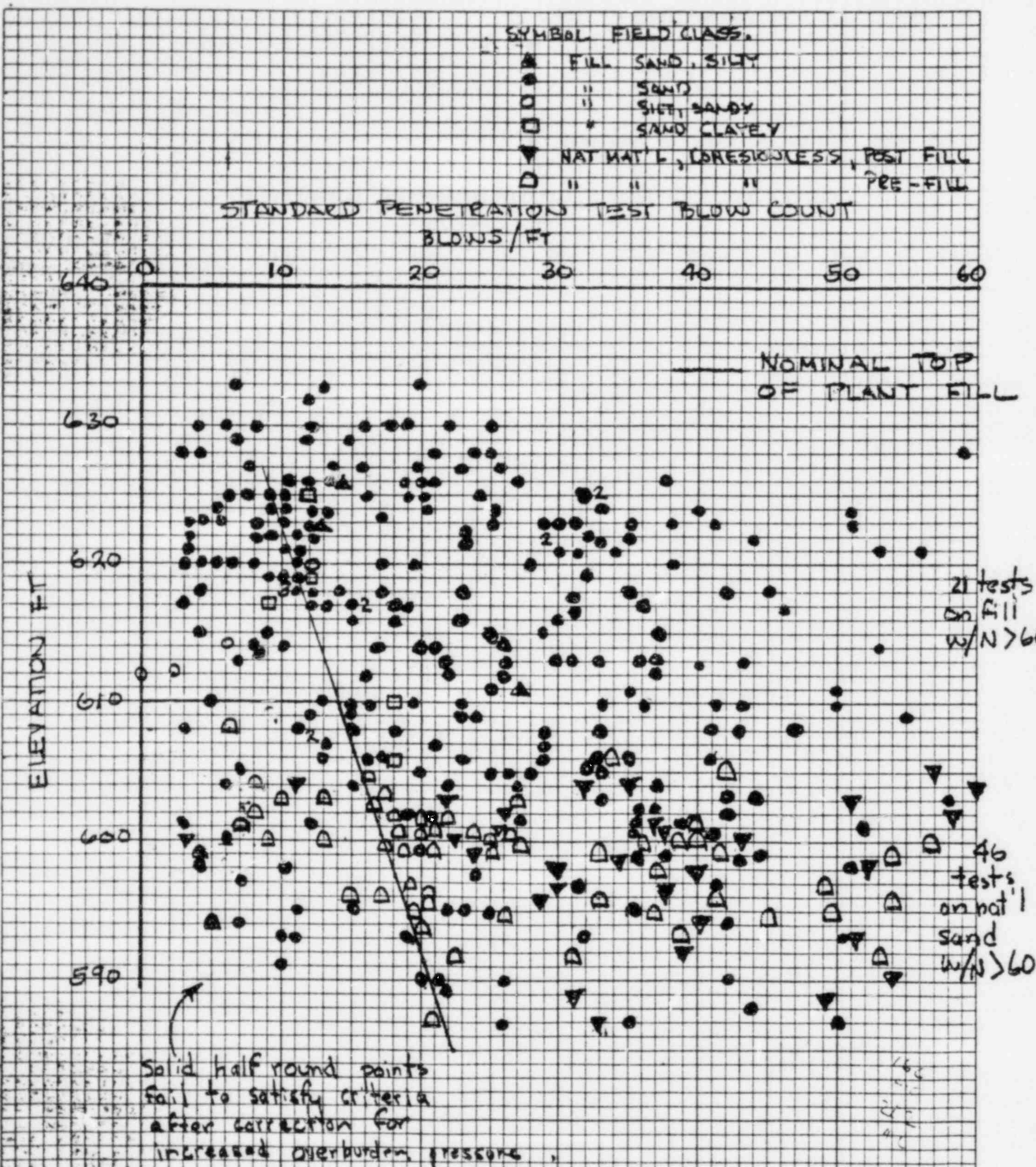



FIGURE BLOW COUNT VS ELEVATION FOR STD. PENETRATION TESTS IN SANDY SOILS IN THE PLANT FILL AREA



Incl 4

Encl 5

## Summary of "Low" Blow Counts in Cohesionless Soils Below Elev. 610

* 27 locations where liquefaction is considered a possibility (w/M=6 event & GWT @ El. 610)

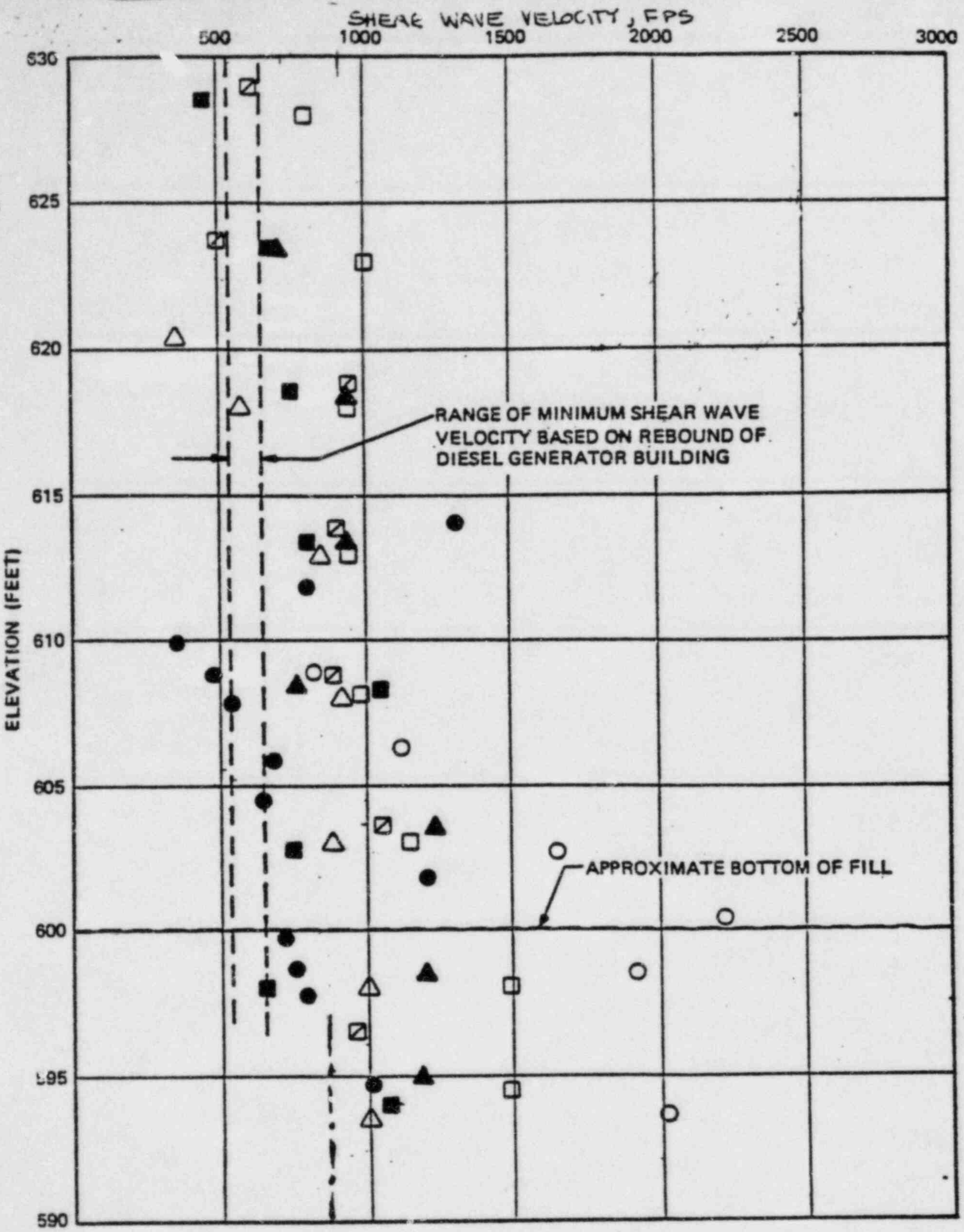
| Boring | Elev | N Value Blows/ft | Location | Cat. I | Fill or Nat'l | Remarks |
|---|---|---|---|---|---|---|
| * SW3 | 608 | 11 | Service Water Pump Storage | Yes | Fill | Pile support planned |
| * SW2 | 608 | 11 | Service Water Pump Storage | Y | F | Pile support planned |
| * DG18 | 609 | 12 | Under Diesel Gen. Bldg. | Y | F | |
| * DG18 | 607 | 13 | Under Diesel Gen. Bldg. | Y | F | |
| * AX13 | 597 | 7 | N.E. of Unit 2 | No | F | |
| * AX13 | 591 | 10 | N.E. of Unit 2 | N | F | |
| * AX4 | 601 | 12 | Between Unit 2 & Turbine Bldg. | Y | F | Underpinning planned |
| * AX4 | 593 | 19 | Between Unit 2 & Turbine Bldg. | Y | F | Underpinning planned |
| * AX15 | 595 | 11 | Between Unit 1 & Turbine Bldg. | Y | F | Removal & repl w/conc |
| * AX15 | 593 | 11 | Between Unit 1 & Turbine Bldg. | Y | F | Removal & repl w/conc |
| * AX7 | 605 | 7 | Between Unit 1 & Turbine Bldg. | Y | F | Removal & repl w/conc |
| * AX7 | 594 | 7 | Between Unit 1 & Turbine Bldg. | Y | F | Removal & repl w/conc |
| * AX7 | 590 | 20 | Between Unit 1 & Turbine Bldg. | Y | F | Removal & repl w/conc |
| * AX5 | 601 | 3 | Between Unit 1 & Turbine Bldg. | Y | F | Removal & repl w/conc |
| * AX5 | 598 | 4 | Between Unit 1 & Turbine Bldg. | Y | F | Removal & repl w/conc |
| * AX11 | 606 | 13 | Under Unit 1 Valve Pit | Y | F | Underpinning planned |
| * AX11 | 600 | 6 | Under Unit 1 Valve Pit | Y | F | Underpinning planned |
| * AX1 | 593 | 10 | Under Unit 1 Valve Pit | Y | F | Underpinning planned |

Summary of "Low" Blow Counts in Cohesionless Soils Below Elev. 610 (Continued)

| Boring | Elev | N Value Blows/ft | Location | Cat. I | Fill or Nat'l | Remarks |
|---|---|---|---|---|---|---|
| * DG19 | 608 | 3 | Under Diesel Gen. Bldg. | Y | Fill | |
| * DG13 | 604 | 6 | Under Diesel Gen. Bldg. | Y | F | |
| * DG7 | 598 | 10 | E. of Diesel Gen. Bldg. | N | F | |
| * DG7 | 595 | 15 | E. of Diesel Gen. Bldg. | N | F | |
| * DG5 | 604 | 15 | S. of Diesel Gen. Bldg. | N | F | |
| * SW6 | 600 | 3 | Service Water Pump Storage | Y | N-B (Natural) | B means blow count in natural mat'l before 35' of fill was placed. Pile support planned |
| D42 | 587 | 21 | Under Diesel Gen. Bldg. | Y | N-A | Ok when corrected |
| 5 | 608 | 6 | N. Part of Turbine Bldg. | N | N-B | Ok when corrected |
| 5 | 604 | 7 | N. Part of Turbine Bldg. | N | N-B | Ok when corrected |
| * D21 | 594 | 5 | E. Side of Turbine Bldg. | N | N-B | |
| 17 | 603 | 13 | S. Part of Turbine Bldg. | N | N-B | Ok when corrected |
| * CT1 | 604 | 11 | N. Condensate Storage Tank | Y | N-A | |
| 355 | 601 | 7 | NW of Intake Storage | N | N-B | Ok when corrected |
| DG28 | 600 | 9 | Between Diesel Gen. & Turbine Bldgs, | Y | N-B | Ok when corrected |
| 22 | 603 | 10 | N. of Borated Water Storage | N | N-B | Ok when corrected |
| 21 | 602 | 8 | NW of Borated Water Storage | N | N-B | Ok when corrected |

Summary of "Low" Blow Counts in Cohesionless Soils Below Elev. 610 (Concluded)

| Boring | Elev | N Value Blows/ft | Location | Cat. I | Fill or Nat'l | Remarks |
|---|---|---|---|---|---|---|
| * 2 | 599 | 4 | N. Part of Auxiliary Bldg. | Y | N-B | |
| 2 | 596 | 15 | N. Part of Auxiliary Bldg. | Y | N-B | Ok when corrected |
| 10 | 600 | 13 | N. Part of Auxiliary Bldg. | Y | N-B | Ok when corrected |
| 10 | 596 | 17 | N. Part of Auxiliary Bldg. | Y | N-B | Ok when corrected |




LEGEND:

- ☒ CONDENSATE TANKS AREA
- □ ■ BORATED WATER STORAGE TANKS AREA
- ○ ● SERVICE WATER PUMP STRUCTURE
- △ ▲ DIESEL GENERATOR BUILDING
- —·· WESTON SURVEY (FSAR 2.5.4.7.2) PRE-CONSTRUCTION

| | |
|---|---|
| BECHTEL<br>ANN ARBOR | |
| MIDLAND POWER PLANT | |
| SHEAR WAVE VELOCITY PROFILE<br>PLANT AREA FILL | |
| JOB NO.<br>7220 | DRAWING NO.<br>FIGURE 35-2 |

D-06