Attachment 1

GEOTECHNICAL ENGINEERS INC.
1017 MAIN STREET · WINCHESTER · MASSACHUSETTS 01890 · (617) 729-1625

PRINCIPALS
DANIEL P. LA GATTA
STEVE J. POULOS
RONALD C. HIRSCHFELD
RICHARD F. MURDOCK
GONZALO CASTRO

May 23, 1983
Project 81907
File 11.0
Ref: 81907-23

Mr. Joseph Kane
NRR Project Officer
U. S. Nuclear Regulatory Commission
Division of Engineering, M/S P-214
Washington, D.C. 20555

Subject: Site Visit of May 10-13, 1983
Midland Plant Underpinning Contract

Dear Mr. Kane:

Enclosed are four copies of our report on the subject site visit as it related to the pier load test.

Other topics were discussed at the meeting, such as: Cracks in the Service Water Pump Structure, heave over the freeze wall, need to stop laying back the soil along the underpinning drifts, method of ringing the wedges to maintain load, and settlements due to underpinning to date.

These topics are not covered in this report as they will be coverd in the minutes of the meeting by the Applicant.

Sincerely yours,

GEOTECHNICAL ENGINEERS INC.

Steve J. Poulos
Principal

SJP:ms
Encl.
cc: Mr. Hari Singh
Mr. Reuben Samuels

REPORT OF SITE VISIT
May 10-13, 1983

MIDLAND PLANT UNDERPINNING

Geotechnical Engineers Inc.

Ref: 81907-23
May 20, 1983

A site visit was made by Mr. Joseph Kane, Dr. Ross Landsman, Mr. Hari Singh, and Dr. Steve Poulos to observe the bearing soil, the underpinning operations and the test pier set-up for Pier W11. In addition, discussions were held about the Applicant's interpretation of the pier load test results.

The purpose of this report is to record our conclusions relative to the pier load test results and to report the results of static cone penetration tests made by the above party in Pier E8.

PIER LOAD TEST RESULTS

The purpose of the pier load test was to determine the compressibility of the bearing layer as indicated by the soil modulus, E_s, and by the rate of secondary compression. During design it was assumed by the Applicant that E_s = 3000 ksf and that the coefficient of secondary compression, C_α = 0.0005 to 0.001 (strain) per log cycle of time.

A concrete pier shown in Fig. 1 was tested to 600 kips. The sides of the pier were greased to reduce skin friction. However, the data indicated that substantial side friction developed. Perhaps only two thirds of the 600-kip load reached the bearing stratum. As a result, considerable uncertainty remains about the equivalent modulus of elasticity of the soil, E_s, based on the data.

The friction probably developed due to the shape of the sides of the pier shaft, which varied $\pm$0.5 in. over its height, as shown in Fig. 2.

Modulus of Hard Clay - The pier compression data and the Carlson stress meter data permit independent estimation of the side friction. Figs. 3 and 4 show the load at the top vs tip settlement, the estimated skin friction, and the estimated tip load vs tip settlement based on the analyses provided during our site visit. From Figs. 3 and 4, the following modulus values are obtained at various stress levels:

Basis	Bearing Stress ksf	Equivalent Modulus, E_s, ksf		
		Surface	At Depth (Mindlin)	Average
Carlson	8.8	2800	1540	2170
Meters	13.0	2680	1470	2080
(Fig. 3)				
Pier	8.8	1620	890	1260
Compression	13.0	1600	880	1240
(Fig. 4)				

AVG. VALUE AT 13 KSF = 1660 KSF

The above values for the surface case are computed from the equation

$$E_s = \frac{q}{(\delta/K\ B)} \qquad \text{Eq. (1)}$$

where:
- q = stress at tip
- δ = settlement at tip
- B = width of pier (2.8 ft)
- $K = 0.75 = 1.0 \times (1 - \mu^2)$
- $\mu = 0.5$ (undrained case)

The Mindlin solution for stresses within an elastic mass indicates that the above modulus should be multiplied by 0.55 to account for the depth effect. However, this solution overcorrects for real soils because it is based on the assumption that tensile stresses above the pier bottom restrain settlements. Real soils do not support tensile stresses. The average of the value of E_s at the surface and at depth is used in the table above in the absence of (1) a more rigorous analysis, (2) knowledge of the distribution of skin friction on the pier, and (3) knowledge of the properties of the fill at this location.

Secondary Compression of Hard Clay - Fig. 5, provided at the site visit, shows the apparent secondary compression in the test pier (W11) for a top load of 600 kips. The rate of secondary compression is impeded by skin friction. It is not known what the value would be in the absence of skin friction. However, the bearing pressure at the tip is considerably larger than the design pressures. Therefore, the rate of secondary settlement for the completed foundation can be expected to be smaller than shown for Pier W11.

The rate of secondary compression measured in all piers ranged over values that are greater than those anticipated in design. The lower end of the measured range is at the

upper end of the anticipated range at the design bearing pressures. For the completed foundation the group action of the piers will reduce the effect of skin friction on the rate of secondary compression. We would anticipate higher rates of secondary compression, perhaps by a factor of two, on average, than were used for design.

As was indicated by the Applicant during the site visit, it is possible to maintain the jack pressure longer than anticipated before transferring the load to the underpinning. In this way the long-term differential movements can be held to the design values.

The important parameter with respect to design is the differential settlement between the underpinned foundation and the auxiliary building. Thus, the actual secondary settlement in inches/log cycle of time must be estimated before the loads are permanently transferred to the underpinning. Extrapolation of such data will provide the required estimate of differential settlement.

STATIC CONE PENETROMETER READINGS - During the site visit Messrs. Kane, Landsman, Singh, and Poulos took measurements of the static cone penetration resistance of the hard clay about 8 ft above the bearing elevation in Pier E8. The readings at various points on the bottom of the pit were taken after digging down 2 in. to 5 in. below the horizontal surface that was exposed. The measurements ranged as follows:

Penetration Depth in.	Dial Reading on Proving Ring	Estimated Undrained Shear Strength ksf
3/4	35-80	2.2 to 5.0
1-1/2	150 to 220	2.4 to 3.6

We found that the greenish-gray clayey zones yielded the lower readings and the zones of tan-gray silt or silty clay yielded the higher readings. The silty zones felt harder to the touch. The plastic zones were at water contents a few percent above the plastic limit, which is consistent with previous data.

Although the silty zones give higher penetration resistance, it may be that undrained triaxial tests would show that silty samples have lower undrained strengths than clayey samples. Disturbance during sampling could have a larger effect on the silty layers than on the clay zones.

DISCUSSION - The undrained strength of the hard clay bearing stratum increases slowly with depth according to data provided by the Applicant. Therefore, the compressibility can be expected to decrease with depth.

For the above reason the modulus measured in the pier load test will be somewhat smaller than the modulus that would affect the settlement of a full size pier and the entire new foundation. Thus, the pier load test gives a low-side estimate of the actual modulus.

The rate of secondary compression of the test pier is too low because of skin friction. The same applies for all other piers. Also, the rate of secondary compression (inches/log cycle) for the entire foundation will be larger than measured for individual piers, because a greater depth of soil will be affected by the loads causing secondary compression. However, the secondary compression will be lower for the deeper layers for the same reason that the modulus increases with depth. The result probably is a net increase in the rate of secondary compression (inches/log cycle) with increasing size of bearing area.

The results of the static cone test results about 8 ft above the bearing elevation in Pier E8 indicate that the measured undrained shear strengths are about 3 to 3.5 ksf. Data presented by Bechtel at the site meeting indicate an average undrained shear strength of 6 to 6.5 ksf 8 ft above the bearing elevation. The reason for the difference is not known.

GEOTECHNICAL ENGINEERS INC.

Project 81907 Page

Client Date May 15, 82 By EF

Subject Checked By

Approved By

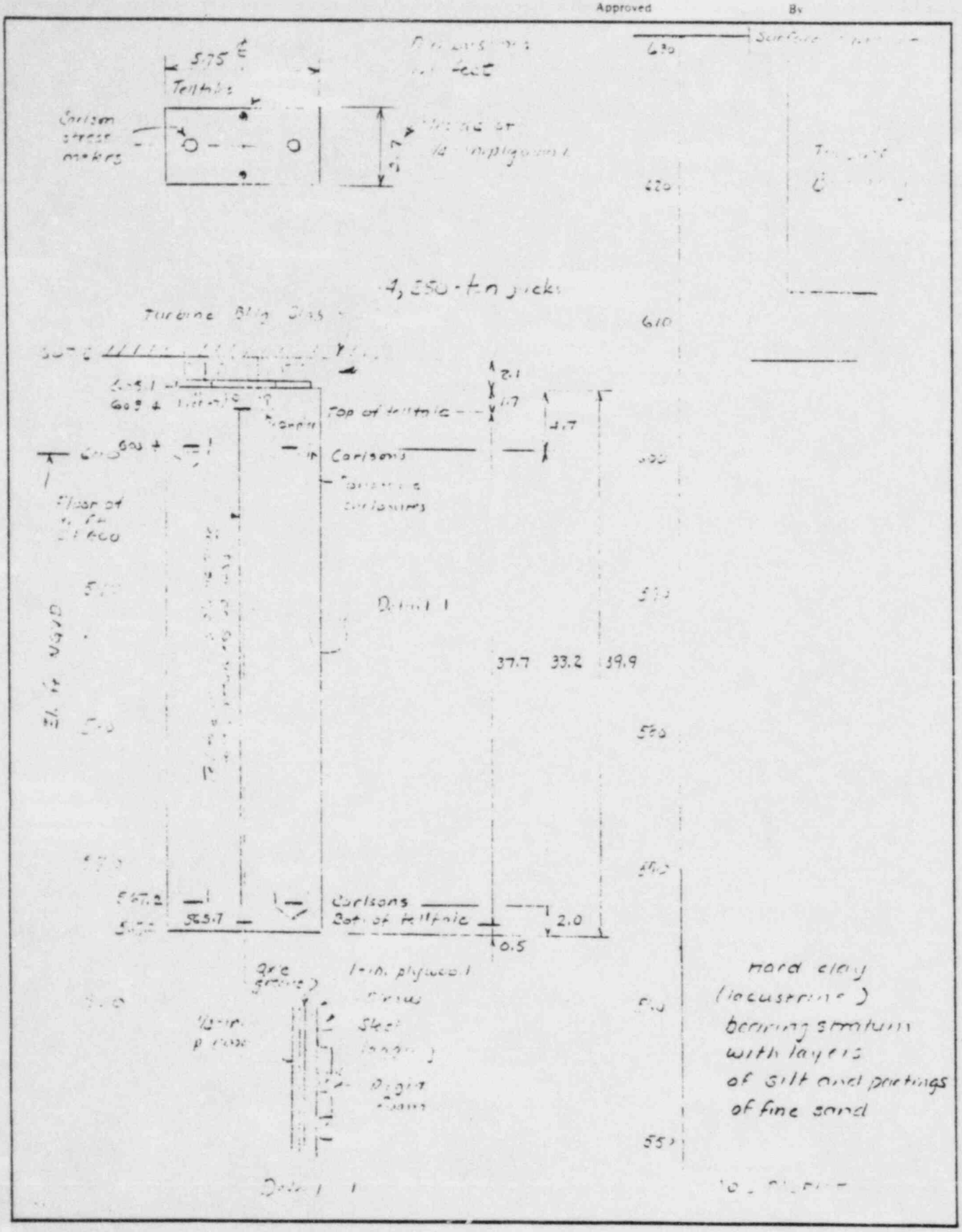


FIG. 1

Φ GEOTECHNICAL ENGINEERS INC.

Client		Project 81907	Page
		Date	By
Subject	Pier W11 Pier Length & Width	Checked	By
		Approved	By

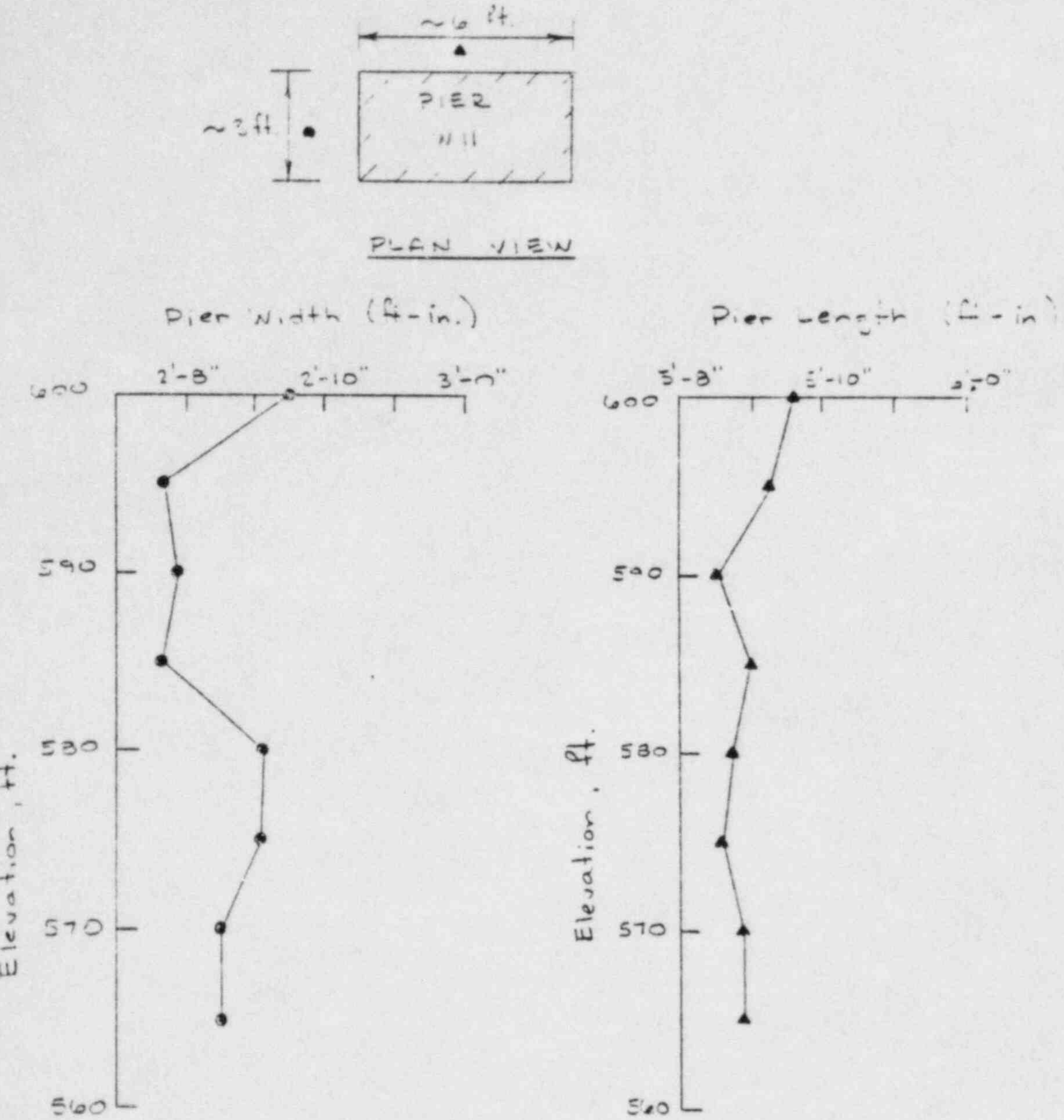
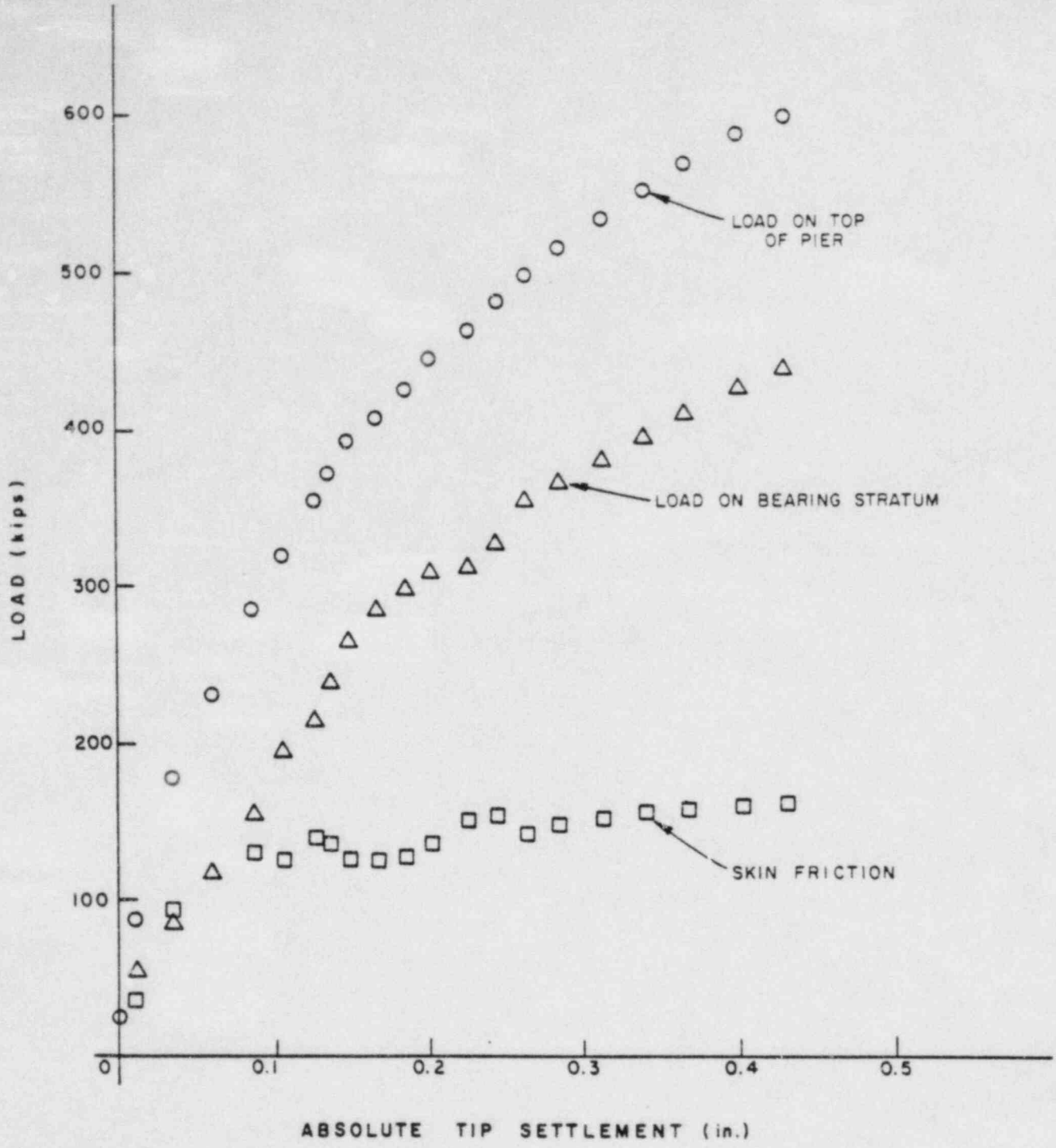


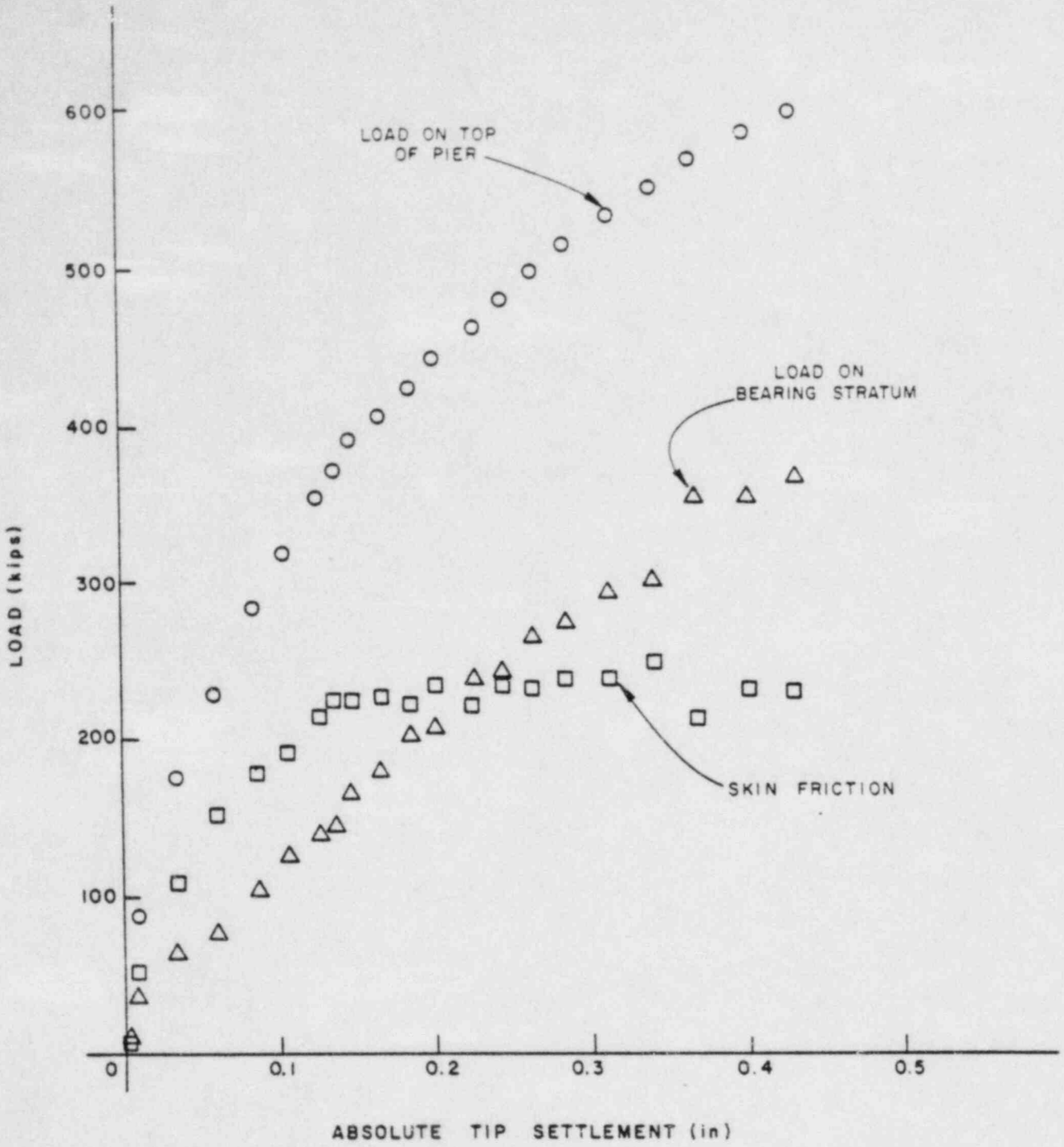
FIG 2



PLOT OF DATA FOR LOAD TEST ON PIER W11
BASED ON CARLSON STRESS METER READINGS

SJP
MAY 19, 1983

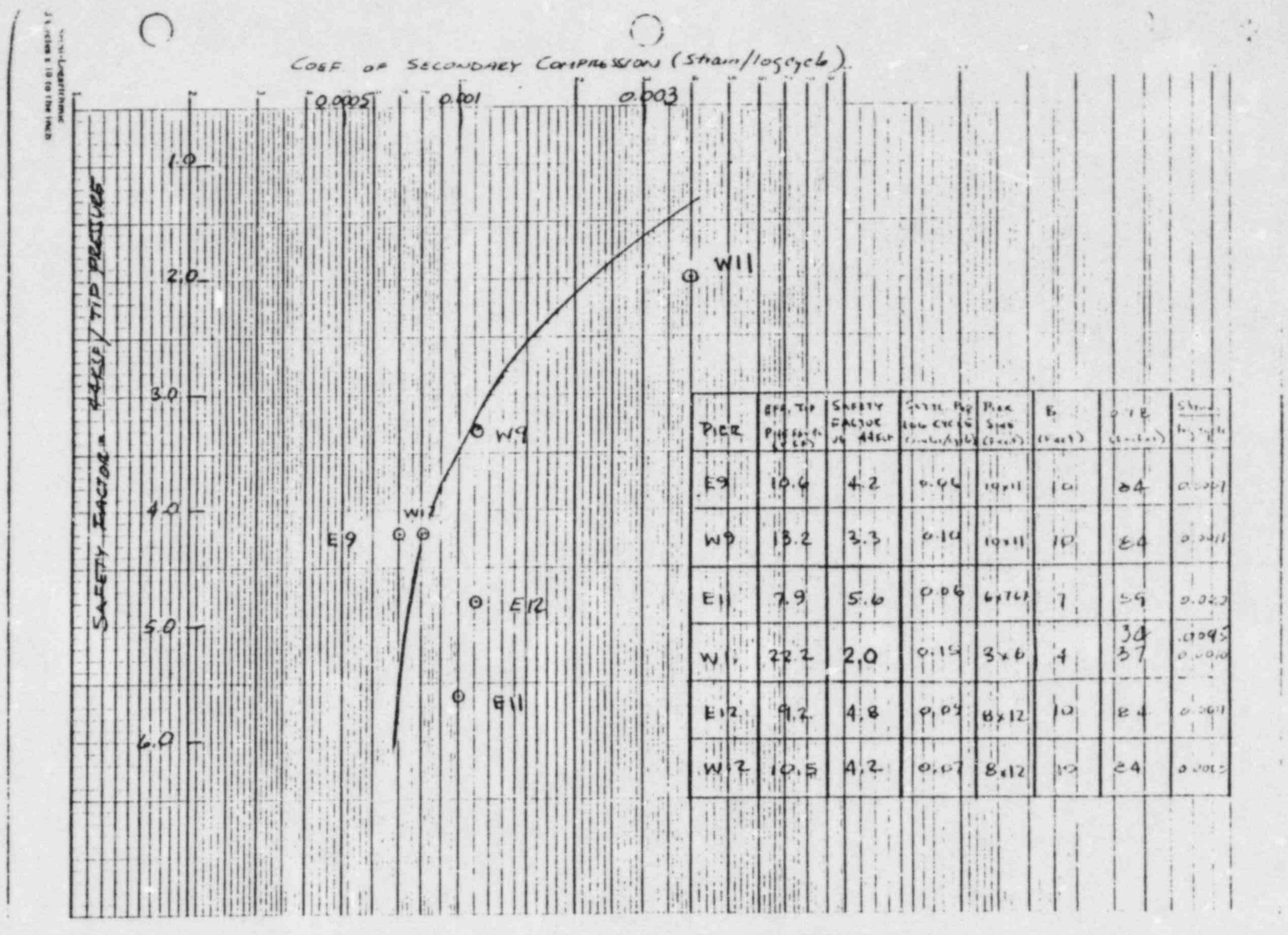
FIG. 3



PLOT OF DATA FOR LOAD TEST ON PIER W11
BASED ON TELLTALE DATA

SJP
MAY 19, 1983

FIG. 4



PIER	EFF. TIP PRESSURE (KSF)	SAFETY FACTOR [illegible]	[illegible] LOG CYCLE [illegible]	PIER SIZE (FEET)	B (FEET)	[illegible]	[illegible]
E9	10.6	4.2	0.44	19x11	10	84	[illegible]
W9	13.2	3.3	0.14	19x11	10	84	[illegible]
E11	7.9	5.6	0.06	6x7 [illegible]	7	59	[illegible]
W11	22.2	2.0	0.15	3x6	4	30 37	.0095 [illegible]
E12	9.2	4.8	0.09	8x12	10	84	[illegible]
W12	10.5	4.2	0.07	8x12	10	84	[illegible]

FIG. 5

Attachment 2

SITE VISIT REPORT
MIDLAND PLANT, UNITS 1 AND 2 (50-329/330)
DATE OF SITE VISIT: MAY 11 AND 12, 1983

REPORT PREPARED BY: Joseph D. Kane, GES, SGEB, DE, NRR

General

This memorandum has been prepared to document the discussions and actions that transpired at the May 11 and 12, 1983 visit with Consumers Power Co. A list of attendees is attached and includes representatives from CPC, Bechtel, Stone and Webster, Parsons-Brinkerhoff, Meuser-Rutledge, Region III, GEI, COE and GES.

The site visit was arranged to primarily view and discuss the pier loading at test Pier W11. Other items viewed during this site visit included inspection of underpinning work beneath the FIVPs and EPAs, both east and west sides, movement instrumentation within the Turbine Building and the widened crack on the roof of the SWPS. Brief discussions were held towards the closing of the meeting on May 12, 1983 on lateral soil movement between Piers W11 and W10; the proposed backfill compaction specifications; lay back slopes of underpinning excavations; proposed pipe tunnel approach to control tower pier CT-1, (UAT approach); drilling holes in advance of installing temporary construction dewatering at SWPS; proposed modification of pier bracing; and presentation of monitoring data recorded during underpinning operations.

The major portion of the site visit on May 11, 1983 was spent in the field visually observing the underpinning construction. An approximate 2-1/2 hour presentation was made by CPC at the end of the first day which summarized the Applicant's evaluation and conclusions on test loading of Pier W11. At the beginning of the second day the NRC staff and its consultants met separately from the Applicant to review the information provided and to discuss their evaluation and findings. A joint meeting with the applicant was then held on the afternoon of May 12. The following paragraphs summarize the two day site visit discussions.

Test Loading of Pier W11. A brief discussion of the field and project soils organizations was presented by the Applicant followed by a summary of pier construction and load transferring operations which had been completed to date. Four representatives from the Applicant presented their evaluation of the pier loading test data. A slide presentation was shown of the foundation materials encountered in the pier excavations to provide the staff with a good understanding of the natural layering of the brown silt lenses in the gray stiff lacustrine clays. With respect to the best estimate of the soil modulus for the foundation soils, the Applicant concluded that values in the range of 2500 ksf to 3000 ksf were reasonable from their evaluation of Pier W11 test data and considerably higher values (in excess of 4000 ksf) were reasonable from loading of Piers W12, E12 and W9. The conclusions of the

NRC staff and its consultants on the results of the pier load testing (soil modulus, Es, and the coefficient of secondary compression) are presented in Enclosure 1 (May 23, 1983 Report by Geotechnical Engineers, Inc.). The Staff position expressed at the May 12, 1983 meeting indicated that an equivalent soil modulus of elasticity equal to 1500 ksf was reasonable for adoption in design of the permanent underpinning wall and a differential settlement equal to 0.50 inch between the underpinned foundation and the auxiliary building was appropriate and reasonably conservative. The alternatives available to the Applicant resulting from the staff's position included the following:

a. Reexamine the design of the permanent underpinning wall for the impact of the lower design soil modulus and larger differential settlement. The Applicant expressed a willingness to consider this alternative and get back to the Staff in approximately one week. Their decision would be provided in a telephone conference to be coordinated with Region III. The staff briefly discussed the extent of future documentation that would be required if this alternative were chosen (identification of soil modulus and differential settlements addressed in structural reanlaysis, changes in soil spring stiffnesses and effects on structural design). The effect on the margin of safety against bearing capacity type failure using cone penetrometer data and the results of shear testing on undisturbed samples was also to be addressed in the light of a potentially less stiff foundation material.

b. Conduct a second-pier load test. This alternative would allow the Applicant to demonstrate that the soil modulus values are equal to or greater than the originally adopted soil modulus values by conducting a second load test where efforts to eliminate skin friction would be improved. In response to the Applicant's questions, the Staff and its consultants indicated that verification of lateral isolation along the sides of the pier shaft would be required, if this alternative were chosen, and an independent dual system for measuring pier tip settlement would be necessary. Installation of the Carlson concrete stress meters would not be necessary. Documentation similar to information provided for Pier W11 (max. test load, loading increments, etc.) would be required. The Staff indicated a need to reconsider the time interval for maintaining incremental loads and resolve this concern with the Applicant, if this alternative is eventually selected.

c. Conduct a plate load test. This alternative was proposed by the Applicant and was considered acceptable by the Staff and its consultants provided the applicable provisions of ASTM D-1194 were followed and a bearing plate with a minimum diameter of 18 inches were used. The Staff agreed that it would not be necessary to load the plate unto failure if the ultimate bearing capacity of 45 ksf were reached nor if the soil modulus exceeded 3000 ksf at the design bearing pressures. Authorization from Region III to proceed with the plate load test is necessary but NRR does not require further documentation prior to conducting the plate load test.

Crack Widening on Roof of SWPS. The site visit afforded the staff the opportunity to inspect a diagonal crack in the roof of the SWPS which had been reported to have recently widened in excess of 10 mils (max opening now at 35 mils). A report by the applicant that evaluates the crack widening had just recently been provided to Region III. GEI again indicated its concern for the accuracy now being used in measuring crack widths (calibrated microscope) and suggested a simple, more accurate and reliable measurement using a caliper and studs glued on either side of the cracks. Drilling of boreholes in preparation for the temporary dewatering at the SWPS were in progress on the day of the site visit but neither dewatering nor underpinning has begun at the SWPS.

Reported Soil Movement. A lateral movement of soil between Piers W11 (completed) and W10 (presently being excavated) was reported to have occurred the night of May 11, 1983. The approximately 2 to 3 foot deep zone was eventually filled with concrete before proceeding with excavation of Pier W10. A check on instrumentation in the immediate area of these piers did not indicate unusual movement.

Pipe Tunnel Approach to Pier CT-1. The Applicant questioned the Staff on its response to information previously furnished which covers the proposed modification for approaching Control Tower Pier CT-1 through the Pipe Tunnel (Drawings SK-C-856, SK-C-857 and SK-C-865). The Applicant stressed the importance of this work to its future construction schedule. The Staff agreed to respond with review comments within a week during the same conference call to be arranged which will provide the Applicant's decision on the alternatives to pier load testing.

Layback Slopes of Underpinning Excavations. Observations of construction operations during the site visit revealed an incorrect procedure being used by the underpinning contractor. Instead of bringing the lagging boards for drift excavations up to the bottom of the Turbine Building foundations, the upper boards were being omitted and the top of the excavations were being sloped inward. This procedure results in greater removal of foundation soil support than is necessary and creates the potential for larger structure movement. Following discussions with the Staff and its consultants, the Applicant agreed to avoid laying back any future excavation slopes and to provide bracing for the entire depth. Two short lengths of drift excavations near the KC piers were to be corrected with the placement of upper lagging boards and backpacking.

Proposed Specification for Backfilling. A preliminary copy of "Technical Specification for Backfill and Compaction of Soil for Feedwater Isolation Valve Pits and Auxiliary Building Underpinning Construction" was provided to the Staff.

The Staff agreed to review the specification for fulfillment of commitments made by the Applicant at previous audit reviews and to provide comments at a future date.

Temporary Dewatering at SWPS. The Bechtel Resident Geotechnical Engineer briefly described the preliminary results of borings completed at the SWPS that are in response to SSER No. 2 acceptance criteria requirements on groundwater control during underpinning. The NRC staff recommended that this information be provided to Region III with a brief engineering evaluation that includes an explanation for unusually low groundwater levels recorded in certain borings. Bechtel's RGE also covered the measures being taken to demonstrate the acceptability of probe jetting in the SWPS area. The staff agreed with Bechtel's procedures for checking the volume of soil material removed during jetting and recommended this procedure continue. The Staff also recommended that 5 or 6 additional jet probings be performed in the fill depth intervals where this material will be excavated and replaced and to monitor the time interval for advancing the holes for each 5 feet of penetration. The staff expressed the opinion that proper utilization of both the measurements of soil volume removed and the time for hole advancement should assure acceptable control to permit jet probing in areas where the plant fill soil will not be replaced. The final procedures to utilize jet probing in areas that will not have fill materials replaced require Region III approval.

Modification to Pier Bracing. In response to the Applicant's questioning on the acceptability of the proposed modification of pier bracing (Tie-Back System) the Staff indicated this was a matter to be discussed between the Applicant, Region III and Structural Engineering Section.

Monitoring During Underpinning. During the site visit the Staff was able to view several monitoring installations in the Turbine Building and to observe the control room for monitoring of Auxiliary Building movements. Representatives from Weiss, Jenny, Elstner and Associates, Inc. demonstrated their capability for calling for an immediate computer listing of temperature, absolute and relative deflection, extensometer, Carlson stress meter and strain gage data. No presentation was made nor were discussions held on the monitoring data that has been recorded to date.

19/83

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

SEP 8 1983

NOTE TO: Elinor G. Adensam, Chief
Licensing Branch No. 4
Division of Licensing

FROM: Darl Hood, Project Manager
Licensing Branch No. 4
Division of Licensing

SUBJECT: RECORD OF SEPTEMBER 2, 1983 TELEPHONE CALL ON SCHEDULING OF FOLLOWUP MEETING ON CONSTRUCTION COMPLETION DATES

The NRC's letter to the applicant dated August 9, 1983, noted that since the April 19-21, 1983, NRC staff visit to assess construction completion schedules for Midland, the applicant had requested a followup meeting to review the material previously provided and to provide additional information, and to discuss reconsideration of scheduling priorities between Units 1 and 2 in light of recent actions by Dow Chemical Company. The letter also noted that at Consumer's request, the staff would be scheduling this meeting in September, 1983.

On September 2, 1983, Mssrs. B. Hershe, N. Leech and others from Consumers Power Company called Darl Hood to advise that the week of October 24, 1983, would be the earliest time that Consumers would be prepared to discuss scheduling priorities between Units 1 and 2. They also noted that some months beyond this may be needed to establish a sufficient data base for scheduling projection purposes. The data base of interest is associated with predictions for the Construction Completion Program.

Accordingly, the followup meeting will not occur in September. Consumer realizes that this delay may impact our willingness to schedule other technical meetings which might be affected by Consumer's eventual schedule decision.

Darl Hood

Darl Hood, Project Manager
Licensing Branch No. 4
Division of Licensing

8310030542