Consumers Power Co Exhibit #3 D
Heller deposition — 6/9/81

Heller Dep #3

DISCUSSION OF THE APPLICANT'S POSITION

ON THE NEED FOR ADDITIONAL BORINGS

FOR

MIDLAND PLANT UNITS 1 AND 2

CONSUMERS POWER COMPANY

DOCKET NUMBERS 50-329 AND 50-330

Report Date: September 14, 1980

DISCUSSION OF THE APPLICANT'S POSITION ON THE NEED FOR ADDITIONAL BORINGS

After the discovery in August 1978 of unexpected settlement of the diesel generator building, borings were made throughout the site to investigate the condition of the plant fill and to provide information for remedial actions. This program resulted in a total of 265 borings.[1]

After the initial discovery of the settlement, 32 borings made in and around the diesel generator building indicated that the building could experience significant settlements that could not be estimated reliably based on laboratory test results. The applicant retained the services of Dr. R.B. Peck and Dr. A.J. Hendron Jr., two of the most knowledgeable and respected authorities in the field of soils engineering. The resumes of Doctors Peck and Hendron, who have consulted in numerous nuclear plant soils issues, are attached in Appendix A. It was recommended by the consultants, and agreed to by the applicant and its architect-engineer, to surcharge the building. This would consolidate the fill, accelerate the settlement, reduce the settlement that will occur after pipe connections are made, and permit a reliable upper limit estimate of settlement to be expected during the life of the plant.[2,3,4] After removal of the surcharge, six additional borings were made to conduct in-situ shear wave velocity measurements. These borings also included making standard penetration tests. Logs of these borings are included in Revision 9 to the Responses to NRC Requests Regarding Plant Fill.

Although the service water pump structure and the electrical penetration areas have exhibited negligible settlement, the borings have indicated that remedial action should be taken for these structures. The remedial action proposed is to underpin the cantilevered portion of the service water structure and the electrical penetration areas.[5] In connection with the design aspects of the underpinning, the services of Dr. M.T. Davisson were utilized. His resume is attached in Appendix A.

The NRC staff has requested that additional borings be made in 18 areas as outlined in the NRC letter of June 30, 1980 on this subject.[6] Discussions with the staff followed on July 31, 1980. The applicant believes that additional borings to justify the adequacy of the remedial action program are unnecessary in that borings, laboratory tests, data collected in connection with the surcharge program, and load testing provide sufficient information. Furthermore, it is estimated that two borings per area (which would be required in accordance with the staff's request) would cost a minimum of $400,000 not including applicant's overhead, project engineering cost, and possible damage to installed components and structures. Accordingly, the applicant's position is:

1. That the additional borings are not necessary, and

2. That the postulated benefits do not justify the cost.

Because of the disagreement with the NRC staff, a formal appeal for relief from the staff's request was made to NRC technical management. This discussion documents the applicant's presentation at the appeals meeting of August 29, 1980, and includes additional information pertinent to the NRC staff concerns. This document also is a partial summary of several discussions with the NRC staff and many formal submittals made during the last 2 years. Applicable references to more detailed information are provided.

A. DIESEL GENERATOR BUILDING

1. Settlement

As a result of the detailed studies of the settlement problems, it was decided to surcharge the diesel generator building with sand in order to consolidate the fill under the structure.

The surcharge was applied in three increments to a maximum height of 20 feet (approximately 2.2 ksf). The stresses prevailing during surcharging at all depths in the fill beneath the building exceeded those that will prevail while the structure is operational including those applied by future site dewatering.[2,3] Figure 1 shows the surcharge history and Figure 2 shows the stress distribution below the building during and after the surcharge. The cooling pond water level was raised to the maximum design level before surcharge reached its maximum level.[3] The groundwater table below the diesel building rose to approximately elevation 625, which is 3 feet below the base of the foundations as shown on Figures 27-5 through 27-49 in the response to NRC Question 27, Revision 6. The primary reason for requiring the pond level to be raised while the surcharge was being applied was to reduce capillary action and increase saturation levels closer to the planned groundwater elevation of 627. Pond water level was maintained at the maximum level throughout the period of surcharging. As can be seen from Figure 1, settlement occurred rapidly as the load was applied. When the surcharge reached its maximum level, the rate of settlement decreased rapidly. As anticipated, excess pore water pressures developed when the load was applied and dissipated rapidly, indicating rapid consolidation of the fill.[4]

Measurements made to date indicate that a small amount of rebound occurred during surcharge removal, and only small settlement took place since removal of the surcharge in August 1979. In addition, as expected during rebound, piezometers showed a slight drop in water level, indicating a negative pore water pressure which later stabilized with groundwater level.[7]

Primary settlement occurred rapidly and settlement measurements indicated secondary consolidation was occurring as verified by the straight line on the semi-log plot shown on Figure 3. This figure is typical of all the settlement curves shown in Figures 27-6 and 27-51 through 27-78 which exhibit a straight line settlement during secondary consolidation. This behavior has been recorded on many projects including the Chicago Auditorium where this straight line secondary behavior has been observed for 60 years. Settlement trends based on rates experienced while the surcharge was in place were extrapolated to predict maximum settlements expected to occur over the life of the plant. This prediction is based on the conservative assumption that surcharge loading conditions remain for the life of the structure. Settlement measurements made during the period between September 14, 1979, and June 12, 1980, show that, on the average, the building experienced less than 0.1 inch of settlement as shown on Figure 4.[4,7,8]

Secondary consolidation was also assessed using data obtained from four deep Borros anchors to provide greater accuracy than from conventional survey techniques.[9] The deep Borros anchors allowed movements to be measured by gages to an accuracy of 0.001 inch.[10] A typical set of measurements is shown on Figure 5. These secondary consolidation measurements, when extrapolated, indicate that settlements less than 1/2 inch would occur during the life of the plant under the design loading.

The technique of extrapolating from full scale test results is the most reliable method for predicting settlement. Normally at the start of a job, sampling and testing are utilized to predict settlements. In this particular situation, the surcharge program provided the opportunity for direct measurements and thereby eliminates the need for sampling and testing. It eliminates shortcomings of theories, sampling, and testing. Measurements in the laboratory are made to an accuracy of 0.001 inch; however, the laboratory sample is only 3/4 of an inch thick. The probable error in estimating the field settlement of a 28-foot layer over the 40-year plant life based on a single 3/4-inch laboratory test sample would be of the order of 1/2 inch due to measurement sensitivity alone, not including the effects of sampling disturbance and representativeness of the samples. Measurements in the field are also made to a 0.001-inch accuracy but the field test sample being measured is about 28 feet thick whereas the laboratory sample is only 3/4 of an inch thick. Thus, the full scale load test results involved far less error and will result in a more reliable prediction.[1,8]

It should also be noted that the approach which utilizes evidence other than the results of laboratory tests for the prediction of settlements has been used on previous nuclear power plant applications. At the Kewanee plant, initial settlement estimates based on laboratory test results predicted that settlement should be of the order of 15 inches. However, when the evidence of preconsolidation by glaciation was incorporated into the evaluation, predicted settlement was reduced to 1-1/2 inches. Measured settlement at the end of construction of the foundation was 1-1/2 inches. Another example was at Quanicassee where laboratory tests indicated high settlements. A preload program in conjunction with geological evidence resulted in a lower but more reliable prediction of settlement. The preloading in that case was accomplished by pumping down the groundwater and measuring the drop in piezometric pressure as well as deformations.[1,8]

The limitations inherent in sampling and testing have been recognized for many years. If sampling and testing are done, the predictions could, because of these limitations, be unrealistically large for certain soil conditions. Sampling and testing are not necessary because of the ability to make a more reliable and conservative estimate of settlement with a full scale surcharge program.[1,8]

Although the surcharge resolves the uncertainties regarding settlement predictions, it does not eliminate the potential for liquefaction. Various methods including chemical grouting to resolve this question were considered.[4] It was determined that the most reliable solution would be to permanently dewater the site fill. The dewatering design details are being determined based on data obtained from the temporary dewatering required for future underpinning activities. This will provide a direct measurement of the groundwater behavior in the fill. Furthermore, the temporary dewatering has the additional advantage of providing information on settlement due to dewatering which is much more accurate than predictions obtained from sampling and testing. Recharge data will be obtained when the temporary dewatering system is shut down.[9]

The approach used to estimate settlement at the diesel generator building relies on full scale measurements of settlement from surcharging and settlement measurements as a result of fill dewatering. These procedures provide a direct, reliable, and conservative means of predicting settlement; therefore, sampling and laboratory testing would not provide better data to refine predictions.[1]

The ability to directly measure over the plant lifetime the actual rate of settlement of any structure (a slow process) and compare the total differential settlement against the design basis for the building connections provides a positive and verifiable resolution of the safety question involved.

2. Bearing Capacity[1]

In addition to NRC concerns on settlement of the structure, there have been concerns raised on the bearing capacity safety factor.

The net ultimate bearing capacity is the soil pressure that can be supported at the base of the foundation in excess of that created at the same level by the weight of material above the base of the foundation. The net ultimate bearing capacity is defined below.

$$\text{Net Ultimate Bearing Capacity} = q_{d_{net}}$$
$$= CN_c + \gamma D_f(N_q - 1) + 1/2\,\gamma BN_\gamma$$

where

C = cohesion intercept

N_c, N_q, N_γ = bearing capacity factors

γ = effective soil unit weight

D_f = foundation embedment depth

B = foundation width

The factor of safety is equal to the net ultimate bearing capacity divided by the net applied pressure below the foundation. The minimum bearing capacity safety factor for the diesel generator building is well above the factor of safety of 3 given in FSAR Subsection 2.5.4.10.1.

Soil parameters selected for use in determining the net ultimate bearing capacity depend on the rate of load application and the rate of pore water pressure dissipation of the foundation soils. For a load being applied instantaneously, it must be assumed that no dissipation of pore water pressure would have occurred. Under the instantaneous loading condition, soil parameters should be selected based on undrained laboratory tests.

Where loads are applied gradually and/or maintained for a period of time to allow pore water pressures to dissipate, soil parameters should be selected based on drained laboratory strength tests or consolidated undrained laboratory strength tests with pore water pressure measurements.

The building loads for the diesel generator building structure were applied gradually and maintained over a period of more than 18 months; therefore, it is appropriate to evaluate bearing capacity based on drained conditions.

Consolidated undrained laboratory strength tests with pore water pressure measurements were conducted on samples of plant area fill having characteristics similar to those under the diesel generator building. To provide a conservative analysis, five samples with low dry unit weights in the range of 114 to 119 pounds/cubic foot were selected. Based on the results obtained from these samples, the effective angle of shearing resistance ($\bar{\phi}$) was found to be 29 degrees and the cohesion intercept ($\bar{C}$) was found to be 114 pounds/square foot. The drained angle of shearing resistance is known to be primarily a function of the plasticity characteristics of the soil and as the plasticity of the samples tested is within the range found beneath the diesel generator building, these tests are representative and testing of samples from below the diesel building would not result in significantly different design values. This laboratory test data is summarized on Table 1. The strength data is presented on a modified effective stress Mohr-Coulomb diagram in Figures 6 and 7. Total and effective strength data at failure shown on Figure 7 are comparable and indicate the pore water pressures existing in the samples tested were close to zero at failure. As shown on Figure 8, the net ultimate bearing capacity factor of safety is approximately 7 using $\bar{\phi}$ = 29 degrees and $\bar{C}$ = 114 psf and approximately 6 if the $\bar{C}$ term is assumed to be zero, assuming the water table will be lowered to below the foundation influence depth.

Under earthquake conditions, an additional loading equal to about 30 percent of the static loading will be applied. This load will be instantaneous and would occur under undrained soil conditions. Factors of safety for seismic conditions will be above acceptable limits.

B. SERVICE WATER STRUCTURE

After the discovery of the unexpected settlement at the diesel generator building, 13 borings were made within and around the portion of the service water structure supported on fill. These borings included standard penetration tests through the fill and terminated in the natural soils. Although there has been no unexpected settlement of the service water structure, the information obtained from the borings indicated that it would be appropriate to underpin the cantilever portion of the service water structure. This will be achieved by using piles driven into the natural soil. At a later date, nine borings were made to conduct shear wave velocity measurements. These borings also included standard penetration tests in the fill and were extended into the natural soils.[5,11]

During the initial site investigation by Dames and Moore and construction phases of the plant, there were borings made into the natural soils in the vicinity of the service water pump structure. Based on information obtained in the initial site investigation, borings made during construction, and borings and laboratory tests made after the discovery of the unexpected settlements in the diesel generator building, preliminary estimates of pile capacity for support of the cantilever portion of the service water structure were made. Based upon an estimated capacity on the order of 100 tons, it was determined that 16 piles would be required. Calculations will be submitted in the response to Question 41. To verify the initial estimate, a preproduction load test program will be conducted which will include loading a pile to yield in order to determine the pile working capacity. The pile will be top driven in a predrilled hole and will penetrate into natural soil. The load test will be conducted as close as possible to the location of the production piles. In production, the piles will be installed in the same manner as the test pile and will be tested by jacking against the building to 1.5 times the design load.[12,13]

Results of the various subsurface investigations conducted at the site also enabled an estimate to be made of the downdrag on the piles. Downdrag has been estimated on the basis of standard penetration tests and results of laboratory tests conducted on plant area fill soils throughout the site. Downdrag values will be verified by pullout testing during the preproduction stages. In this case, a pile will be driven in a predrilled hole in the same manner as the production piles. The pile will only penetrate through the fill and will not penetrate through the natural soil. The pile will be load tested in tension and the downdrag will be estimated on the basis of this test. Based on the above, downdrag will be factored into the final design.[12]

There is no need for additional borings as borings to date, preproduction testing, and testing to be performed during production will provide sufficient information.

C. AUXILIARY BUILDING

After the discovery of the unexpected settlement of the diesel generator building, 18 borings were made along the southern portion of the auxiliary building, both inside and outside of the electrical penetration and control tower areas. These borings penetrated the fill and were terminated in the natural soil. The borings included making standard penetration tests.[5]

During the initial site investigation by Dames and Moore, borings were made in this general area. Although there has been no unexpected settlement of the auxiliary building and electrical penetration areas, information obtained from the borings indicated that it would be appropriate to underpin the electrical penetration areas of this structure. This will be achieved using caissons bearing on the natural soils. This has been addressed in the response to NRC Question 12.[4,14,15]

The bearing capacity of the caissons to be installed in the electrical penetration areas was determined on the basis of laboratory test results conducted during the initial site investigation by Dames and Moore and has been factored into the preliminary specification for caisson construction. Bearing capacity calculations will be transmitted in the response to Question 42. During installation of caissons, each caisson will be load tested. A minimum of two caissons will be load tested to twice the working load and the remaining caissons will be load tested to 1.5 times the working load.[1,14]

Downdrag may also occur on the caissons. Estimates of downdrag were made on the basis of results of soils borings made beneath the electrical penetration area foundations. These estimates will be incorporated in the design. It should be noted, however, that downdrag around the caissons should be minimal because these caissons will be installed with friction breakers and bentonite slurry which are necessary to facilitate penetration of the caissons through the soil. Therefore, the friction around the caissons during service life will be minimal due to the presence of bentonite slurry. At least the last 4 feet of penetration into the natural soils will be hand dug without the use of friction breakers or casing.[14]

There is no need for additional borings because borings to date and testing to be performed during construction will provide sufficient information.

D. COOLING POND DIKE

The staff has requested that borings be taken in certain areas of the cooling pond dike.

The adequacy of the design and construction of the cooling pond dike is not a proper subject for consideration in the hearing on the NRC's December 6, 1979, Order Modifying the Midland Construction Permit. The scope of the hearing and the jurisdiction of the hearing board are limited and determined by the December 6, 1979, order. (See Public Service Company of Indiana, Incorporated, Marble Hill Nuclear Generating Station, Units I and II, ALAB-316, 3 NRC 167, 170, 1967.)

The December 6, 1979 Order clearly sets forth the subject matter for a hearing in the event one was requested. At Page 6, the Order provides:

> In the event a hearing is requested, the issue to be considered will be:
>
> (1) Whether the facts set forth in part two of this Order are correct; and
>
> (2) Whether this order should be sustained.

The first issue identified clearly provides no basis for an open-ended review of the design or construction of the cooling pond dike. No reference to the dike, a nonsafety-related and non-Q-listed structure, is made in Part Two of the Order.

Nor would the second issue provide such a basis. The basis upon which the order could be sustained is set forth in Part Four of the Order. The text of Part Four clearly indicates that the order was rendered pursuant to the Atomic Energy Act, not NEPA. Further, the Order is limited in scope to "remedial actions associated with the soil activities for safety related structures and systems founded in and on plant fill." Hence, the purview of the hearing is, by the direct terms of the Order, limited to a Safety Review of safety-related structures and systems. As pointed out above, the dike is not Q-listed, is not safety-related, and hence is outside of the scope of the soils hearings.

Although this is an inappropriate subject for NRC consideration in this hearing, the following information indicates why the dikes were adequately constructed.

Heavy equipment was used to construct the dike, whereas in the confined areas of the plant small hand-held equipment was utilized in many excavated areas. Prior to dike construction, the area was stripped of all soil which contained organics and deleterious materials. The area was excavated to an acceptable firm foundation for an inspection trench and an impervious cutoff. The excavation extended to a minimum of 8 feet below original ground level and a minimum of 2 feet into undisturbed materials of the impervious cutoff.[8]

After completion of the excavation, the subcontractor was required to request an inspection by the contractor's field engineers.

The clay embankment fill material was then placed in lift thicknesses not to exceed 12 inches and compacted with four passes of a 50-ton rubber-tired roller or equivalent compactive effort. Other equipment used was qualified on test pads using the proper materials and roller passes to the above specification. Other material sections of the dike were also placed utilizing methods described above. Care was employed to ensure material separation between zones of the embankment to prevent material contamination. If, for example, the sand zone was to be crossed by equipment, the area would be marked and the contaminated material would be removed and replaced with approved sand.[8,11]

Inspections were performed by the fulltime subcontractor's inspector for lift thickness, proper material, roller passes, and moisture conditioning.[8] The inspector would call for field density tests after approximately every 500 cubic yards were placed to verify that proper placement was accomplished.[16] Random over-inspections were conducted by a representative of the applicant during normal placement.

After completion of the dikes, several methods of monitoring the dikes were implemented. Twenty-four settlement monuments were placed around the dike. All readings show little or no settlement except for three monuments, which are located at the southeast corner of the dikes. These monuments show approximately 1-7/8 inches of initial settlement, which took place before pond fill. Since June 6, 1978, only 0.010 inch of settlement has been recorded.[1,17]

Four holes were drilled in the dike to install power poles. These holes extended approximately from elevation 632 to elevation 623 which was the approximate water elevation at that time. Visual inspection of these holes revealed firm, well compacted material, which is documented in inspection reports by the contractor's geotechnical personnel and describes the material in these holes as firm clay free of any standing water. In addition, penetrometer readings ranged from 1.8 to 2.7 tons/square foot. In a boring taken for this activity, blow counts were taken and show that the clay is stiff. (Blow counts ranged from 11 to 41.)

Prior to cooling pond fill, piezometers were installed in two locations. These were at the northeast dike and the east dike at depths to 67 feet. At each location there are ten piezometers starting at the pond side of the dike and extending to the river flood plain on the outside of the dike. Piezometers in the dike show the sand drain is performing as expected. Standard penetration tests in the fill at these locations show blow counts between 10 and 60, with two exceptions at approximately 70, and two exceptions near the surface at 3 and 7. Logs of these borings will be provided in the response to Question 46.

There are 19 groundwater monitoring wells around the dikes, extending to various depths from 32 feet to 234 feet. These are used to monitor the elevation and quality of the groundwater. As expected, water level in the monitoring wells is fluctuating with groundwater level changes.

Since completion of the pond fill there have been two inspection walkdowns around the dike by the contractor's geotechnical personnel accompanied by the applicant. No significant areas of concern have been identified.

This supports the conclusion that the dike is performing as intended.

The soils consultants have advised against making additional borings in the dike now that the pond has been filled, because of possible damage to the embankment due to the drilling operation.[3]

E. RETAINING WALL

The retaining walls adjacent to the service water pump structure (Seismic Category I) and circulating water pump structure (non-Seismic Category I) are both founded on natural soil and on backfill material. A construction joint separates sections of the walls that are on natural soil (except for a short distance which was excavated and backfilled during the construction of the service water pump structure) from the sections on backfill.

After discovery of the unexpected settlement of the diesel generator building, four borings were made near the retaining walls. The borings penetrated the fill and were terminated in the natural soil. During construction phases of the plant, there were borings made into the natural soil in the vicinity of the walls.[11]

Borings made adjacent to the retaining walls show that: (1) granular fill was placed and compacted behind the walls; (2) the outer walls are founded on stiff to very stiff clay fill; (3) the inner walls are founded on natural dense sands, and hard clays and silts that also underlie the fill supporting the outer walls.

The soil parameters used in the original design are compared in the following table with the values derived from the boring records and laboratory tests of the soil samples taken to date throughout the site.

	Design Values	Allowable Values from Boring and Laboratory Tests
A. Natural soil		
Cohesion	2.0 ksf	4.0 ksf
Bearing for static condition	7.25 ksf	12.9 ksf
Bearing for seismic condition	9.63 ksf	19.35 ksf
B. Backfill Soil		
Angle of internal friction	20°	35°
Bearing for static condition	3.34 ksf	3.3 ksf
Bearing for seismic condition	4.25 ksf	5.0 ksf

The design values are within the parameters derived from the borings and laboratory tests and, therefore, the design is conservative.

The factors of safety of the retaining wall against sliding and overturning, using the design parameters, are within the requirements given in FSAR Subsection 3.8.6.3.4. Slope stability evaluation based on borings to date show an adequate factor of safety.

The measured total settlement and differential settlement are each less than 1/4 inch from September 1978 to July 1980.[1,18]

Therefore, additional borings are not required in this area because available borings and settlement data provide information sufficient for evaluation of the adequacy of the walls.

REFERENCES

1. NRC Meeting, 8/29/80, Midland, Michigan
2. Responses to NRC Requests Regarding Plant Fill, Volume 3, Tab 7, letter from A.J. Hendron to S.S. Afifi, 10/23/78
3. Responses to NRC Requests Regarding Plant Fill, Volume 3, Tab 12, Bechtel Meeting Notes No. 882, 11/7/78
4. Responses to NRC Requests Regarding Plant Fill, Volume 4, Tab 75, letter from R.B. Peck to S.S. Afifi, 7/23/79
5. Responses to NRC Requests Regarding Plant Fill, Question 9
6. NRC letter to Consumers Power Company, Docket No. 50-329/330, 7/30/80; Table 37-1, Item 3
7. Responses to NRC Requests Regarding Plant Fill, Question 27
8. NRC Meeting, 7/31/80, Washington, D.C.
9. Responses to NRC Requests Regarding Plant Fill, Volume 3, Tab 70, letter from Mssrs. Peck, Hendron, Davisson, Loughney, and Gould to S.S. Afifi, 7/2/79
10. Responses to NRC Requests Regarding Plant Fill, Volume 3, Tab 57, letter from S.S. Afifi to Mssrs. Davisson and Hendron, 5/22/79
11. FSAR Subsection 2.5.4.3.2
12. NRC Meeting, 2/28/80 and 2/29/80, Midland, Michigan
13. Responses to NRC Requests Regarding Plant Fill, Volume 3, Tab 55, Meeting Notes, 5/10/79
14. Responses to NRC Requests Regarding Plant Fill, Volume 4, Tab 79, letter from C.H. Gould to S.S. Afifi, 8/3/79
15. Responses to NRC Requests Regarding Plant Fill, Question 12
16. FSAR Subsection 2.5.6.4
17. NRC Midland Site Meeting, Dike Tour, 8/28/80
18. Consumers Power Company letter to NRC, Serial 9697, 9/12/80, Settlement Update

TABLE 1

LABORATORY TEST DATA

SUMMARY OF SOIL PROPERTIES

TO DETERMINE p' - q' RELATIONSHIP

Boring - Sample - Test Series	γ_d (pcf)	w (%)	$p' = \frac{\bar{\sigma}_1 + \bar{\sigma}_3}{2}$ (psf)	$p' = \frac{\bar{\sigma}_1 - \bar{\sigma}_3}{2}$ (psf)
T9 - 8 - 213	117.9	14.4	2,000	1,100
T15 - 3 - 222	118.6	14.2	7,200	3,850
T16 - 5 - 225	114.4	16.9	2,100	1,225
TR2 - U2 - 140	114.6	14.6	3,600	1,800
TR5 - 2 - 147	117.9	14.1	6,000	3,100

NOTES:

γ_d = dry unit weight

w = water content

$\bar{\sigma}_1$ = effective major principal stress

$\bar{\sigma}_3$ = effective minor principal stress

Figure 1
(See Reference 1)

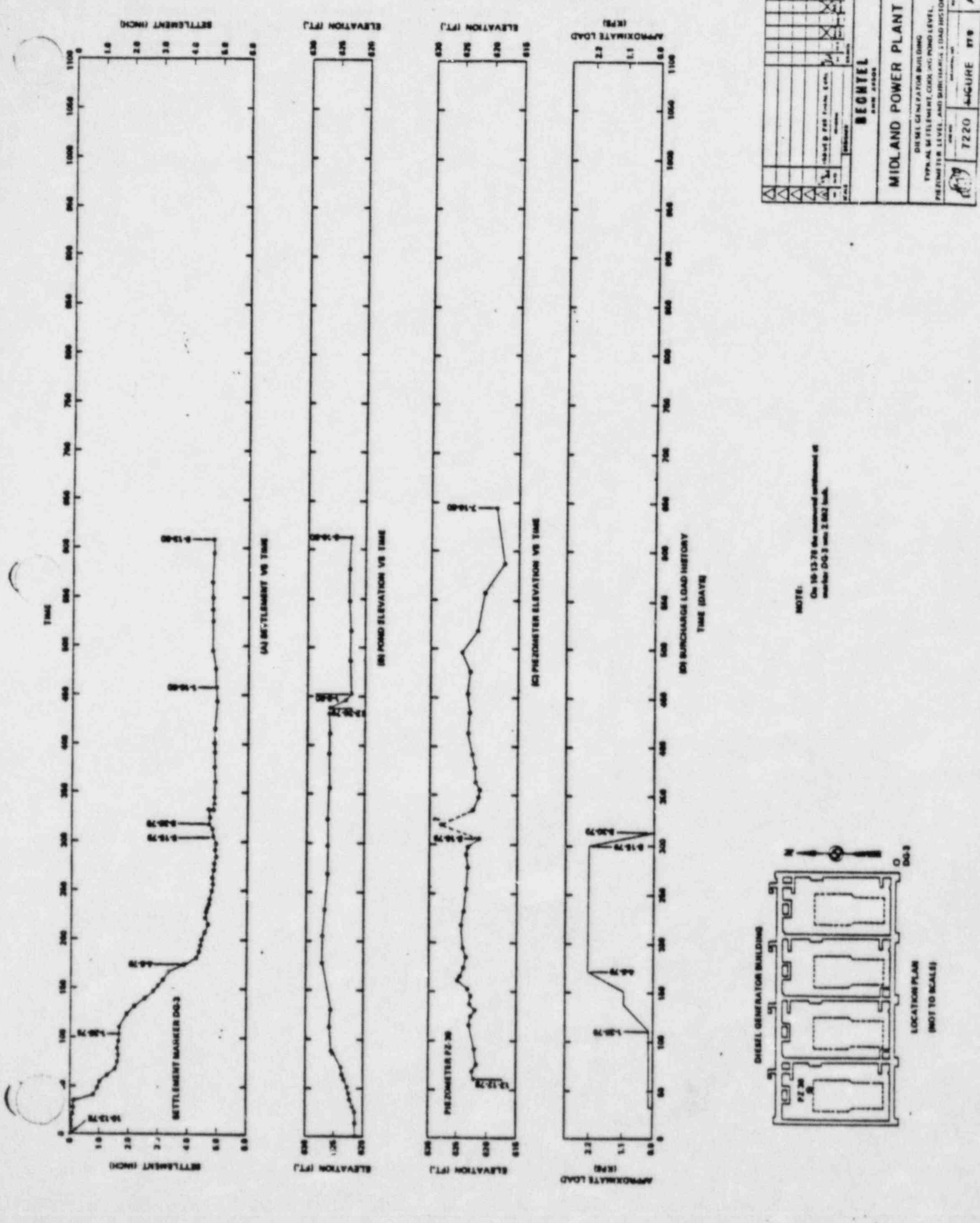
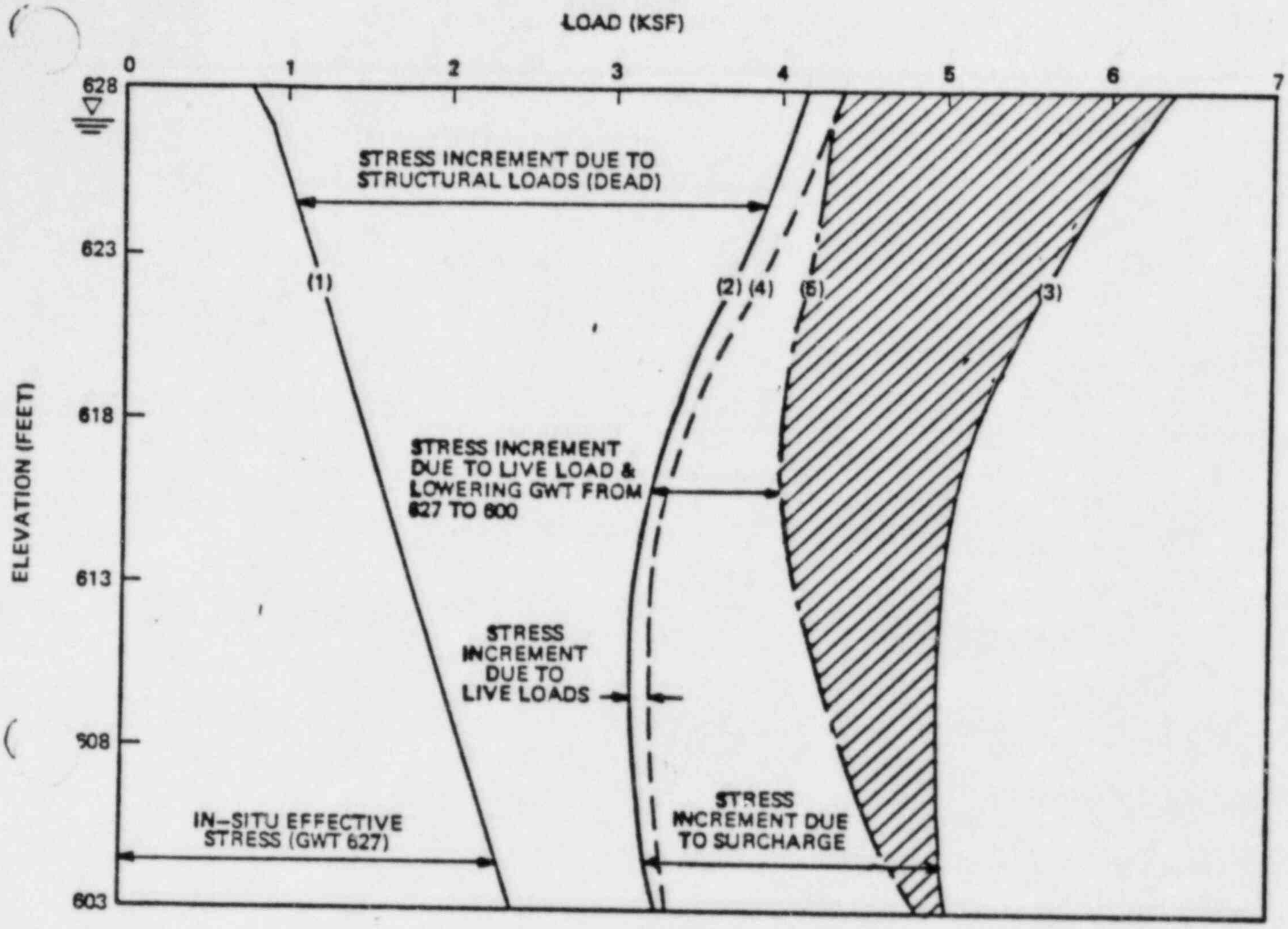


Figure 2
(See Reference 1)



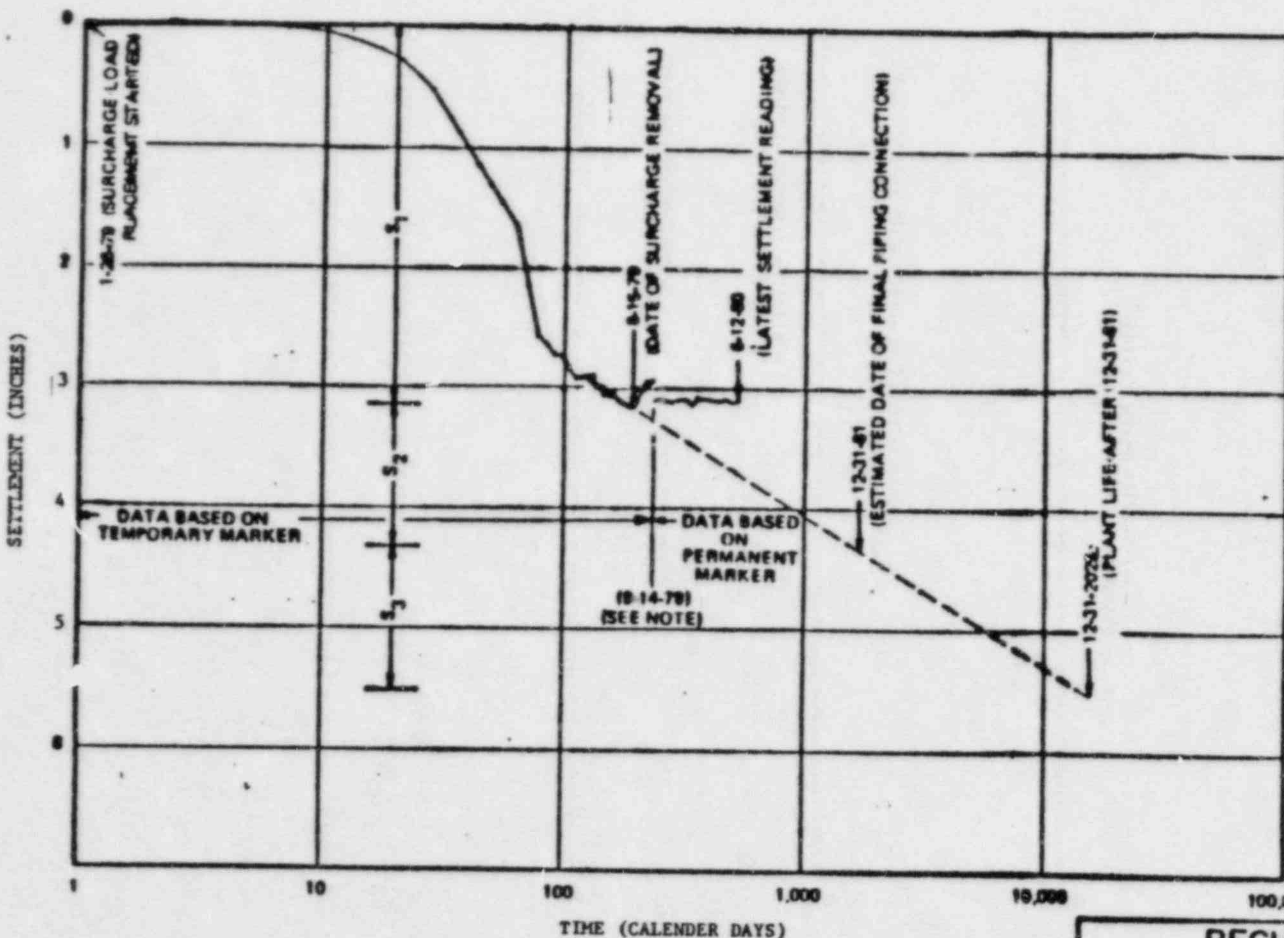
NOTES:

1. (1) In-situ effective overburden pressure GWT at 627.
2. (2) Total effective pressure due to in-situ effective overburden pressure and structural dead loads.
3. (3) Total effective pressure at the end of surcharge due to in-situ effective overburden pressure, structural dead loads, & surcharge loads.
4. (4) Total effective pressure due to in-situ effective overburden pressure, structural dead loads, & live loads.
5. (5) Total effective pressure during the life of plant operation due to in-situ effective overburden pressure, structural dead loads, dewatering loads, & live loads.

COMPARISON OF EFFECTIVE STRESS AT
1) END OF SURCHARGE AND 2) DURING
LIFE OF PLANT OPERATION

SOUTHWEST CORNER OF DIESEL GENERATOR BUILDING

Figure 3
(See Reference 1)



LEGEND

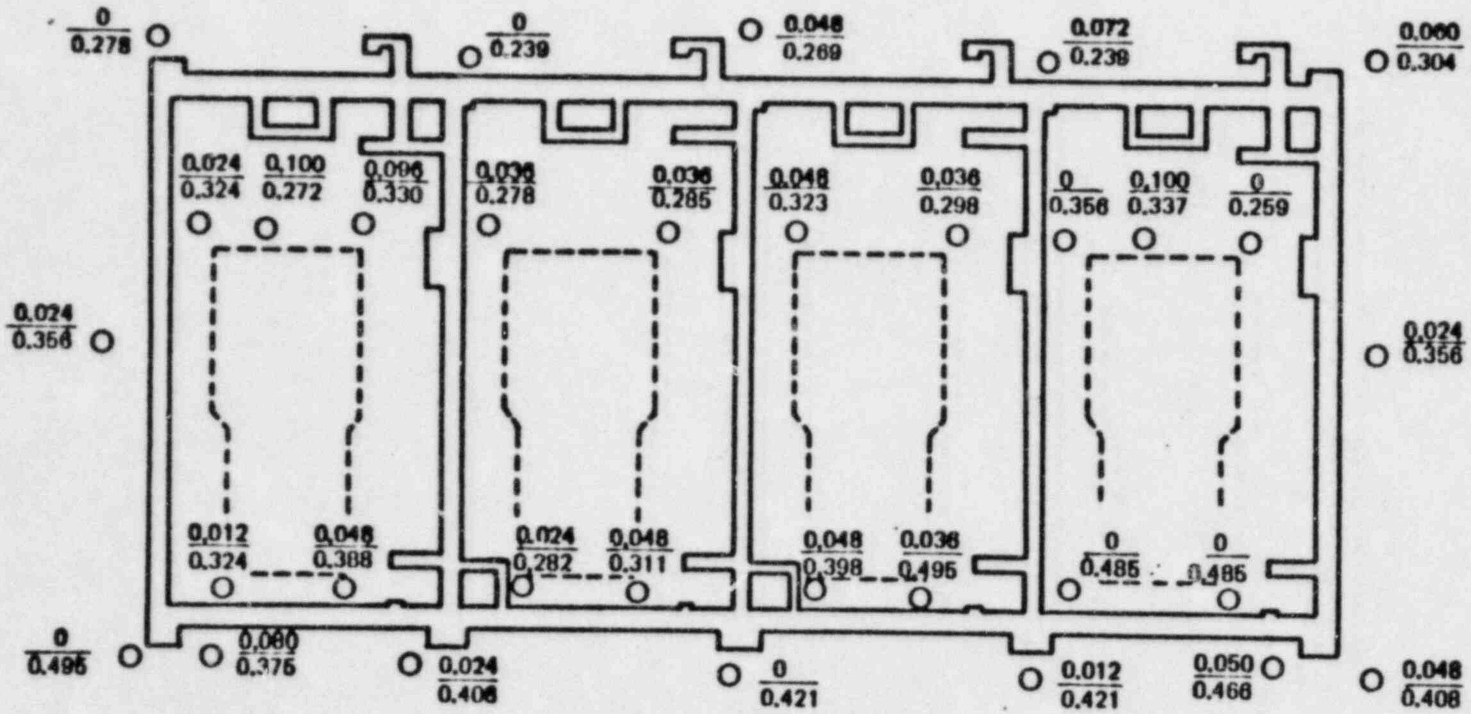
—— MEASURED SETTLEMENT

- - - PREDICTED SECONDARY COMPRESSION SETTLEMENT ASSUMING SURCHARGE REMAINS

NOTE:
The permanent marker could not be monitored from 3-22-79 to 9-14-79 due to surcharge. Temporary markers at elevation 664'-0"± were used during this period to estimate the settlement of the permanent markers. On 9-14-79 the settlement was again based directly upon the permanent markers.

BECHTEL ANN ARBOR		
MIDLAND POWER PLANT		
MEASURED AND PREDICTED SETTLEMENT VS LOG OF TIME (DG-3)		
JOB NO.	DRAWING NO.	REV.
7220	FIGURE	

DIESEL GENERATOR BUILDING



LEGEND:

O — BUILDING / PEDESTAL SETTLEMENT MARKER

0.012 — MEASURED SETTLEMENT BETWEEN 8-15-79 and 6-12-80 IN INCHES
0.421 — PREDICTED SETTLEMENT BETWEEN 8-15-79 and 6-12-80 IN INCHES ASSUMING SURCHARGE REMAINS DURING PLANT LIFE

NOTE:

The measured settlements do not include the heave observed approximately betwe..n 8-15-79 & 9-14-79.

BECHTEL ANN ARBOR	
MIDLAND POWER PLANT	
MEASURED VS PREDICTED SECONDARY COMPP .SSION SETTLEMENT (8-15-79 / 6-12-80) ASSUMING SURCHARGE REMAINS	
JOB NO. 7220	DRAWING NO. FIGURE 27-16

Figure 4
(See Reference 1)

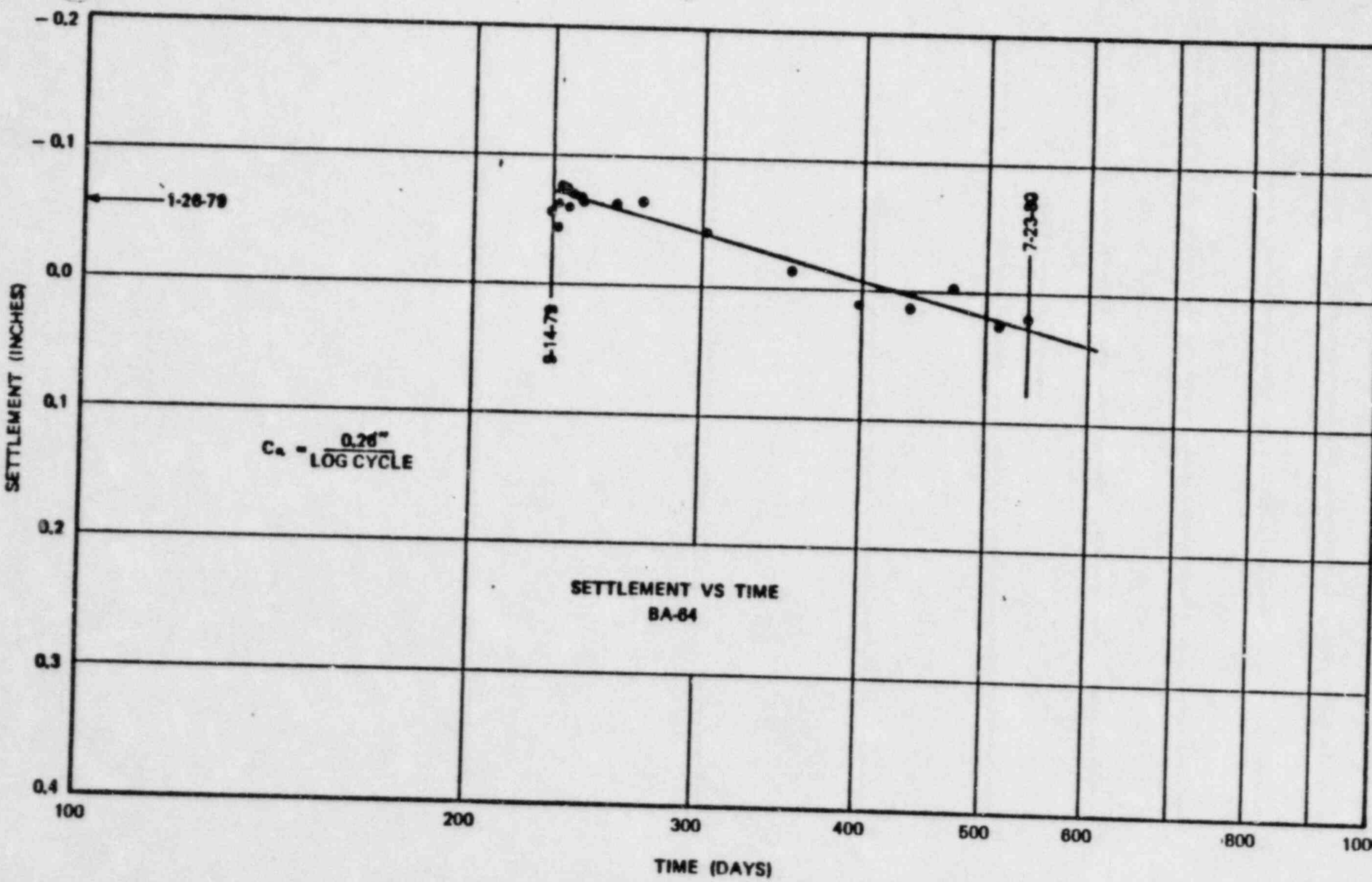
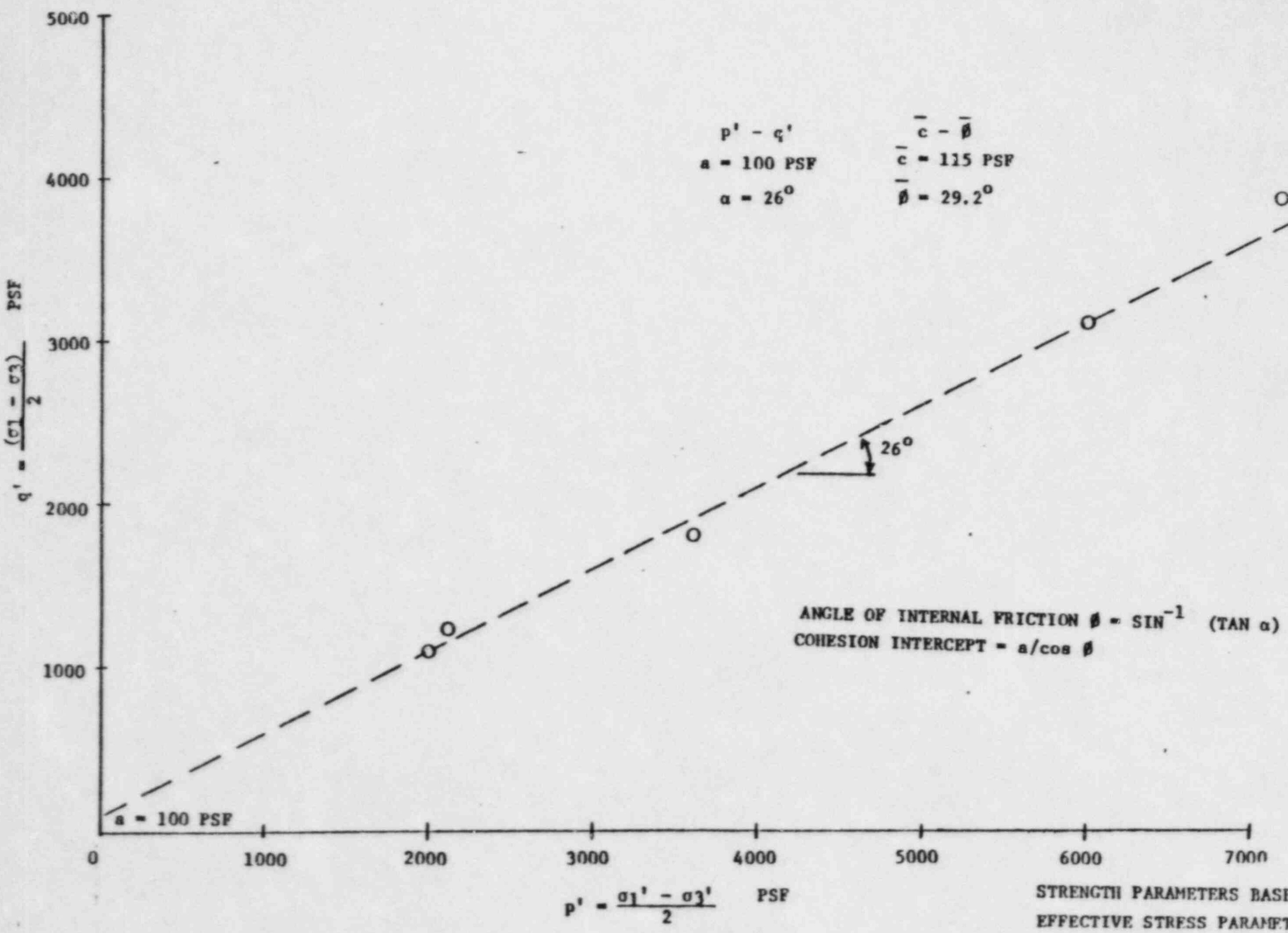
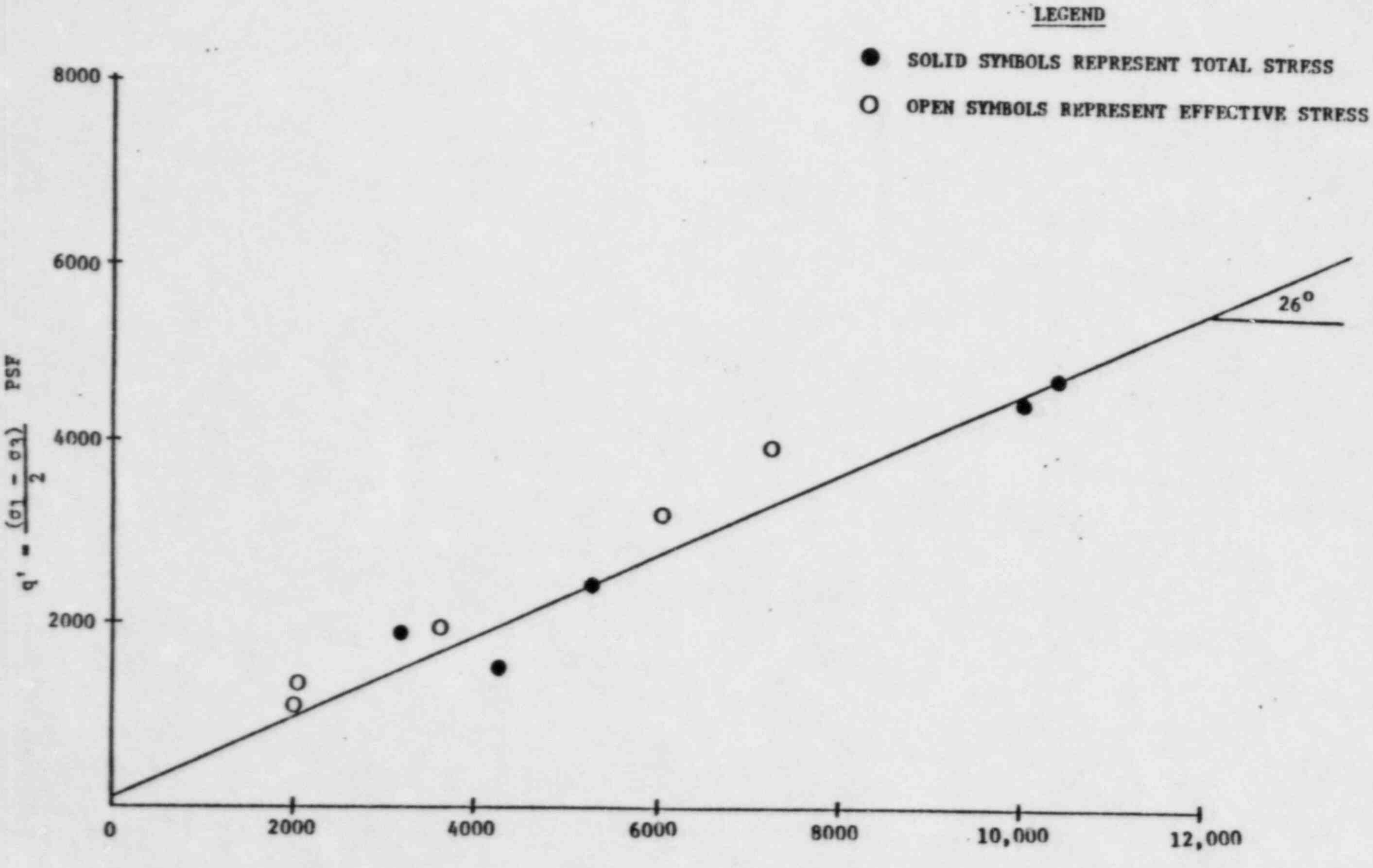


Figure 5
(See Reference 1)



STRENGTH PARAMETERS BASED ON
EFFECTIVE STRESS PARAMETERS

Figure 6
(See Reference 1)



COMPARISON OF TOTAL AND EFFECTIVE
STRESS STRENGTH PARAMETERS

Figure 7
(See Reference 1)

Figure 8 (Sh 1 of 2)
(See Reference 1)

BEARING CAPACITY (D/G BLDG)

A. BASED ON ALL CIU TESTS

$\bar{\phi} = 29^{\circ}$

$\bar{c} = 260$ psf

a). Use T & P

$N_c = 27 \quad N_q = 16 \quad N_\gamma = 15$

$q_d = (260)\ (27) + (125)\ (6)\ (16) + 1/2\ (125)\ (10)\ (15)$

$= 7,020 + 12,000 + 9,375$

$= 28395$ psf

$(q_d)_{net} = 27,645$

$$F.S. = \frac{27,645}{3,400} = 8.13$$

b). Use Vesic

$N_c = 27.9 \quad N_q = 16.4 \quad N_\gamma = 19$

$q_d = (260)\ (27.9) + (125)\ (6)\ (16.4) + 1/2\ (125)\ (10)\ (19)$

$= 7,254 + 12,300 + 11,875 = 31,425$ psf

$(q_d)_{net} = 30,679$ psf

$$F.S. = \frac{30,679}{3,400} = 9.02$$

B. BASED ON FIVE SAMPLES WITH LOWER DENSITIES

$\bar{\phi} = 29^{\circ}$

$\bar{c} = 114$ psf

$N_c = 27 \quad N_q = 16 \quad N_\gamma = 15$

$$q_d = (114)\ (27) + (125)\ (6)\ (16) + 1/2\ (125)\ (10)\ (15)$$

$$= 3,078 + 12,000 + 9,375$$

$$= 24,453 \text{ psf}$$

$$(q_d)_{net} = 23,703 \text{ psf}$$

$$\text{F.S.} = \frac{23,703}{3,400} = 6.97$$

IF WE NEGLECT $\bar{c}$, ASSUME = 0

$$q_d = (125)\ (6)\ (16) + 1/2\ (125)\ (10)\ (15)$$

$$= 12,000 + 9,375$$

$$= 21,375 \text{ psf}$$

$$(q_d)_{net} = 20,625 \text{ psf}$$

$$\text{F.S.} = \frac{20,625}{3,400} = 6.07$$

APPENDIX A

RESUMES FOR CONSULTANTS M.T. DAVISSON,

A.J. HENDRON, AND R.B. PECK

Personal Data Summary of M. T. Davisson

Full Name: Melvin Thomas Davisson   Birth Date: 23 December 1931

Present Positions:

Professor of Civil Engineering, University of Illinois, Urbana, Illinois
Consulting Foundation Engineer

Background:

Native of Ohio. BCE from University of Akron, M.S. and Ph.D. from University of Illinois. Earlier work experience was in construction and structural engineering.

Consulting:

Difficult foundations in waterfront construction including bulkheads, cofferdams and piers; braced cuts, underpinning, grain storage structures; protective construction to resist nuclear blast; deep ocean soil mechanics; foundation vibrations; deep foundations; dynamics of pile driving. Examples are: Hudson River Pier 40 for the Holland-America Lines; Bulkhead supporting McCormick Place in Chicago; Grain Terminal at Sorel, P. Q.; Pile foundations for Locks and Dams in the Arkansas River Project; Minuteman-type construction for U.S. Air Force; Shelter construction for U. S. Army and Navy; Research problems at Nevada Test Site and Suffield Experimental Station; Recommendations for R and D programs in deep-ocean engineering for U. S. Navy; Pile supported runway extensions at LaGuardia Field for Port of New York Authority; R and D on vibratory pile driving for Shell Oil Co.; Foundation vibration problems involving electric power plants and structures such as the No. 14 Newsprint Machine for Price Bros. at Alma P. Q. Foreign projects in Europe, Asia, South America, Central America, Canada and Puerto Rico.

Research:

Behavior of deep foundations (piles, drilled piers, etc.) Settlement of foundations. Soil dynamics. Foundation vibrations. Dynamics of pile driving. Wave equation analysis of impact and vibratory pile driving

Teaching:

Several courses in soil mechanics and foundation engineering for seniors and graduate students. Special course in deep foundations for advanced graduate students.

Technical and Professional Societies:

American Society of Civil Engineers
American Concrete Institute
American Railway Engineering Association
American Society for Testing and Materials
National Society of Professional Engineers

Personal Data Summary of M. T. Davisson, continued

Committee Memberships:

American Railway Engineering Association, Committee 8, Concrete Structures and Foundations.
American Concrete Institute, Committee 543, Concrete Piles.
American Society of Civil Engineers, Committee on Deep Foundations.
American Society for Testing and Materials, Committee D-18, Sub. 11, Tests on Deep Foundations and Committee D-7, Sub. 7, Timber Piles
Highway Research Board, Committee on Soils, Geology and Foundations, Chairman, Subcommittee on Bridges and Other Structures.

Professional Registration:

Professional Engineer - Ohio and Illinois
Structural Engineer - Illinois

Honors and Awards:

Recipient of the Second Annual Alfred A. Raymond Award, 1959, for the paper "Lateral Stability of a Flexible Pier." First place award in international competition for original papers on foundation engineering.
Recipient of the Collingwood Prize, 1964, presented by the American Society of Civil Engineers for the paper, "Laterally Loaded Piles in a Layered Soil System."

Publications:

See attached list.

Publications:

1. R. B. Peck, M. T. Davisson and V. Hansen, discussion of: "Soil Modulus for Laterally Loaded Piles," by B. McClelland and J. A. Focht, Jr., Transactions, ASCE, Vol. 123, 1958, pp. 1065-1069.

2. M. T. Davisson, discussion of: "Experimental Study of Beams on Elastic Foundations," by R. L. Thoms, Proceedings, ASCE, Vol. 87, No. EM1, February 1961, pp. 171-172.

3. D. U. Deere and M. T. Davisson, "Behavior of Grain Elevator Foundations Subjected to Cyclic Loading," Proceedings, Fifth International Conference on Soil Mechanics and Foundation Engineering, Paris, Vol. 1, 1961, pp. 629-633.

4. R. B. Peck and M. T. Davisson, discussion of: "Design and Stability Considerations for Unique Pier," by J. Michalos and D. P. Billington, Transactions, ASCE, Vol. 127, Part IV, 1962, pp. 414-424.

5. R. B. Peck and M. T. Davisson, discussion of: "Friction Pile Groups in Cohesive Soil," by R. L. Kondner, Proceedings, ASCE, Vol. 89, No. SM1, February 1963, pp. 279-285.

6. M. T. Davisson and H. L. Gill, "Laterally Loaded Piles in a Layered Soil System, "Proceedings, ASCE, Vol. 89, No. SM3, May 1963, pp. 63-94.

7. A. J. Hendron and M. T. Davisson, "Static and Dynamic Behavior of a Playa Silt in One-Dimensional Compression," Technical Documentary Report No. RTD TDR-63-3078, AFWL, Kirtland Air Force Base, September 1963.

8. H. Kane, M. T. Davisson, R. E. Olson and G. C. Sinnamon, "A Study of the Dynamic Soil-Structure Interaction Characteristics of Soil," Technical Documentary Report No. RTD TDR-63-3116, AFWL, Kirtland Air Force Base, December 1963.

9. M. T. Davisson and S. Prakash, "A Review of Soil-Sole Behavior," Highway Research Record No. 39, NAS-NRC Publication 1159, Washington, 1963, pp. 25-48.

10. M. T. Davisson, "Estimating Buckling Loads for Piles," Proceedings, Second Pan American Conference on Soil Mechanics and Foundation Engineering, Brazil, Vol. 1, 1963, pp. 351-371.

11. A. J. Hendron, Jr. and M. T. Davisson, "Static and Dynamic Constrained Moduli of Frenchman Flat Soils," Proceedings, Symposium on Soil-Structure Interaction, Tucson, June 1964, pp. 73-97.

12. M. T. Davisson and T. R. Maynard, "Static and Dynamic Compressibility of Suffield Experimental Station Soils," Technical Report No. WL TR-64-118, AFWL, Kirtland Air Force Base, April 1965.

13. M. T. Davisson, discussion of: "Buckling of Long, Unsupported Timber Piles," by E. J. Klohn and G. T. Hughes, Proceedings, ASCE, Vol. 91, No. SM4, July 1965, p. 224.

14. M. T. Davisson, T. R. Maynard and V. G. Koile, "Static and Dynamic Behavior of Sands in One-Dimensional Compression," Technical Report No. AFWL-TR-65-29, AFWL, Kirtland Air Force Base, December 1965.

15. M. T. Davisson and K. E. Robinson, "Bending and Buckling of Partially Embedded Piles," Proceedings, Sixth International Conference on Soil Mechanics and Foundation Engineering, Montreal, Vol. 1, 1965, pp. 243-46.

16. M. T. Davisson, "Design of Deep Foundations for Tall Buildings Under Lateral Load," Proceedings, Structural Engineering In Modern Building Design. Illinois Structural Engineering Conference, Chicago, 1966, pp. 157-174.

17. A. H. Hunter and M. T. Davisson, "Measurements of Pile Load Transfer," ASTM Special Technical Publication, No. 444, Symposium on Deep Foundations, San Francisco, 1968, pp. 106-117.

18. M. T. Davisson and J. R. Salley, "Lateral Load Tests on Drilled Piers," ASTM Special Technical Publications No. 444, Symposium on Deep Foundations, San Francisco, 1968, pp. 68-83.

19. M. T. Davisson and V. J. McDonald, "Energy Measurements for a Diesel Hammer," ASTM Special Technical Publication, No. 444, Symposium on Deep Foundations, San Francisco, 1968, pp. 295-337.

20. M. T. Davisson, discussion of: "Skin Friction for Steel Piles in Sand," by Harry M. Coyle and I. H. Sulaiman, Proceedings, ASCE, Vol. 95, No. SM1, January 1969, pp. 373-374.

21. A. H. Hendron, Jr., M. T. Davisson and J. F. Parola, "Effect of Degree of Saturation on Compressibility of Soils from the Defense Research Establishment Suffield," Report S-69-3, Waterways Experiment Station, Vicksburg, Mississippi, April 1969.

22. M. T. Davisson, "Static Measurements of Pile Behavior," Proceedings, Conference on Design and Installation of Pile Foundations and Cellular Structures, Lehigh University, Bethlehem, April 1970, pp. 159-164.

23. M. T. Davisson, "Design Pile Capacity," Proceedings, Conference on Design and Installation of Pile Foundations and Cellular Structures, Lehigh University, Bethlehem, April 1970, pp. 75-85.

24. M. T. Davisson and J. R. Salley, "Model Study of Laterally Loaded Piles," Proceedings, ASCE, Vol. 95, No. SM5, September 1970, pp. 1605-1627.

25. M. Alizadeh and M. T. Davisson, "Lateral Load Tests on Piles - Arkansas River Project," Proceedings, ASCE, Vol. 96, No. SM5, September 1970, pp. 1583-1604.

26. M. T. Davisson, "Lateral Load Capacity of Piles," Highway Research Record No. 333, Washington, 1970, pp. 104-12.

27. M. T. Davisson, "BRD Vibratory Driving Formula," Foundation Facts, Vol. VI, No. 1, 1970, pp. 9-11.

28. M. T. Davisson and J. R. Salley, "Settlement Histories of Four Large Tanks on Sand," Proceedings, Performance of Earth and Earth-Supported Structures, Purdue University, Lafayette, June 1972, pp. 981-996.

29. M. T. Davisson, "Settlement Histories of Two Pile Supported Grain Silos," Proceedings, Performance of Earth and Earth-Supported Structures, Purdue University, Lafayette, June 1972, pp. 1155-67.

30. M. T. Davisson, "Inspection of Pile Driving Operations," Technical Report M-22, Department of the Army, Construction Engineering Research Laboratory, Champaign, July 1972.

31. M. T. Davisson, "High Capacity Piles," Proceedings, Lecture Series, Innovations in Foundation Construction, SM&FD, Illinois Section ASCE, Chicago, 1973.

32. M. T. Davisson and D. M. Rempe, "Wave Theory Simplified," Piletalk Seminar, New Jersey, 1974.

33. M. T. Davisson, "Pile Foundations and the Computer," Use of Computers in Foundation Design and Construction, Metropolitan Section ASCE, New York, April 1974.

Professional Background and Experience

Name: Alfred J. Hendron, Jr.

Address: 2230c Civil Engineering Building
University of Illinois at Urbana-Champaign
Urbana, IL 61801

Date of Birth: October 4, 1937

Marital Status: Married with 2 children

Citizenship: Natural Born - U.S.

Education

Ph.D.	1963	University of Illinois Urbana, Illinois	Major: Soil Mechanics Foundations Minors: Geology Theoretical and Applied Mechanics
M.S.	1960	University of Illinois Urbana, Illinois	Civil Engineering
B.S.	1959	University of Illinois Urbana, Illinois	Civil Engineering

Positions Held

September 1970 - Present	Professor of Civil Engineering University of Illinois
September 1968 - September 1970	Associate Professor of Civil Engineering University of Illinois
September 1965 - September 1968	Assistant Professor of Civil Engineering University of Illinois
September 1963 - September 1965	1/Lt. U. S. Army Corps of Engineers Research Engineer U. S. Army Engineer Waterways Experiment Station
June 1961 - September 1963	Research Associate University of Illinois
June 1960 - September 1960	Engineer, Shannon & Wilson Soil Mechanics and Foundation Engineers Seattle, Washington

Alfred J. Hendron, Jr.

Offices held and other services to professional societies

(1) Member of the Research Committee of the Soil Mechanics and Foundations Division of the American Society of Civil Engineers (1967-69).

(2) Member of Subcommittee 12 of Committee D-18, ASTM, Properties of Soil and Rock, 1965-1970.

(3) Co-chairman of Panel on "Stress Wave Propagation in Soils," International Symposium on Soil Dynamics, Albuquerque, New Mexico, sponsored by ASCE & NSF, August 1967.

(4) Panel member for "Dynamic Loading," Session of a national Specialty Conference on Placement and Improvement of Soil to Support Structures," sponsored by the Soil Mechanics and Foundations Division of the American Society of Civil Engineers, M.I.T., August 1968.

(5) April 1968 - Gave lectures on rock mechanics to Metropolitan Section ASCE, New York City.

(6) April 1969 - Gave lectures on rock mechanics to Metropolitan Section ASCE, Washington, D.C.

(7) Selected to give a lecture on "Field Instrumentation in the Design of Underground Structures in Rock," Metropolitan Section, ASCE, New York City, May 1970.

(8) Panel member on "Dynamic Loadings and Deformations," Session for ASCE, Soil Mechanics and Foundations Division Specialty Conference on "Lateral Stresses in the Ground and the Design of Earth Retaining Structures," Cornell University, June 1970.

(9) Member of Panel on "Deformation Modulus of Rock Foundations," ASTM Symposium on Deformation Properties of Rock, Denver, February 1969.

(10) Selected by NSF as one of the U. S. Members to exchange meeting with Japanese Engineers on the Topic of Ground Motions produced by earthquakes, U. of California at Berkeley, August 1969.

(11) Member of Committee on Soil Dynamics, Soil Mechanics Division, ASCE, 1970 - present.

(12) Member of Publications Committee for Journal of the Soil Mechanics and Foundations Division, ASCE, 1970 - present.

Alfred J. Hendron, Jr.

Examples of Foundation Engineering and Earthquake Engineering Experience

1. Consultant to Williams Brothers Construction Company on slope stability problems encountered in construction of the Transandean Pipeline in southern Colombia, S.A.

2. Consultant to Woodward-Clyde and Associates on the Foundation Design of Davis-Besse Nuclear Reactor for earthquake loadings.

3. Consultant, as an associate of Dr. N. M. Newmark, on the foundations for a 40 story building in Vancouver, B.C., designed for earthquake loading.

4. Consultant to Waterways Experiment Station on the Earthquake Stability of Dam Slopes.

5. Consultant to H. G. Acres Ltd. on Seismic considerations for Nuclear Reactor Foundations as a part of a study for 6 New England States on Projected Power Needs.

6. Consultant, as an associate of Dr. N. M. Newmark, to the Divisions of Reactor Licensing and Reactor Safety of the Atomic Energy Comission, on the adequacy of nuclear reactor foundations tc resist earthquake loading, September 1967 - present. The following is a list of the Nuclear Power Station Foundations reviewed during this time:

Ft. Calhoun	Arnold
Cooper	Pilgrim
Surry	Crystal River
Shoreham	Prairie Island
Salem	Farley
Rancho Seco	Calvert Cliffs
Diablo Canyon	Oconee
Sequoyah	Indian Point
Hatch	Bailey
Brunswick	D. C. Cook
Kewaunee	Zimmer
Fitzpatrick	3 Mile Island
Fermi	Russellville
Turkey Point	Easton
Bell	

7. Dynamic stability assessment of 3 TVA dams subjected to design earthquakes.

Alfred J. Hendron, Jr.

Experience on Design of Protective Structures and Nuclear Effects

1. Consultant to TRW Systems, Redondo Beach, California on Dynamic Soil Properties pertinent to the hardness of the Minuteman System.

2. Presently member of a panel in Dept. of Defense to review design of all Safeguard Structures for Vulnerability and hardness.

3. Consultant to Omaha District Corps of Engineers on the construction of underground protective structures in rock.

4. Consultant to Air Force Space and Missile Systems Organization on Hardness of Minuteman Structures as an associate of Dr. N. M. Newmark.

5. Consultant on problems in soil dynamics and rock mechanics to the U. S. Army Engineer Waterways Experiment Station, Vicksburg, Ml.

6. A member of the "Decoupling Advisory Group" formed by the Defense Atomic Support Agency. Responsibility is to comment on stability problems which might be encountered in building underground cavities 100-360 ft in diameter and to give the shear strength properties of rock masses which are important in determining the decoupling characteristics of cavities over-driven by the detonation of a nuclear device.

7. Received Army Commendation Medal in 1965 for representing the Chief of the Coprs of Engineers as a consultant to the Norwegian Government and NATO on the engineering of large underground facilities.

Recent Publications

"The Behavior of Sand in One-Dimensional Compression," Ph.D. Thesis, U of I, Dept. of Civil Engr., July 1963; "The Dynamic Stress-Strain Relations for a Sand as Deduced by Studying its Shock Wave Propagation Characteristics in a Laboratory Device," w/T. E. Kennedy, Proceedings of the 1964 Army Science Symposium, Vol. II, West Point, N.Y., June 1964; "Static and Dynamic Constrained Moduli of Frenchman Flat Soils," with M. T. Davisson, Proceedings of the Symposium on Soil-Structure Interaction, Univ. of Arizona, Tucson, Arizona, Sept. 1964; "Damage to Model Tunnels Resulting from an Explosively-Produced Impulse," with G. B. Clark and J. N. Strange, U. S. Army Engineer Waterways Experiment Station, Vicksburg, Mississippi, Research Report No. 1-6, Report 1, May 1965; "The Design of Surface Construction in Rock," w/D. U. Deere, F. D. Patton, and E. J. Cording, Ch. II in Failure and Breakage of Rock, American Inst. of Mining Metallurgical and Petroleum Engineer, 1967. "The Effect of Soil Properties on the Attenuation of Air Blast-Induced Ground Motions," with H. E. Auld, pp. 29-47, Proceedings of the International Symposium on Wave Propagation and Dynamic Properties of Earth Materials, University of New Mexico Press, 1968. "Mechanical Properties of Rock," Chapter 2, pp. 21-53, of the book "Rock Mechanics in Engineering Practice," edited by K. G. Stagg and O. C. Zienkiewicz, published by John Wiley & Sons, London, 1968, 442 pg.

Alfred J. Hendron, Jr.

"Dynamic Behavior of Rock Masses," with N. N. Ambraseys, Chapter 7, pp. 203-236 of the book "Rock Mechanics in Engineering Practice" edited by K. G. Stagg and O. C. Zienkiewicz, published by John Wiley and Sons, London, 1968, 442 pages. "Foundation Exploration for Interstate 280 Bridge over Mississippi River near Rock Island Illinois," with J. C. Gamble and G. Way, Proceedings of the Twentieth Annual Highway Geology Symposium, University of Illinois, Engineering Experiment Station, Urbana, 126 pp. "Compressibility Characteristics of Shales Measured by Laboratory and In Situ Tests," with G. Mesri, J. C. Gamble and G. Way, pp. 137-153, ASTM Special Technical Publication 477, "Determination of the In Situ Modulus of Deformation of Rock," June 1970. "Rock Engineering for Underground Caverns," with E. J. Cording and D. U. Deere (In Publication, ASCE Proceedings of a Symposium on the Design of Large Underground Openings, Phoenix, Arizona, February, 1971). "Dynamic Stability of Rock Slopes," with E. J. Cording, (In Publication, Proceedings of the 13th Symposium on Rock Mechanics, Univ. of Illinois, 1971). "State of the Art of Soft-Ground Tunneling," with R. B. Peck and B. Mohraz, Proceedings of the 1st North American Rapid Excavation and Tunneling Conference, Chicago, Illinois, June 5-7, 1972, AIME, 1972, pp. 259-286. "Specifications for Controlled Blasting in Civil Engineering Projects," with L. L. Oriard, Proceedings of the 1st North American Rapid Excavation and Tunneling Conference, Chicago, Illinois, June 5-7, 1972, AIME, pp. 1585-1610.

<u>Consulting Experience Directly Applicable for the Design of Large Underground Chambers for Storage</u>

1. 1971-present: Consultant to Gulf Oil on 4 large underground chambers for storage of gas, Fannett Dome, Texas.

2. 1972-present: Consultant to Dome Petroleum on the use of salt caverns in Windsor Canada for gas storage. Caverns in service now, status reviewed 3 or 4 times a year.

3. Consultant to Morton Salt on control of solution mining in the following brinefields
   - Port Huron, Michigan
   - Rittman, Ohio
   - Hutchinson, Kansas

4. Consultant to the Solution Mining Research Institute on subsidence and cavity stability
   Report on a study of sinkhole development above cavities in two brinefields and discussion of means for detecting this behavior sufficiently in advance to prevent such behavior.

5. Consultant to BASF-Wyandotte, Wyandotte, Michigan on control of subsidence and prevention of sinkhole formation above cavities in bedded salt.

6. Consultant to Duke Power Co. on current design of Bad Creek underground powerhouse.

7. Past consultant to British Columbia Hydro-Authority on stability of the Portage Mountain Underground Powerhouse. (96 ft span, 1000 ft long, 180 ft high).

8. Consultant to Morton Salt on the possible use of the Silver Springs brine field for gas storage.

9. Consultant to U. S. Department of Defense on many tunnels and underground chambers at Nevada Test Site.

10. Past consultant to U. S. Corps of Engineers on the use of large underground structures in rock for protective construction.

11. Consultant to NATO and Norwegian Government in 1965, as a Corps of Engineer officer, on large underground chamber construction. Received Army commendation medal for this assignment.

NAME: Ralph B. Peck

EDUCATION: B. S., Civil Engineering
Rensselaer Polytechnic Institute

D.C.E.
Rensselaer Polytechnic Institute

Post-doctoral studies, Engineering
Harvard University

PROFESSIONAL LICENSES: Illinois: Structural and Professional Engineer (1942)
Member, Illinois Structural Engineer Examining Board since 1959
Hawaii (1956)
California (1963)

FIRM: Ralph B. Peck - Civil Engineer: Geotechnics (1975-Present)
(Bechtel Consultant)

EXPERIENCE and QUALIFICATIONS:

Summary

45 Years: Internationally known consultant on foundation and stability conditions for tunnels, heavy loaded structures, and subways. Former professor of foundation engineering at University of Illinois. Dr. Peck is the author of more than 70 technical publications dealing with foundations, earth pressures, tunnels, slopes, earthdams, etc. He collaborated on Soil Mechanics in Engineering Practice, Foundation Engineering, and From Theory to Practice in Soil Mechanics. In 1944, he was awarded the Norman Medal of the American Society of Civil Engineers.

1930-Present: Dr. Peck is an internationally known consultant specializing in soil mechanics and foundation engineering. He has investigated bracing systems for open cuts for subways and deep excavations and has served as consultant on large dams in the United States, Colombia, Puerto Rico, Hawaii, Costa Rica, British Columbia, New Brunswick, The Philippine Islands, Canal Zone, and Greece.

Professor Peck has been a member of the boards of consultants for flexible paving design, pipe cover studies, the Garrison Dam Test tunnel, foundations for the Savannah River project, dynamic soil testing, Lincoln AFB missile sites for the Corps of Engineers.

He has also worked on defense projects for the Rand Corporation, the Ramo-Wooldridge Corporation, and the Aerospace Corporation.

1950-1975: For twenty-five years, Dr. Peck taught on the college level. He was a lecturer at Illinois Institute of Technology, then assistant professor, associate professor, and professor of foundation engineering at University of Illinois.

Consumers Ex 14 i.d.
10/8/80 (Hood)

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

JUN 13 1979

Docket No. 50-329/330

MEMORANDUM FOR: Dudley Thompson, Executive Officer for Operations Support, IE

FROM: Harold D. Thornburg, Director
Division of Reactor Construction Inspection, IE

SUBJECT: COMMENTS ON RIII ENFORCEMENT PACKAGE ON MIDLAND SETTLEMENT PROBLEMS DATED APRIL 3, 1979

We have reviewed the above referenced package which under J. Davis's memorandum of March 21, 1979 was forwarded to XOOS as the responsible coordinating group within IE. These comments are provided to be consistent with this memorandum and the follow-up memorandum you provided to your enforcement personnel also on March 21, 1979.

In summary, it is our opinion that four of the five false statements identified by the Region will probably be substantiated to be material false statements and that they were made in careless disregard of the facts. Therefore, it would follow that there would probably be four instances of a material false statement each of which would have a civil penalty of $5,000 imposed for it. The fifth item is not, in our opinion, a material false statement.

The enclosure presents our detailed recommendations on this matter. If you have questions please contact us.

Harold D. Thornburg
Harold D. Thornburg, Director
Division of Reactor Construction Inspection, IE

Enclosure:
Comments on Midland
Enforcement Package

CONTACT: R. E. Shewmaker, IE
49-27551

7908070022

COMMENTS ON MIDLAND ENFORCEMENT PACKAGE TRANSMITTED TO THORNBURG FROM KEPPLER, DATED 4/3/79

1. The material false statement items (probably 4) should be put into an Appendix A entitled, "Notice of Violation," and will be those items with a civil penalty. An Appendix B entitled, "Notice of Proposed Imposition of Civil Penalties" should be prepared. The other items of noncompliance should be addressed in an Appendix C, "Notice of Violation."

2. All statements quoted from the SAR in the citations should be clearly identified by amendment number and/or revision number and date.

see Item 3 pg I-4 in 50.54 Request

3. A check of Statement 1 regarding fill and backfill placement shows it is apparently from the original version of the FSAR. Revision 1, 11/22/77 has a different statement and is the current version. Some of the other statements referenced have been revised now after the investigation. This must be reexamined. If the statements quoted in the RIII draft can be utilized in an enforcement action then we judge the statement to be a material false statement. In reaching this conclusion we note that there is a need to quote or provide a copy of the text from construction drawings C-45 stating that Zone 2 material is to be used as Class I fill if the citation is to be properly supported.

see pg I-4 response 54 Request

4. Statement #2 can probably be classed as a material false statement if the results of the interview with the cognizant engineer and/or the calculation sheet prove that 3.0 ksf was used in the settlement calculations.

" Item 4 pg I-4

5. Statement #3 is viewed to be a material false statement, but there is a need to fully document what was actually done in the execution of the calculations. Again a copy of the calculation sheet and/or a statement of the cognizant engineer is needed to properly support the finding.

" Item 4 pg I-4

6. Statement #4 can probably be classed as a material false statement if the results of the interview and/or the calculations are provided to support the finding.

7. Statement #5 is judged to not be a material false statement. This is due to the fact that the statement quoted is written as a predicted future value for settlement.

8. For those statements which will become material false statements with a civil penalty, remove them from the draft Appendix A and move the remainder to the new Appendix C.

9. All statements judged to be material false statements must be examined to see in what "state of mind" or in what circumstances the licensee made the statement. This is relevant to the question of "civil penalty" vs. "second chance." In our judgment these instances appear to be situations of "careless disregard" of the facts which would warrant civil penalty.

cc w/enclosure:
J. G. Keppler, RIII
D. W. Hayes, RIII
T. W. Brockett, XOOS
G. W. Reinmuth, RCI
R. E. Shewmaker, RCI

7 1 8 1979

-- Consumers Ex#151e
10/3/80 (Hood)

C

1979

MIDLAND SOIL SETTLEMENT/QA CONCERN

1. 50.54(f) sent to Consumers Power Company in March 1979. At that time IE recommended to NRR that a show cause be issued to stop construction. It was agreed (NRR/IE) that 50.54(f) would be sufficient.

2. General question of QA adequacy of Utility/AE was discussed internally by IE/NRR on August 16. IE was to ask region to make a finding as to adequacy of QA implementation. Special consideration was to be given soils settlement matter in relation to the reports of QA deficiencies in other areas.

3. Latest response to 10 CFR 50.54(f) follow-on questions regarding QA of plant fill received on 11/13/79. (Tentative QA Branch position suggests response still unsatisfactory.)

4. Review of Midland Soils Settlement submittals given to Corps of Engineers at end of October. (Tour of site made by Corps of Engineers & NRR staff November 14.)

5. To date, 5 utilities' replies to 50.54(f) have not described acceptance criteria for remedial action, prior to such action. Applicant views the remedial actions as "proof tests" which preclude need for such criteria. Staff decision as to acceptability of remedial action must await completion of the program, and applicant must proceed entirely at his risk.

6. In a meeting on November 28, IE developed a new position:

   a. Overall QA performance acceptable because it identifies QA deficiencies;

   b. IE now raises question as to the acceptability of the design fix and draws the conclusion that the modification constitutes a departure from the principal architectural and engineering criteria;

   c. IE suggests Stello/Denton meeting ASAP to develop a decision for enforcement actions relative to applicant's failure to comply with design approved by CP.

Stello

Darl gave this 11-29-79
document ~~as~~ to
Edcase on 11-29-79

Consumers Scott 16 id
10/3/80 (HOOD)

D. Hood

AUG 24 1979

MEMO TO FILE

FROM: D. Hood, Project Manager, Light Water Reactors Branch No. 4, DPM

SUBJECT: INTERNAL MEETING ON STATUS OF MIDLAND SOILS SETTLEMENT

On August 16, 1979, members of NRR, I&E Headquarters and OELD met to discuss the status of the staff's review of the soils settlement matter at the Midland site. The purpose was to determine the status of the staff's decision pursuant to 10 CFR 50.54f (which is applicable to construction permits by 10 CFR 50.55(c).) The principal background documents to date are listed in Enclosure 1. Meeting attendees are listed in Enclosure 2.

Mr. Knight reported that the principal technical solutions proposed by the applicant for the major structures appears to be basically sound such that, properly implemented, they can be expected to provide for adequate structural foundation support. He noted, however, that certain details of the applicant's reply were not sufficient and further information will be required from the applica For example, the details of the applicant's load combination calculations and stress limits applicable to differential settlement, NRR's need for a more quantitative assessment to determine that nozzle loads transmitted from settled pipes to the attached valves, pumps, tanks, etc will remain within ASME Code allowables, and a more thorough monitoring program to follow actual performance during operation. These findings and further requests are being documented and will be completed in late August.

Messrs Haass and Gilray of QAB noted that some instances of poor performance in QA areas revealed in the I&E investigation report indicates that additional QA measures beyond those typically imposed by the NRC may be warranted. QAB's review is in its final stages of documentation and should be completed before the end of August.

what?

Mr Thornburg noted I&E is continuing its review of the performance aspects of the QA program and considering the soils settlement matter in relation to the reports of QA deficiencies in other areas. Mr. Thornburg anticipates that I&E will reach its conclusions by mid-September 1979.

OELD referenced a Memorandum and Order from ASLB dated August 2, 1979 which asks for clarification of the staff's position regarding consideration of the diesel generator building settlement issue. The board cannot determine from the staff's response whether the staff simply prefers not to issue a partial SER or whether there are other considerations making early consideration of this issue impossible or impractical. Mr. Omstead will prepare a reply clarifying the staff's DES schedule and explaining why isolation of the DG building issue is not practical.

Mr. Rubenstein described the approach which DPM will take in arriving at an NRC position on the technical qualification findings for the SER. The approach
? → is that defined in a W. Haass memo dated 12/15/78, which calls for inputs from QAB, I&E, DOR and DPM.

Mr. Vassallo emphasized the need for timely decisions to be reached by the staff and for similar status meetings in the near future.

Darl Hood

D. Hood

ENCLOSURE 1

BACKGROUND DOCUMENTATION

Background Documentation relevant to NRR's 10 CFR 50.54(f) requests dated March 21, 1979 include the following: The applicant's reply dated April 24, 1979, was revised May 31, 1979 (revision 1), and July 9, 1979 (revision 2). Further information was supplied by the applicant during meetings attended by both I&E and NRR on March 5 and July 18, 1979. In addition, certain information was requested by NRR technical branches as part of the FSAR review prior to issuance of the 10 CFR 50.54(f) requests and are replied to through FSAR amendments. Site visits by NRR staff to observe settlement were made March 6 and June 7, 1979, and December 3, 1978. NRR participation with I&E results from a Transfer of Lead Responsibility which was distributed to technical review branches as part of a technical assistance request dated November 27, 1978.

Background documentation directed to I&E includes a 50.55(e) notification by the applicant dated September 29, 1978, for which six interim reports have been issued to date (November 7, 1978; December 21, 1978; January 5, 1979; February 23, 1979; April 30, 1979; and June 25, 1979). I&E has conducted a preliminary investigation and has documented its summary findings, along with the applicant's discussion of these findings, in a letter to the applicant dated March 15, 1979. Enforcement actions due to potential material-false statements in the FSAR as may be applicable to some of these I&E findings are presently under internal review, assisted by NRR staff as appropriate.

ENCLOSURE 2

ATTENDEES

J. Knight
D. Skovholt
W. Haass
D. Vassallo
S. Varga
L. Rubenstein
D. Hood
H. Thornburg
R. Shewmaker
R. Backman
W. Omstead
R. Lieberman
J. Gilray
J. Spraul

Midland Staffing

Agenda - Oct 1, 1980

Opening Remarks	Lear
Background	Heller
DG Bldg	Kane
SW Structure	Singh
Aux Bldg	Singh
Cooling Pond Dike	Kane, Bivins
Retaining Walls	Singh, Otto
Summary of Corps of Eng review on recent submittals of additional explorations	Singh, Otto
Statement by Corps on importance of the requested borings & testing on the safety review	Simpson, Otto
Concluding Remarks	Lear

Others -
W Paton
D Hood
R Vollmer
J Knight
A Cappucci

Consumers Ex 17id
10/8/80 (Hood)

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

RECEIVED
JWC
SEP 02 1980
MIDLAND PROJECT
MANAGEMENT

August 27, 1980

Docket Nos. 50-329
and 50-330

CC: SHH, JWC, GSK, TRT, TCC, JEB, KWeidner, DMB, NJSaari, MEGibbs (IL&B), Serial

File 0485.16
JARutgers

Mr. J. W. Cook
Vice President
Consumers Power Company
1945 West Parnall Road
Jackson, Michigan 49201

Dear Mr. Cook:

SUBJECT: REQUEST FOR ADDITIONAL INFORMATION REGARDING DEWATERING OF MIDLAND SITE

Amendment No. 74 to your application dated February 28, 1980, provided information regarding a permanent dewatering system proposed for the Midland site in response to Request No. 24 from Mr. L. Rubenstein's letter of November 19, 1979. The review by the hydrologic section of our Hydrologic and Geotechnical Engineering Branch indicates the need for further information regarding that response as identified in Enclosure 1. This information is in addition to related requests contained in our letter of August 4, 1980.

We would appreciate your reply to Enclosure 1 at your earliest opportunity. Should you need clarification of these requests for additional information, please contact us.

Sincerely,

R Tedesco

Robert L. Tedesco, Assistant Director
for Licensing
Division of Licensing

Enclosure:
Request for Additional
Information

cc w/encl:
See next page

~~8009120199~~

August 27, 1980

Mr. J. W. Cook
Vice President
Consumers Power Company
1945 West Parnall Road
Jackson, Michigan 49201

cc: Michael I. Miller, Esq.
Isham, Lincoln & Beale
Suite 4200
1 First National Plaza
Chicago, Illinois 60603

Judd L. Bacon, Esq.
Managing Attorney
Consumers Power Company
212 West Michigan Avenue
Jackson, Michigan 49201

Mr. Paul A. Perry, Secretary
Consumers Power Company
212 West Michigan Avenue
Jackson, Michigan 49201

Myron M. Cherry, Esq.
1 IBM Plaza
Chicago, Illinois 60611

Ms. Mary Sinclair
5711 Summerset Drive
Midland, Michigan 48640

Frank J. Kelley, Esq.
Attorney General
State of Michigan Environmental
Protection Division
720 Law Building
Lansing, Michigan 48913

Mr. Wendell Marshall
Route 10
Midland, Michigan 48640

Grant J. Merritt, Esq.
Thompson, Nielsen, Klaverkamp & James
4444 IDS Center
80 South Eighth Street
Minneapolis, Minnesota 55402

cc: Mr. Steve Gadler
2120 Carter Avenue
St. Paul, Minnesota 55108

Mr. Don van Farowe, Chief
Division of Radiological Health
Department of Public Health
P. O. Box 33035
Lansing, Michigan 48909

William J. Scanlon, Esq.
2034 Pauline Boulevard
Ann Arbor, Michigan 48103

U. S. Nuclear Regulatory Commission
Resident Inspectors Office
Route 7
Midland, Michigan 48640

August 27, 1980

[illegible] Naval Surface Weapons Center
Attn: [illegible]
[illegible]
[illegible]
[illegible], Maryland [illegible]10

[illegible]
[illegible] Engineering
[illegible] Engineering Center
[illegible] Box [illegible]49
[illegible] Park, California [illegible]

[illegible] William [illegible]
[illegible] S. Corps of Engineers
[illegible]D - T
[illegible] Floor
[illegible]
[illegible], Michigan [illegible]26

[illegible]

[illegible]

Mrs. Sandra D. [illegible]st
[illegible] Fourth Street
Bay City, Michigan [illegible]06

[illegible]
[illegible]
[illegible], Michigan 4[illegible]40

[illegible]
[illegible]
[illegible], Michigan [illegible]06

[illegible] C. [illegible], Sr.
[illegible]
[illegible], Michigan [illegible]03

[illegible]
[illegible] Street
[illegible], Michigan [illegible]01

cc: Mr. William A. Thibodeau
3245 Weigl Road
Saginaw, Michigan 48603

Mr. Terry R. Miller
3229 Glendora Drive
Bay City, Michigan 48706

SUPPLEMENTAL REQUESTS REGARDING PLANT FILL

49. Your response to our Request 24 states that if the dewatering system should fail, more than 90 days would occur before groundwater levels would rise to elevation 610 feet, the groundwater elevation at which liquefaction would become a problem. We are concerned that this water level rise might occur over a period considerably less than 90 days in view of the following apparent discrepancies in equations and input parameters:

    a. The error function solution to the partial differential equation describing unsteady groundwater flow which you used to determine permeability, appears to be incorrect; the correct form should have a 4 in the denominator, instead of a 2 as you have shown. The correct equation is:

$$h = H\left(1 - \operatorname{erf}\frac{x}{\sqrt{4K\bar{h}t/n_e}}\right)$$

    where:

    h = water level rise at $X=0$

    H = water head at $X=0$

    $\bar{h}$ = average depth of water

    erf = error function

    K = permeability

    X = distance

    t = time

    n_e = effective porosity

b. In the above equation since $\bar{h}$ is the average depth, its value should lie between h and H. In applying this equation to compute a permeability K of 11 feet per second and a corresponding rebound time of 90 days, you used 0.1 foot for h, 1.6 feet for H, but 20 feet for $\bar{h}$. Use of a smaller value of $\bar{h}$ (somewhere between 0.1 and 1.6 feet) would result in a higher permeability and a rebound time considerably shorter than 90 days.

c. Your value for x in the above equation is 325 feet, which you say is the shortest distance between the critical area and the recharge source, i.e., the distance between the southeast corner of the diesel generator building and the southwest corner of the circulating water intake structure. However, Figure 24-1 shows this distance to be about 240 feet. Use of this smaller value for x will also result in a rebound time shorter than the 90 days which you have computed.

(1) Please justify or correct the above apparent discrepancies and, if appropriate, provide revised analyses to better define the rebound time to be expected following a prolonged dewatering system failure. A more conservative analysis might involve utilizing the recovery data from the appropriate pump tests, i.e., K = 31 fps.

(2) In determining rebound time, it is our position that you should also postulate failure of non-Seismic Category I piping at critical locations. This should include the circulating water conduits.

(3) Demonstrate that there remains adequate time to install and implement a back-up dewatering system to prevent groundwater from rising above elevation 610 feet.

50. Your Response to Request 24 concludes that there is groundwater recharge from the cooling pond in the area of the intake and pump structures because pumping tests at well PD-15A resulted in very lettle drawdown at observation wells SW-1, SW-4 and RR-1. However, for several indicated reasons, you also concluded that there is very little recharge in the area of the discharge structure and one of these reasons is that there is very little drawdown at observation wells PD-3 and PD-20B as shown by Figure 24-14. These appear to be contradictory conclusions (i.e., how can very little drawdown indicate recharge at one location and no recharge at another nearby location?). Provide additional information to support and clarify your conclusion that there is negligible recharge in the area of the circulating water discharge structure. (Also see related Request 47(2)).

51. Your response to Request 24 regarding the area well dewatering system concludes that 22 wells pumping at an average rate of 5 gpm would be needed to remove groundwater stored within the backfill and natural sands. Two more wells are provided for infiltration and pipe leakage. You have not demonstrated whether 24 wells would also be a sufficient number to maintain the area groundwater at the desired elevation following removal of the groundwater already in storage. Provide

additional information to demonstrate that 24 wells will maintain groundwater levels below elevation 610 feet and provide the design basis used for this determination. Additionally, justify your use of 14 percent for an average Significant Yield Coefficient.

52. Your response to Request 24 discusses the source of groundwater which you have determined from pumping tests in the vicinity of the Service Water Pump Structure and the Circulating Water Intake and Discharge Structures. However, no tests appear to have been conducted to determine if Dow Chemical's Tertiary Water Treatment Pond, shown on FSAR Figure 2.1-1A and located just west of the nuclear plant, represents a potential source of groundwater. We are aware of your conclusion that inflow of groundwater from outside the plant area is precluded by the cooling pond dike which encompasses the nuclear plant site; however, you have provided no information to support this conclusion with respect to the Dow pond. Also lacking is information on the details of your West Plant Dike shown on FSAR Figure 2.5-46. Provide information to demonstrate whether the Dow pond is or will be a source of groundwater at your plant site. As a minimum, include the following:

(1) Provide a general description of the Dow pond (size, depth, capacity, purpose, contents, sealing method, etc.). Specify maximum elevation of the water in the Dow pond with relationship to the groundwater levels below the plant. Include a sketch showing distances and elevations of the Dow pond relative to the West Plant Dike.

ils on your West Plant Dike. Compare the West Plant
cooling pond dike, including any similarity in their
onstruction and their source of construction materials.
hat plant excavation extended to the area where the
ike is located; discuss whether and how excavation for
fected construction of the West Plant Dike.
ilt drawings of the West Plant Dike.
esults of any tests conducted to reach a conclusion
of the Dow pond on the groundwater beneath the

d is a potential source of groundwater, provide
e chemistry of this water (both present and future)
ts effects on the dewatering system and other under-
nts (piping, tanks, etc.). Identify any agreements
ave to monitor and control the contents or influence
d during plant operation.
ater elevations in the warehouse area which is
the Dow pond and the West Plant Dike.

the interceptor well system design in response to
that seepage would flow into a 400 foot slot located
oling pond. You assumed that part of this slot
because the intake and pump structures would cut off
from the cooling pond. To account for this cut off,
slot would be located 450 feet from the cooling
feet. This assumption reduced the quantity of inflow

Figures 24-9 and 24-10 indicate that 5 to 10 feet of natural sand exists below the intake and pump structures (See Request 47(3)). Consequently, these structures may not cut off or reduce the seepage from the cooling pond. You should therefore recompute total groundwater inflow without any reduction for the structures and recompute the number of interceptor wells required. Reposition and space wells accordingly. Alternately, provide additional information to support your conclusion that the structures serve as positive cut offs.

Consumers Ex # 18...
10/8/80 (HOOD) C

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

Hood

JAN 3 1980

Docket Nos. 50-329/330

MEMORANDUM FOR: Roger Fortuna, Assistant Director for Investigation, OIA

FROM: Harold D. Thornburg, Director, Division of Reactor Construction Inspection, IE

SUBJECT: INFORMATION PERTINENT TO THE MIDLAND ORDER TO MODIFY THE CONSTRUCTION PERMIT INCLUDING THE MATERIAL FALSE STATEMENT CONSIDERATIONS

We are enclosing several documents which present the facts and issues involved in the Midland soils and foundation problems that were identified when excessive settlement was observed in the diesel generator building. The resolution of these problems and actions taken have been a joint IE and NRR effort which culminated in the Order to Modify the license on December 6, 1979.

Part of the efforts involved in these problems was the consideration given to several items which were being reviewed as possible material false statements.

Enclosure 1 is a listing of the pertinent documents that relate to this matter. Those noted with an asterisk reflect what we consider to be the key documents you may want to focus on first to define what the issues were. If you need additional information on this matter, please contact us.

Harold D. Thornburg
Director
Division of Reactor
Construction Inspection
Office of Inspection and Enforcement

Enclosures:
1. Documentation List on the Midland Soil/Foundation Problems
2. Attachments listed on Enclosure 1

CONTACT: R. E. Shewmaker, IE
49-27551

~~8002120130~~

cc/w enclosure 1:
H. Denton, NRR
E. Case, NRR
R. Mattson, NRR
D. Vassallo, NRR
S. Varga, NRR
J. Knight, NRR
L. Rubenstein, NRR
W. Haass, NRR
D. Hood, NRR
J. Murray, ELD
W. Olmstead, ELD
J. Keppler, RIII
G. Fiorelli, RIII
G. Reinmuth, IE

DOCUMENTATION LIST
MIDLAND SOIL/FOUNDATION PROBLEMS

1. 10 CFR 50.55(e) reports. (Note: Large drawings are not included here due to reproduction problems. They are on file in IE.)

   a. Initial Report with letter dated 9/29/78
   b. Interim Report #2 with letter dated 11/7/78
   c. Letter dated 12/21/78
   *d. Interim Report #3 with letter dated 1/5/79
   *e. Interim Report #4 with letter dated 2/23/79
   *f. Interim Report #5 with letter dated 4/30/79
   *g. Interim Report #6 with letter dated 6/25/79
   h. Letter dated 8/10/79 with enclosure
   *i. Interim Report #7 with letter dated 9/5/79
   *j. Interim Report #8 with letter dated 11/2/79

2. Transfer of Lead Responsibility to NRR dated 11/17/78

3. Board Notifications

   a. Memo Olmstead to Vassallo, 11/3/78
   b. Memo Thornburg to Gower, 11/9/78
   c. Memo Bryan to Vassallo, 11/13/78
   d. Memo Vassallo to Engelhardt, 11/13/78

   e. Memo Keppler to Thornburg, 4/20/79
   f. Memo Thornburg to Thompson, 5/14/79
   g. Memo Thompson to Vassallo, 5/17/79
   h. Memo Vassallo to Christenbury, 5/29/79
   i. Memo Thornburg to Keppler, 6/5/79

4. IE Inspection Reports

   *a. 78-12, 11/17/78
   b. 78-13, 11/3/78
   c. 78-14, 11/9/78
   *d. 78-20, 3/22/79
   e. 78-22, 3/2/79
   f. 78-23, 3/36/79
   *g. 79-06, 4/9/79
   h. 79-08, 4/27/79
   i. 79-09, 5/8/79
   j. 79-10, 6/6/79
   k. 79-13, 5/30/79
   l. 79-15, 8/22/79
   m. 79-16, 7/9/79
   *n. 79-19, 10/1/79

5. Enforcement Actions

   *a. Memo Keppler to Thornburg, 2/15/79
   *b. RIII Position Paper, 2/23/79
   *c. Memo Keppler to Thornburg, 3/12/79

m Keppler to Howell of Consumers Power Company, 3/15/79
er to Thornburg, 4/3/79
burg to Thompson, 6/13/79
to File, 8/9/79
burg to Gower, 9/27/79
to Keppler, 10/4/79
er to Thornburg, 10/29/79

equest and Responses (Note: Large drawings are not included
duction problems. They are on file in IE.)

ton to Howell of Consumers Power Company, 3/21/79
ell to Denton, 4/24/79
ell to Denton, 5/31/79
ell to Denton, 7/9/79
enstein to Howell, 8/29/79
enstein to Howell, 9/11/79
ell to Denton, 9/13/79
ell to Denton, 11/13/79
enstein to Howell, 11/19/79

spondence

stein to Knight, 9/27/79
y from 7/18/79, dated 10/16/79
y from 9/5/79, dated 10/16/79
y from 11/14/79, dated 12/3/79

odify the Construction Permit, 12/6/79
of Amendment #72, 12/19/79 per the Order
r Hearing by Consumers Power, 12/26/79

Consumers Ex#19 i.d. 10/5/80 (Hood)

UNITED STATES
NUCLEAR REGULATORY COMMISSION
REGION III
799 ROOSEVELT ROAD
GLEN ELLYN, ILLINOIS 60137

April 3, 1979

MEMORANDUM FOR: Harold D. Thornburg, Director, Division of Reactor Construction Inspection, IE

FROM: James G. Keppler, Director

SUBJECT: ENFORCEMENT ACTION RE: MIDLAND DIESEL GENERATOR BUILDING AND PLANT FILL AREA

As you are aware, we have sent to Consumers Power Company a report on our two meetings held with them and a report of the investigation into the causes of the diesel generator building settlement. In my memorandum to you dated March 12, 1979, I summarized our findings and our concerns resulting from this investigation.

In view of NRR's involvement in the technical issues in this case, and the need for a determination as to the materiality of FSAR statements we consider to be false, we are not in a position at this time to recommend specific enforcement action which should be taken.

Attached to this memorandum are the specific FSAR statements and the basis for our conclusion that they are false. Also attached are copies of our letter dated March 22, 1979, which transmitted the Investigation report to the licensee and a draft Notice of Violation setting forth the items of noncompliance based on the investigation findings. The draft Notice of Violation includes all of the FSAR discrepancies described in Attachment 1 as examples of noncompliance with Criterion III of 10 CFR 50, Appendix B. If it is determined that any of these matters constitute material false statements, we assume they would then be treated separately, and removed as examples of noncompliance with this criteria.

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We request that the items of noncompliance be given technical and legal review and that a determination be made of the materiality of FSAR discrepancies so that upon resolution of the technical issues, we will be in a position to move more promptly toward taking enforcement action.

James G. Keppler
James G. Keppler
Director

Attachments:
1. FSAR False Statements
2. Draft Notice of Violation
3. Ltr dtd 3/22/79, with Investigation Report

cc w/attachments:
D. Thompson, IE

Midland FSAR Statements

1. Statement

Section 2.5.4.5.3, Fill, states: "All fill and backfill were placed according to Table 2.5-9."

Table 2.5-9, Minimum Compaction Criteria, contains 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."

Section 2.5.4.10.1, Bearing Capacity, states: "Table 2.5-14 shows the contact stress beneath footings subject to static and static plus dynamic loadings, the foundation elevation, and the type of supporting medium for various plant structures."

Table 2.5-14, Summary of Contact Stresses and Ultimate Bearing Capacity for Mat Foundations Supporting Seismic Category I and II Structures, contains, in part; the following:

"Unit	Supporting Soils
Diesel Generator Building	Controlled compacted cohesive fill.

Finding

Construction Drawing C-45, Class I fill material areas, specifies the foundation material for Class I structures to be Zone 2 material which is identified in FSAR Table 2.5-10, Gradation Ranges for Fill Material, as Random Fill and is described as "Any material free of humus, organic or other deleterious material." It was ascertained that materials other than "clay" or "controlled compacted cohesive fill" were used for support of structures.

2. Statement

Section 2.5.4.10.3.1, Plant Layout and Loads, states: "The building loads superimposed by the structures on undisturbed soil or compacted fill are given in the soil pressure plan, Figure 2.5-47."

Figure 2.5-47, Soil Pressure Diagram Category I and II Structures, shows the superimposed load density for the Diesel Generator Building to be 4.0 KSF (4000 lbs. per sq. ft.).

Finding

It was ascertained through a review of the settlement calculations and an interview of the individual who performed those calculations that 3.0 KSF was used.

3. Statement

Section 2.5.4.10.3.3, Soil Parameters, states: "The soil compressibility parameters used in the settlement calculation are presented together with soil profile in Table 2.5-16."

Table 2.5-16, Idealized Soil Profile and Parameters for Elastic Half-space Settlement and Heave Analysis, contains the following:

Layer	Idealized Soil Type	Elevation Interval (ft)	Thickness (ft)	Average $\frac{C_c \cdot r}{1+e_o}$ (1)
A	Fill (CL)	634-609	25	0.003
B	Fill (CL)	609-603	6	0.003

NOTE: Final groundwater table is taken at elevation 627.

(1) Values were estimated from the mathematical relationship between Young's Modulus and Compression and rebound indexes and averaged with those obtained from consolidation tests. Young's Modulus was estimated from empirical relationship with shear strength.

Finding

It was ascertained through a review of the statement calculations for the Diesel Generator Building and an interview with the individual who performed these calculations that an index of compressibility of 0.001 not 0.003, was used for the elevation interval 603-634.

4. Statement

Section 2.5.4.10.3.5, Analysis, states: "For settlement computations, a total of 41 settlement points are established on a grid and at selected structure locations as shown in Figure 2.5-48. . . . To account for possible time-dependent relationship, the estimated total settlements at each of the 41 points were obtained respectively by adding 25% of the calculated settlement values of loading Case A to the calculated ultimate settlement values of loading Case B. These values are presented in Figure 2.5-48."

Section 3.8.4.1.2, Diesel Generator Building, states: "The walls are supported by continuous footings with bases at elevation 628'-0". Each diesel generator rests on a 6'-6" thick reinforced concrete pedestal which is not structurally connected to the building foundation for purposes of vibration isolation."

Finding

It was ascertained through a review of the settlement calculations for the Diesel Generator Building and an interview with the individual who performed these calculations that the data in Figure 2.5-48 regarding the Diesel Generator Building are based on calculations performed on the erroneous assumption that the Diesel Generator Building was constructed on a mat foundation.

5. Statement

Section 3.8.5.5, Structural Acceptance Criteria, states: "Settlements of shallow spread footings founded on compacted fills are estimated to be on the order of 1/2 inch or less. These settlements are essentially elastic and occur as the loads are applied."

Finding

It was ascertained through an interview with the individual who wrote this section of the FSAR that the above statement was taken from the Dames and Moore report submitted as part of the PSAR. He assumed the statement was valid for inclusion in the FSAR. He said there was no other basis to support the statement.

(NOTE: In this regard the licensee has subsequently stated this statement ". . . is not applicable to the as-built configurations and conditions of the diesel generator building and has been eliminated from the FSAR in Revision 18.")