UNITED STATES OF AMERICA
NUCLEAR REGULATORY COMMISSION

BEFORE THE ATOMIC SAFETY AND LICENSING BOARD

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

TESTIMONY OF HARI NARAIN SINGH CONCERNING DIESEL GENERATOR BUILDING

Q.1. Please state your name and position with the U.S. Army Corps of Engineers.

A. My name is Hari N. Singh. I am a Civil Engineer in the Geotechnical Branch of the Engineering Division, NCD Chicago District of the U.S. Army Corps of Engineers.

Q.2. How did the U.S. Army Corps of Engineers get involved in the review process of the Midland Plant, and what are the areas of its responsibilities?

A. Pursuant to an interagency agreement between the U.S. Nuclear Regulatory Commission (NRC) and the U.S. Army Corps of Engineers (the Corps) which became effective in September 1979, the Corps undertook to provide technical assistance to the NRC. The Corps provides assistance on the geotechnical engineering aspects of the Midland Plant.

Q.3. Have you prepared a statement of your professional qualifications?

A. Yes, a copy is attached.

Q.4. Please state the nature of your responsibilities with respect to the Midland Plant.

A. My involvement with the Midland Plant began in May 1980, when I was assigned the responsibility as the Corps' lead reviewer for the geotechnical concerns at the Midland Plant. As lead reviewer, I worked with engineers and geologists in the Geotechnical Engineering Section of the Detroit District, who were engaged in reviewing the materials used in the foundation design of the plant. As the full-time lead reviewer, my responsibilities were to coordinate with all the reviewers, examine their comments, perform my own review, discuss comments with the Section Chief and prepare a final letter report to be transmitted to the NRC. The structures being reviewed include the following: 1) Auxiliary Building, 2) Reactor Building Units 1 and 2, 3) Diesel Generator Building, 4) Borated Water Storage Tanks Units 1 and 2, 5) Service Water Pump Structure, 6) Diesel Fuel Storage Tanks, 7) Seismic Category

I Piping and Conduits, 8) Retaining Walls, and 9) the dikes adjacent to the Emergency Cooling Water Reservoir (ECWR).

Q.5. What are the existing soil problems at the Diesel Generator Building Site?

A. (a) Settlement: The soil surface, supporting the Diesel Generator Building, has settled excessively as well as unevenly, causing warping of the footings and cracking of the building's walls. Further, whether or not the process of settlement is stabilized has not yet been determined; therefore, further propagation of the existing cracks and development of new cracks might continue jeopardizing the safety of the structure.

It began in August, 1978, when through the normal settlement monitoring program, it was discovered that the partially completed Diesel Generator Building had settled more than the expected settlement for the structure at that time. A preliminary investigation of the foundation soils, which consist of compacted fill materials, revealed that soils were inadequately compacted, and that the soils were heterogeneous in nature. They consisted of sands with relative density varying from loose to very dense and clay with consistency soft to very stiff. As a consequence of such poor soil properties, in some area under the structure, the foundation soils did not provide adequate support to the structure, resulting in excessive and uneven settlements. As of 19 January 1979, the corners of the east wall of the structure had settled approximately 50% more than their counterparts of the west wall, with maximum settlement of 4.25" at the southeast corner and minimum settlement of 2.09" at the northwest corner (See Attachment 1) and there was no evidence to indicate that the foundations have stabilized.

To accelerate the settlements under the existing loads, and to minimize them under the future loads (dead loads of additional construction, live loads, dewatering loads, etc.), so that necessary piping connections to the structure could be made with assurance that no overstressing in piping could occur due to future settlements of the building, the applicant surcharged the partially built structure and a portion of the surrounding areas with 2200 lbs per square foot of surcharge loading. The full surcharge remained in effect for 132 days, beginning on 6 April 1979.

The surcharge, as expected, produced additional consolidation in the fill materials which accelerated the settlements, but it raised questions; (1) whether the precompression stress produced by the surcharge would exceed the stresses that would be created by future loads, and as such any future settlement would be insignificant to the safety of the structure, (2) whether the rigidity of the structure prevented the surcharge loads to become effective in producing consolidation in areas of more compressible soil, and in future a redistribution of loads on the foundation surface is possible, and (3) whether the additional settlements created by the surcharge load (See Attachment 2 for settlements due to surcharge), have done any permanent damage to, or have induced stresses in the partially completed structure, and the piping underneath the structure, which would be detrimental to the ability of the structure to withstand severe future environmental loads (earthquake, tornado, etc.). The Corps of Engineers questioned the validity of the surcharge results, and in its report of 16 July 1980, which was transmitted to the applicant on 7 August 1980 by the NRC, requested the applicant to verify the field observed settlements by settlements computed on the basis of results of laboratory tests conducted on representative soil samples (details of request for soil explorations and testing given in item...). The Corps also requested the applicant in its report of 7 July 1980 and 15 April 1981, and at various meetings (structural audit in April 1981 at Ann Arbor, and in a meeting at Bethesda in first week of June 1981) to compute stresses in the foundation due to settlements. The above requests would have provided answers to the three questions, but as of today, no response to the above request has been received and as such, the Corps of Engineers is not in a position to complete its review and testify regarding the adequacy of the Diesel Generator Building.

Q.6. What are the effects of the past settlements and the settlements created by the surcharge?

A. Structures founded on soil mass settle to a degree depending upon the compressibility and uniformity of the soil mass under the foundation. Before building a structure, soil explorations and testings are carried out to determine soil characteristics, which form the basis to determine the most suitable location for the structure and to proportion its foundation. One of the main purposes of exploration and testing is to enable the engineer to select the site which will cause minimum settlement so that no additional consideration for settlements is needed in design.

The settlements of the foundation soil under the Diesel Generator Building have exceeded the expected limits of settlements. Because of the structure being rigid, approximately uniform settlements were expected under the building. However, the settlements observed prior to the surcharge indicate uneven settlements creating differential settlements resulting in curvatures. Consequently, additional flexural and shear stresses have been induced in the structure. Subsequent to the surcharge, the magnitude of the settlements further increased and the curvatures of the footings in some area increased causing further increase in bending and shear stresses. Attachment 3 shows a qualitative assessment of increase in curvature of the footings under the east wall of the Diesel Generator Building. The wall supported by this footing has shown considerable increase in number of cracks, since the surcharge load was applied (number of cracks prior to surcharge 10, as per response to question 14, 10 CFR 50.54 (f), Figure 142, number of cracks since surcharge 16). The additional curvature created by the surcharge appears to be a major factor in creating these cracks.

The Corps of Engineers, in its reports of 15 April 1981, indicated that an analysis of stresses induced by the warping should be performed taking into account the differential settlements over the life span of the plant (40 years). As of this date, the applicant has not furnished the requested analysis.

Q.7. What are the results of soil exploration and testings (July 1981 reports of Woodward-Clyde consultants and Dr. Peck's enclosures)?

A. In response to the Corps of Engineers request for soil exploration and testing, the applicant retained the Woodward-Clyde consultant to perform borings and testings. The consultant performed soil exploration in the areas designated by the Corps of Engineers around the Diesel Generator Building, conducted laboratory testings and presented the exploration and test results in a two volume report. In early August 1981, the applicant transmitted to the Corps of Engineers a copy of the report and a copy of its evaluation by Dr. Ralph Peck. Part one of the report contains the following:

(1) Boring logs for 12 borings advanced by the Woodward-Clyde consultant.
(2) Index properties test results.
(3) Particle-size distribution curves.
(4) Shear strength test results.
(5) Consolidation test results.
(6) Supporting data for CIU triaxial tests.
(7) Supporting data for CAU triaxial tests.
(8) Supporting data for consolidation tests.

Part two of the report contains the Index properties of the consolidation tests specimens (Tables 1&2), the maximum past consolidation pressure (Table 3) reportedly computed independently by three different engineers of the Woodward-Clyde consultants' staff. Also contained in part two are; (a) the results of the preconsolidation pressures (Table 4) computed by the three engineers using the results of the consolidation tests carried out by the Goldberg - Zoino - Dunnicliff (GZD) in 1978; (b) Graphical comparison of the precompression pressures with the actual pressures at various depth below the ground surface (Figure 3 & 4); and (c) strains - log P curves for the Woodward-Clyde test as well as GZD tests (Appendix A and Appendix B).

Q.8. Did you evaluate and draw your own conclusions on Woodward-Clyde report and Dr. Peck's evaluation of the test results? If yes, then what are your comments?

A. The Corps of Engineers has reviewed the results of the exploration, testing and precompression pressures provided in the Woodward-Clyde Consultants' report, and Dr. R.B. Peck's evaluation of the test results provided in a separate volume. The following are the review comments:

(1) Corps of Engineers' representatives observed the soil exploration program carried out by the Woodward-Clyde Consultant and found that it had been carried out in accordance with the state-of-the-art method. Drilling operation, taking samples from

ground, logging visual classifications, recording readings from the various gages on the drill-rig, handling of samples, transportation to the testing laboratory and extrusion of the samples from the tubes, etc. were carried out by experienced drillers, geologists and lab technicians. The Corps of Engineers is satisfied by the soil exploration program.

(2) The drained shear strength parameters ($\bar{\phi}$, $\bar{c}$) determined by the consolidated undrained tests with pore pressure measurement ($\overline{CIU}$, $\overline{CAU}$) and presented in Tables D-1, D-2 are better than those used by the applicant in its computation of the bearing capacity analysis, which was submitted by the applicant in response to Question 40(2), 10CFR 50.54(f). However, in its response to Question 40(2), the applicant has not demonstrated that shear strength parameters, $\bar{\phi}$ = 29.2° and $\bar{C}$ = 114 lbs per square foot, used in its analysis were the representative parameters for the soil underneath the Diesel Generator Building.

(3) The results of the consolidation tests indicate that all the tests were carried out to a maximum consolidation stress of 64 tons per square foot (tsf). The maximum past consolidation pressure (preconsolidation pressure) have reportedly been computed by three engineers independently on the basis of the tests carried to 64 tsf consolidation pressure as well as on the basis as if the tests stopped at 16 tsf consolidation pressures. The computed preconsolidation pressures are tabulated in Table 3 of part 2. The results indicate that preconsolidation pressure computed on the basis of the consolidation tests carried to 64 tsf stress are not consistent with the settlements that have occurred under the foundations of the Diesel Generator Building prior to surcharging. For example, the computed preconsolidation pressures for eight samples taken from COE-13A and COE-13B varies from 1.48 tsf to 5.20 tsf with an average of 3.41 tsf. Excluding the effects of 1.1 tsf of surcharge and approximately 1.05 tsf of overburden (overburden pressure at mid-depth of the clay column at COE-13A, with average soil density of 140 lbs/cf), the net average precompression prior to surcharge turned out to be 1.2. tsf. With this preconsolidation pressure in the clay soil at COE-13 and its close vicinity, the south-east corner of the Diesel Generator Building, any settlement caused by a foundation pressure of 1.2. tsf and less would have been negligible, being the results of precompression. However, field measurements ha~~s~~ve indicated that the southeast corner of the building had settled 4.25" (See Attachment-1) under a foundation pressure of 0.7 tsf (See Attachment-2, Fig. 4-A, 10CFR 50.54(f)). Thus the observed settlement of 4.25" under a pressure 0.7 tsf at the southeast corner is inconsistent with the preconsolidation pressure computed on the basis of consolidation test carried to 64 tsf consolidation pressure.

(4) The e - log $\bar{p}$ curves for the samples, that show high precompression pressures at 64 tsf maximum consolidation pressure,

appear to have been affected by factors other than consolidation, such as high elastic deformation and some crushing of sand particles. Therefore, portions of the e - log $\bar{p}$ curves influenced by these non-consolidation factors should not be considered in computing preconsolidation pressure.

The Corps of Engineers replotted the e - log $\bar{p}$ curves (Attachment 5) for boring COE 13A and COE 13B at a larger scale than those used in the Woodward-Clyde report. These curves provided somewhat better perception of the behavior of the curves; the points of maximum curvature were more perceptible, the straight line portions of the curves were more defined, and with some curves a trend of increasing curvature at larger consolidation pressure was noticed. Of the eight samples tested for which preconsolidation pressures were computed on 64 tsf and 16 tsf maximum consolidation pressures, four samples (S-1B, S-3D, S-4B, S-9B) showed practically no change in their preconsolidation pressures computed on the above two basis. The remaining four (S-3C, S-6C, S-8B, S-5C) showed considerable variation in their precompression pressure under the two testing conditions with higher values at 64 tsf. A close review of the curves indicates that curves for the later group show an unusual behavior. Two of the specimen (S-3C, S-5C) show increase in curvature, and the other two (S-6C, S-8B) show constant curvature at higher pressure after showing a trend of decrease at gradually increasing pressure from 0.0 to somewhere between 16 tsf to 32 tsf. This may be due to high elastic strains and some crushing of the sand components which constitutes approximately 40% of the soil. The high rebound at 64 tsf indicates influence of high elastic strain after 16 tsf. The curve, beyond the point at which increase in curvature begins, does not represent consolidation; the change in void ratio might be the results of the sliding as well as crushing of the particles and high elastic strains. The Corps of Engineers is of the opinion that the portions of the e - log $\bar{p}$ curves showing increasing curvature should not be considered in computation of preconsolidation pressure. In cases of curves with constant curvature after some specific value of consolidation stress, the initial portion of the curve with constant curvature should be used in computing preconsolidation pressure.

(5) In paragraph 3 of page 3 of Dr. R.B. Peck's evaluation, it has been concluded that preconsolidation pressures for the surchaged clay of Boring 9 (COE-9) were substantially greater than those determined by means of sampling and testing. This conclusion was reached on the basis of information obtained from pocket penetrometer, verbal description of soils and a empirical equation $\frac{c}{pn} = .10 + .004Ip$. In the opinion of the Corps of Engineers, soil information obtained by proper sampling and testing, as in the case of the soils in this discussion, are more reliable than those obtained on the basis of the index properties and verbal soil description. The three factors used by Dr. Peck provide only rough guidance to engineers and cannot be relied upon. The results obtained using these factors could very well be used to design an ordinary structure, but for a Category I structure of a nuclear power plant. It is not advisable to depend on them. The value obtained by actual test should be used.

The Corps of Engineers computed and compared c_{pn} values obtained from the empirical equation (c_{pn} = .10 + .004Ip) and from actual laboratory tested values, using test data provided in C of E Engineering Manual EM-1110-2-5008 dated 15 October 1980 (See Attachment). The empirical equation provided a c_{pn} value approximately 45% higher than actual value. Therefore, it is concluded that the empirical equation (c_{pn} = .10 + .004Ip) provides very approximate values of c_{pn} and cannot be used with confidence in important structures such as the Diesel Generator Building.

The test results obtained from Pocket Penetrometers are not reliable; they provide some guidance for visual classification during exploration. Messrs M.G. Spangler and R.L. Handy have stated on page 101 of their book "Soil Engineering", "The pocket penetrometer is sometimes used on drive samples obtained from standard penetration test, but little or no reliance should be placed on such tests." Therefore, the Corps of Engineers does not concur with Dr. Peck's conclusions obtained using Pocket Penetrometer values to evaluate the test results.

The verbal description of soil, normally done during soil exploration, is not an accurate method of determining the engineering properties of soils. Consistency of clay determined by visual inspection or by pocket penetrometer tests are not reliable. This is because the pocket penetrometer values are not reliable and the results of the visual classifications may vary from one individual to the other. Thus, to rely on the consistency of soil recorded by verbal description as described for samples of COE-9 is not a sound engineering practice. Therefore, the shear strength inferred from the visual soil classification as used on page 3 of Dr. Peck's evaluation is not justified.

(6) In paragraph 4 of page 2 of his evaluation, Dr. Peck has evaluated the accuracy of the preconsolidation pressure computed for three samples taken from three different elevations of a 7' high clay column. The computed preconsolidation pressure for the sample located in the middle of the 7' high clay column is lower than that for samples located near the top and bottom of the column. Quoting Dr. Peck, he has concluded, "In reality, the preconsolidation pressure must have been nearly identical at all three points, unless the fill was extremely heterogeneous. The latter conclusion is not born out by the detailed log of Boring 12A. Therefore, one must conclude that the preconsolidation pressure determined for the sample at intermediate is too low. The most conservative interpretation would place the preconsolidation pressure for intermediate point at value greater than 2.1 tons per square foot, the least value estimated by any of the three engineers for overlying sample."

The Corps of Engineers disagrees with Dr. Peck's conclusion. The lower preconsolidation pressure at the intermediate point might have been caused due to a variation in compactive efforts during compaction of the fill material. It has already been established that inadequate compaction has caused the settlements under the structure. The soil layers in the top and the bottom of the 7' clay column might have been compacted better than those in the middle, which caused difference in preconsolidation pressure.

(7) Referring to paragraph 3, page 2 of Dr. Peck's evaluation; the Corps of Engineers agree that strain - log p curves are smooth curves, without obvious breaks between flatter upper and steeper lower branch. But the point of maximum curvature can be determined within reasonable accuracy if the curves are plotted on somewhat larger scale than that used in Woodward-Clyde report. No doubt, there will be some variation in choice of point of maximum curvature, but the margin of interpretation shown by the three engineers is too large. The Corps of Engineers' evaluation of precompression pressures for samples of COE-13 is given and compared in Attachment 6 with those provided in Woodward-Clyde report. The Corps' values are consistently less than those of Woodward-Clyde values.

(8) Conclusions:

(a) Shear strength parameters determined for the foundation soils under the Diesel Generator Building are more reliable based on test data than those previous arbitrarily assumed parameters used in bearing capacity analysis and furnished in response to Question 40 (10CFR 50.54(f)). Therefore, the bearing capacity of the foundation soils is adequate.

(b) The precompression pressures, computed on the basis of the consolidation test results obtained after extending the tests to full 64 tsf consolidation pressure, are not valid in all cases, because the e - log p curves for these cases show an increase in curvature at higher pressures, a behavior not expected in consolidation of soils. Also, the inconsistency between settlements and preconsolidation pressures described in paragraph 3 substantiates the fact that precompression stresses provided in report are not accurate. Therefore, the preconsolidation pressures computed and reported in the Woodward-Clyde report are not acceptable.

(c) The precompression pressures for many samples, for example, samples of Boring COE-9, have indicated that the preconsolidation pressures are less that the total design external pressure (stresses due to dead load, semi-permanent loads, etc.), therefore, some additional settlements should be determined and be used in determination of stresses in the structure.

(d) The applicant has not yet furnished any information that indicates that it has determined the stresses in the structure incorporating the effects of total settlements (the settlements that has already occurred, future primary and secondary settlements).

(e) The settlement stresses are permanent in nature and as such are equivalent to the stresses produced by dead loads. Therefore, in checking structure stresses in various load combinations it must be considered as dead load stress.