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Exhibits from

10/9/80 Deposition

of

Lyman Heller

Exhibits 1-36

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Consumers Power Co Ex #1 - Heller deposition

PROFESSIONAL QUALIFICATIONS SYNOPSIS

U.S. NUCLEAR REGULATORY COMMISSION

LYMAN W. HELLER, Leader, Geotechnical Engineering Section

My name is Lyman Wagner Heller. I presently reside at 18605 Rolling Acres Way, Olney, Maryland 20832, and am employed as Section Leader, Geotechnical Engineering Section, Hydrologic and Geotechnical Engineering Branch, Division of Engineering, Office of Nuclear Reactor Regulation, U.S. Nuclear Regulatory Commission, Washington, D.C. 20555.

I received Bachelor of Science degrees in Agricultural Engineering and Civil Engineering from the University of Illinois in 1950 and 1957, respectively. I received Master of Science and Doctor of Philosophy degrees in Civil Engineering, with majors in soil and foundation engineering, from the University of Florida in 1959 and 1971, respectively.

Since joining the AEC (now NRC) in February of 1974, I have reviewed or participated in the review of the geotechnical features of about 25 power plants and other nuclear facilities. Prior to my present position, which I assumed in December, 1974, I was employed for 9 years as Chief of the Analytical Section, Soil Dynamics Branch, Soils Division at the Waterways Experiment Station, U. S. Army Corps of Engineers. In this position, I was responsible for special analytical and experimental Corps studies in soil and foundation dynamics as well as earthquake engineering aspects of earth and rock-fill dams. The results of these studies have been published as Corps reports and as papers in national and international symposia and proceedings. Prior to my employment with the Corps of Engineers, I was employed for 6 years as a Research Civil Engineer in the Soils and Pavements Division, Civil Engineering Department, Naval Civil Engineering Laboratory, Bureau of Yards and Docks, Department of the Navy. In this position, I was responsible for soil and foundation studies related to buried protective structures to resist the effects of nuclear weapons as well as design criteria for piles and other waterfront foundations. My other professional experience includes University teaching appointments, from Instructor to Adjunct Professor, employment with a consulting engineering firm, and employment as a project and product engineer in industry.

I am a member of the American Society of Civil Engineers and Sigma Xi-Scientific Research Society of America. I have been a registered professional engineer in the State of Florida since 1959.

Lyman Heller 10/7/80

A Name and address of employer's organization (include ZIP Code, if known)

U. S. Nuclear Regulatory Commission
Washington, D. C. 20555

Dates employed (give month and year)
From 12/74 To Present

Duties: Supervise the review and evaluation of geotechnical aspects of nuclear plants and fuel facilities conducted by section staff and consultants. Prepare budgets, manpower requirements, and personnel actions. Engage consultants and write and monitor contracts with experts and other government agencies. Perform particularly difficult reviews of complex problems. Prepare reports and testimony for licensing committees and boards. Testify at hearings and before appeal boards. Responsibilities: Make decisions on the safety and adequacy of power plant foundations, earth dams and the stability of mine tailings dams.

For agency use (skill codes, etc.)

B Name and address of employer's organization (include ZIP Code, if known)

U. S. Atomic Energy Commission
Washington, D. C. 20555

Dates employed (give month and year)
From 2/74 To 12/74

Duties: Evaluated proposed sites, foundations, and dams for nuclear reactors and nuclear facilities with respect to the geotechnical engineering features to assure that adequate design and construction measures are taken to prevent adverse safety problems. Develop criteria and standards. Prepared reports and gave testimony to committees and hearing boards. Write contracts. Responsibilities: Made recommendations for accepting, rejecting or modifying proposed designs for foundations and geotechnical features of nuclear facilities. Accomplishments: About 5 plants were reviewed from 1974 to 1975. Consultant contracts were written and implemented.

For agency use (skill codes, etc.)

C Name and address of employer's organization (include ZIP Code, if known)

U. S. Army Engineer Waterways Experiment
Station, P. O. Box 631
Vicksburg, Mississippi 39180

Dates employed (give month and year)
From 6/55 To 2/74

Duties: Supervised work of the Analytical Section, Soil Dynamics Branch, Soils and Pavements Division. Monitored contracts for research activities by industry and government agencies. Carried out complex research and development studies in soil dynamics, foundation dynamics and earthquake engineering. Performed analyses of ground motion, foundation-structure interaction, and the earthquake resistance of earth and rock-fill dams. Initiated personnel actions. Responsible for direction and application of research work.

For agency use (skill codes, etc.)

Lyman Heller *10/7/80*

U. S. Naval Civil Engineering Laboratory *2/65 - 6/65*
Port Hueneme, CA

Duties: Performed complex research activities on projects involving soil mechanics, foundations, and related disciplines as applied to problems of buried protective structures subjected to dynamic loading by the effects of nuclear weapons. Monitored contracts. Responsible for validity of analytical techniques and data and application of results. Accomplishments: Reports of research studies were issued. Scaled field blast load tests were evaluated.

For agency use (skill codes, etc.)

Name and address of employer's organization (include ZIP Code, if known)

U. S. Naval Civil Engineering Laboratory *10/60 - 2/65*
Port Hueneme, CA

Duties: Worked as a Senior Project Engineer in the Soils and Pavements Division of the Laboratory on research tasks concerned with the study, interpretation and prediction of ground motion and its effects on buried protective structure. Performed research, analysis, and developed design criteria for laterally loaded piles. Responsible for planning research work, obtaining consultants and monitoring contracts and issuing reports to be developed into design manuals. Accomplishments: Reports were issued and two U. S. patents filed.

For agency use (skill codes, etc.)

U. S. Naval Civil Engineering Laboratory From 6/59 To 10/60
Port Hueneme, CA

Duties: Worked as a Project Engineer in the Soils and Pavements Division of the Laboratory. Planned and conducted research tasks involving the soil-structure interaction behavior of laterally loaded piling. Supervised field and laboratory tests. Responsible for analyzing data, monitoring contracts, and writing reports of findings, including recommendations. Accomplishments: Work left by previous project engineer was continued and coordinated successfully with him.

For agency use (skill codes, etc.)

G Name and address of employer's organization (include ZIP Code, if known)

Dates employed (give month and year) From 1/59 To 5/59

Department of Engineering Mechanics
University of Florida
Gainesville, Fl

Duties: Taught third-year engineering students an engineering mechanics course entitled "Strength of Materials." Participated in faculty conferences and committee work. Responsible for planning and delivering classroom lectures, preparing examinations and awarding final grades. Accomplishments: About 50 students were trained in this one-semester course.

Lymon Heller 10/7/80

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Graduated from high school June, 1946

Entered University of Illinois February, 1947

Graduated, B.S. in Agricultural Engineering (Machine Design Opt) June 1953

Employed by John Deere Ottumwa Works Ottumwa, Iowa.
June, 1950 to January 1951
Product engineer - Designed and detailed farm machinery (hay rake, baler motor, forage crop pick-ups)

Drafted into U.S. Army January, 1951
Basic Training - Fort Leonard Wood, Mo. (16 weeks)
Assigned 3rd Army Hdqtrs Atlanta, Ga. Worked in Mapping and Planning Office.
Assigned to Fort Belvoir Virginia. Worked in the Office of the Chief of Engineers, Cravelly Point, Va. on Ordinance Equipment and Vehicle Compatibility (Procurement and Supply).
Discharged at Fort Belvoir Virginia, January 1953.

Employed by Herman Nelson Corporation, Moline, Illinois, (Later purchased by American Air Filter Corporation, Louisville), February 1953 to (August), 1954 Worked as a product engineer on heating and ventilating equipment (Unit

Lyman Heller 10/7/8.

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and production coordination along with lab input.

Entered University of Illinois Law School September 1954.

Withdrew University of Illinois Law School October, 1954.

Employed by Bendix Aviation Corporation, Davenport, Ia. as project engineer October 1954 - August 1955 worked on design and detailing of oxygen breathing apparatus, experimental model check out.

Entered University of Illinois College of Engineering, Civil Engineering, September 1955

Worked part time for Consulting Engineering Firm, Clark Daily and Dietz while in school (Urbana, Ill.) worked on bridges, pile cap, retaining wall for interstate highway interchanges.

Worked part of one summer (1956) for County Engineer's office, Tuscola, Illinois (Douglas County ? - Wendy Davis?) Helped survey oil well locations on elevations, sidewalk, street and gutter work, quantities, asphalt, bridge pile foundations (timber)

Graduated U/Ill B.S. Civil Engineering, June 1957.

Worked summer at Clark Daily & Dietz Urbana, Illinois

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Entered University of Florida Graduate School Sept. 1957
Worked half time as Teaching Assistant in
Department of Civil Engineering. Taught Civil Eng.
Drafting and Strength of Materials Laboratory.

Worked summer of 1958 at the Army Engineer
School, Fort Belvoir, Va. Wrote Series 10
correspondence course on Reinforced Concrete.

Graduated with M.S. in Engineering, January, 1959.

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Publications

Technical Report R-[illegible], "Effects on Structural and Harbor Installations of Ground Shock Induced by Underwater Nuclear Explosives," 30 June 1963. Published by U. S. Naval Civil Engineering Laboratory, Port Hueneme, California. SECRET.

Technical Report R-[illegible], "Failure Modes of Impact-Loaded Footings on Dense Sand," [illegible] January 1964. Published by U. S. Naval Civil Engineering Laboratory, Port Hueneme, California.

Technical Report R-[illegible], "Lateral Thrust on Piles," 15 June 1964. Published by U. S. Naval Civil Engineering Laboratory, Port Hueneme, California.

Discussion of paper, "Lateral Resistance of Piles in Cohesive Soils," by Bengt B. Broms. Published in Journal of the Soil Mechanics and Foundations Division, ASCE, November 1964.

Technical Report R-[illegible], "Motion of Subsurface Inclusions Subjected to Surface Blast Loading," April 1965, Published by U. S. Naval Civil Engineering Laboratory, Port Hueneme, California.

Technical Note N-[illegible], "Failure Modes of Sand Surrounding a Laterally Displaced Pile Subjected to Static and Impact Loads," July 1965. Published by U. S. Naval Civil Engineering Laboratory, Port Hueneme, California.

Technical Report R-[illegible], "Motion of Subsurface Soil Inclusions Subjected to Surface Blast Loading-Results of Series II Tests," October 1966. Published by U. S. Naval Civil Engineering Laboratory, Port Hueneme, California.

Miscellaneous Paper No. 4-881, "Kaiser Landing Mat Failure Study (MX-19)", March 1967, Published by U. S. Army Engineer Waterways Experiment Station, Vicksburg, Mississippi.

Miscellaneous Paper No. 4-[illegible] (with Y. K. Park), "Gelatin Modeling Methods for the Study of Foundation Dynamics, Ground Motion, and Seismic Phenomena," February 1968. Published by U. S. Army Engineer Waterways Experiment Station, Vicksburg, Mississippi.

Symposium paper, (with R. A. Weiss), "Ground Motion Transmission from Surface Sources," a paper selected, presented and published in the Proceedings, International Symposium on Wave Propagation and Dynamic Properties of Earth Materials, 21-25 August 1967, Albuquerque, New Mexico.

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"A Comparative Summary of Current Earth Dam Analysis Methods for Earthquake Response," issued by Office, Chief of Engineers, as Enclosure 1 to Engineer Technical Letter No. 1110-2-77, 9 December, 1969.

"Earthquake Studies for Earth and Rockfill Dams," issued by Office, Chief of Engineers, as Engineer Technical Letter No. 1110-2-79, 12 January 1970.

Technical Report S-71-14, "Earth Vibration Effects and Abatement for Military Facilities, Report 1, Site Selection for Ground Motion Studies," November 1971, U. S. Army Engineer Waterways Experiment Station, Vicksburg, Mississippi.

Symposium paper, "Finite Element Analysis of the Response of Rifle Gap Dam to the Effects of Rulison Underground Detonation," presented and published in the Proceedings of the Symposium on the Application of the Finite Element Method in Geotechnical Engineering, May 1972, U. S. Army Engineer Waterways Experiment Station, Vicksburg, Mississippi.

Meeting paper, "The Particle Motion Field Generated by the Torsional Vibration of a Circular Footing on Sand," presented at the Annual and National Environmental Engineering Meeting, American Society of Civil Engineers, 16-22 October 1972, Houston, Texas.

Meeting paper, "Seismic Effects on an Earth Dam from an Explosion," presented and published at the National Structural Engineering Meeting, American Society of Civil Engineers, 9-13 April 1973, San Francisco, California.

Conference paper, "Earth Dam Motion due to a Deep Nuclear Explosion," presented at the Fifth World Conference on Earthquake Engineering, June 25-29, 1973, Rome, Italy.

Discussion (with R. G. Easterling) of paper, "Statistics of Liquefaction and SPT Results" by Christian et al, Published in Journal of Geotechnical Engineering ASCE, October, 1976.

Discussion of paper, "Sand Liquefaction in Large Scale Simple Shear Tests," by De Alba et al, Published in Journal of Geotechnical Engineering, ASCE, July, 1977.

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Patents

(1) "Gage for Measuring Strain and Strain Propagation in a Soil Material due to a Dynamic Load." (Patent No. 3,456,496)

(2) "Soil Testing Apparatus," (with H. L. Gill) (Patent No. 3,456,778)

Professional Memberships and Current Activities

American Society of Civil Engineers

Geotechnical Engineering Division, Committee on Soil Dynamics: Subcommittee (Organizing) member on the session "Soil Dynamics and Geotechnical Aspects of Nuclear Facilities Design" (April, 1979, Boston)

Geotechnical Engineering Division, Committee on Engineering Geology, Organizing committee on "Symposium on Capable Faulting (November, 1977, Seattle)

Technical Council on Lifeline Earthquake Engineering, Member of Advisory committee

Structural Engineering Division, Specialty Conference on "Civil Engineering and Nuclear Power" Chairperson of session on Soil-Structure Interaction, Knoxville, Tenn., Sept., 1980

International Association for Earthquake Engineering (non-Member)
Invited and contributed to International Workshop on Strong Motion Earthquake Instrument Arrays, Subgroup on Array Design for Local Effects (May, 1978, Honolulu)

National Science Foundation Workshop to define Research Needs and Priorities for Geotechnical Earthquake Engineering Applications: Chairman of Panel on Assessment of Seismic Stability of Soil (June, 1977, Austin, Texas)

National Science Foundation, Member of U. S. Soil Dynamics Delegation to the Peoples Republic of China - August-September 1979.

[redacted]

Sigma Xi, The Scientific Research Society of America
ERDA Club

Lymon Heller

Graduate and Short Courses

Technical Courses

Operations Research (1 semester) University of Southern
ornia, Spring, 1960.

Matrix Methods in Engineering (1 semester) University of
ornia, Spring, 1960.

Techniques for Measuring and Analyzing Random Data (4
UCLA, August, 1965.

Rock Mechanism Seminar (16 hours), Foundations Sciences
June, 1966.

Rock Mechanics in Civil Engineering (30 hours) University
Illinois, June, 1966.

Plasticity (1 semester), Mississippi State University,
1967.

Recent Developments in the Design and Construction of E
Rockfill Dams (40 hours) Universtiy of California,
1968.

Engineering Geology I (1 semester), Mississippi State U
Spring, 1968.

Seminar on Seismic Design for Nuclear Power Plants, (40
MIT, March, 1969.

Engineering Geology II, (1 semester), Mississippi State
Spring, 1969.

Design of Earth and Rockfill Dams, (1 semester), Mississ
University, Spring, 1970.

Earthquake Resistant Design of Engineering Structures,
University of California, June 1972.

Analysis and Design in Geotechnical Engineering, (40 hour
June, 1974.

Risk and Decision in Geotechnical Engineering (40 hours)
June, 1976.

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The Evaluation of Dam Safety (40 hours) Engineering Foundation Conference, November, 1976.

Geotechnical Engineering Conference, U. S. Army Corps of Engineers (40 hours), November, 1977.

<u>Management Courses</u>

Techniques of Managerial Communication (40 hours) U. S. Army Management Engineering Training Agency, February, 1967.

Basic Management Techniques I (40 hours) Civil Service Commission, October 1970.

Basic Management Techniques II, (40 hours) Civil Service Commission, March 1972.

Engineers - Motivating and Managing (16 hours) University of Wisconsin, January, 1973.

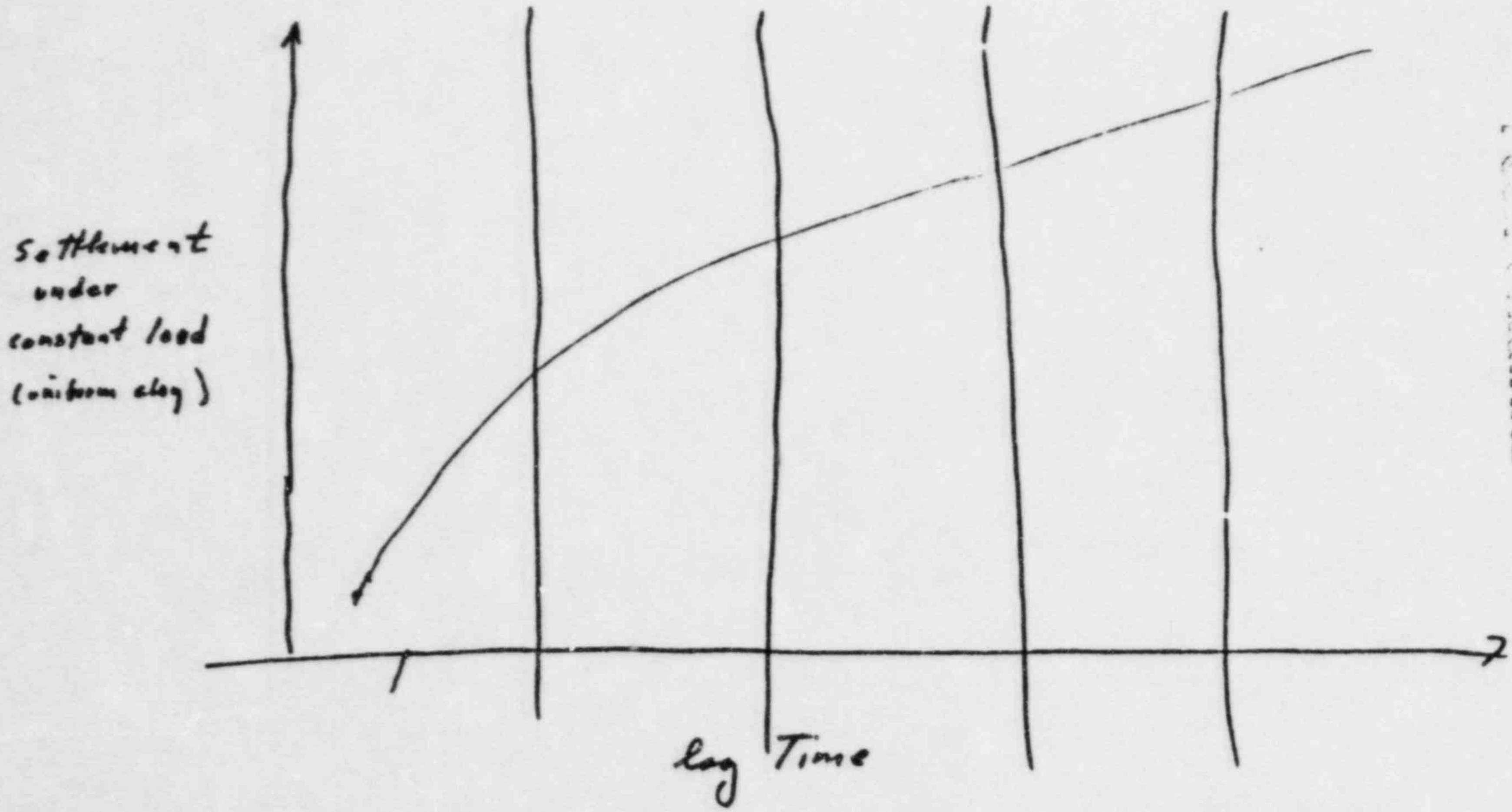
NRC Management Seminar (8 hours), Nuclear Regulatory Commission, July, 1976.

Management for Supervisors, (32 hours), Nuclear Regulatory Commission, September, 1977.

NRC Personnel Practices Seminar, (32 hours) Nuclear Regulatory Commission, October, 1978.

Systems Safety and Reliability Analysis Techniques (56 hours), Nuclear Regulatory Commission, November, 1979.

Consumers Power Co. Exhibit 2 - Heller Deposition dated 10/7/80



Consumers Power Co. Exhibit #[illegible]
Heller deposition — [illegible]

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.[1] 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.[4]

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.[4,18]

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.[11,8]

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.[1,2]

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.[1,2]

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.[4]

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[7]

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.

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.[7]

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.[8]

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.[illegible]

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.[illegible]

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.[illegible]

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.[illegible]

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.[illegible]

Inspections were performed by the fulltime subcontractor's inspector for lift thickness, proper material, roller passes, and moisture conditioning.[illegible] The inspector would call for field density tests after approximately every 500 cubic yards were placed to verify that proper placement was accomplished.[illegible] 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.[illegible]

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.[28]

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	30°	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.[1,10]

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 I

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 - 146	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)

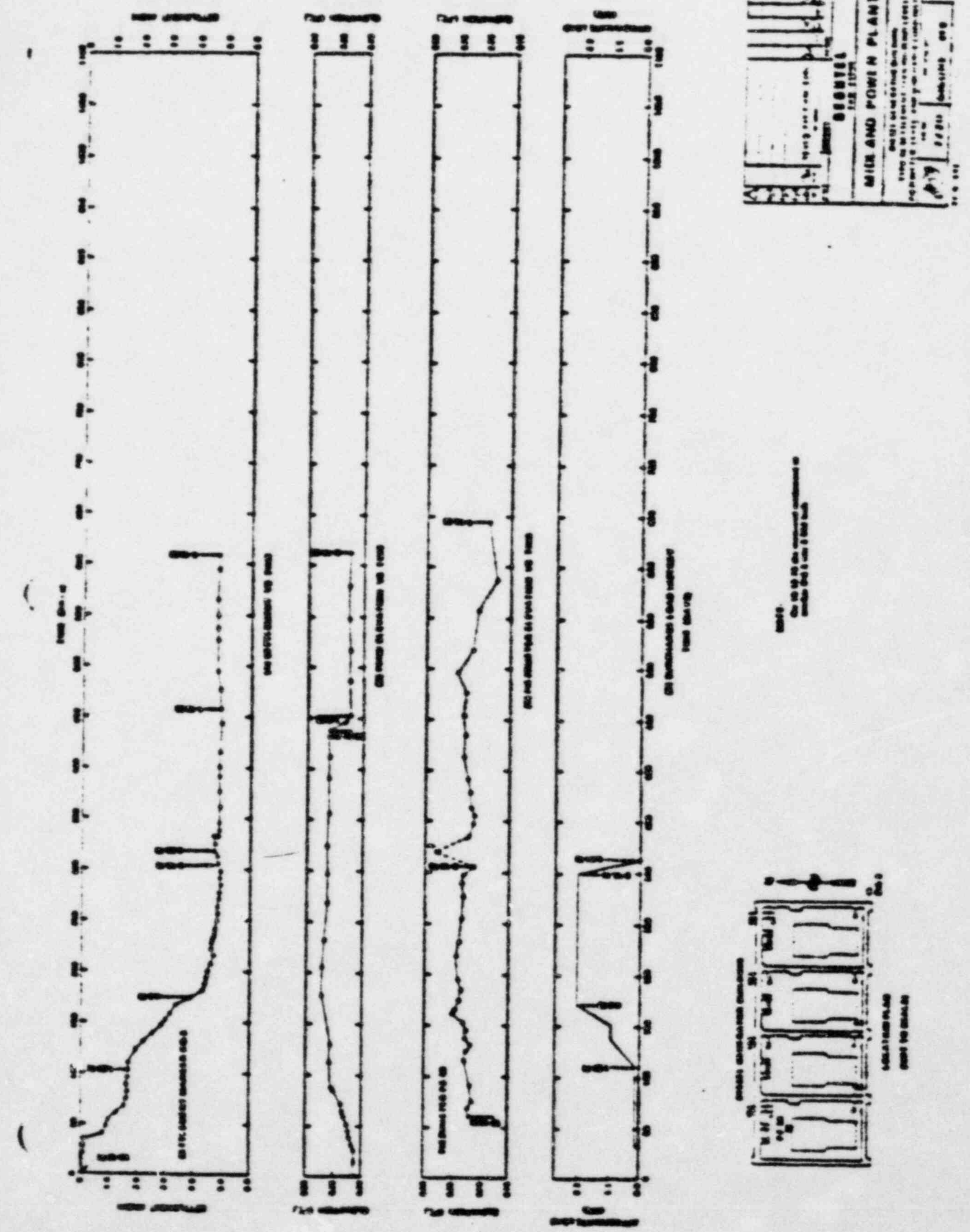
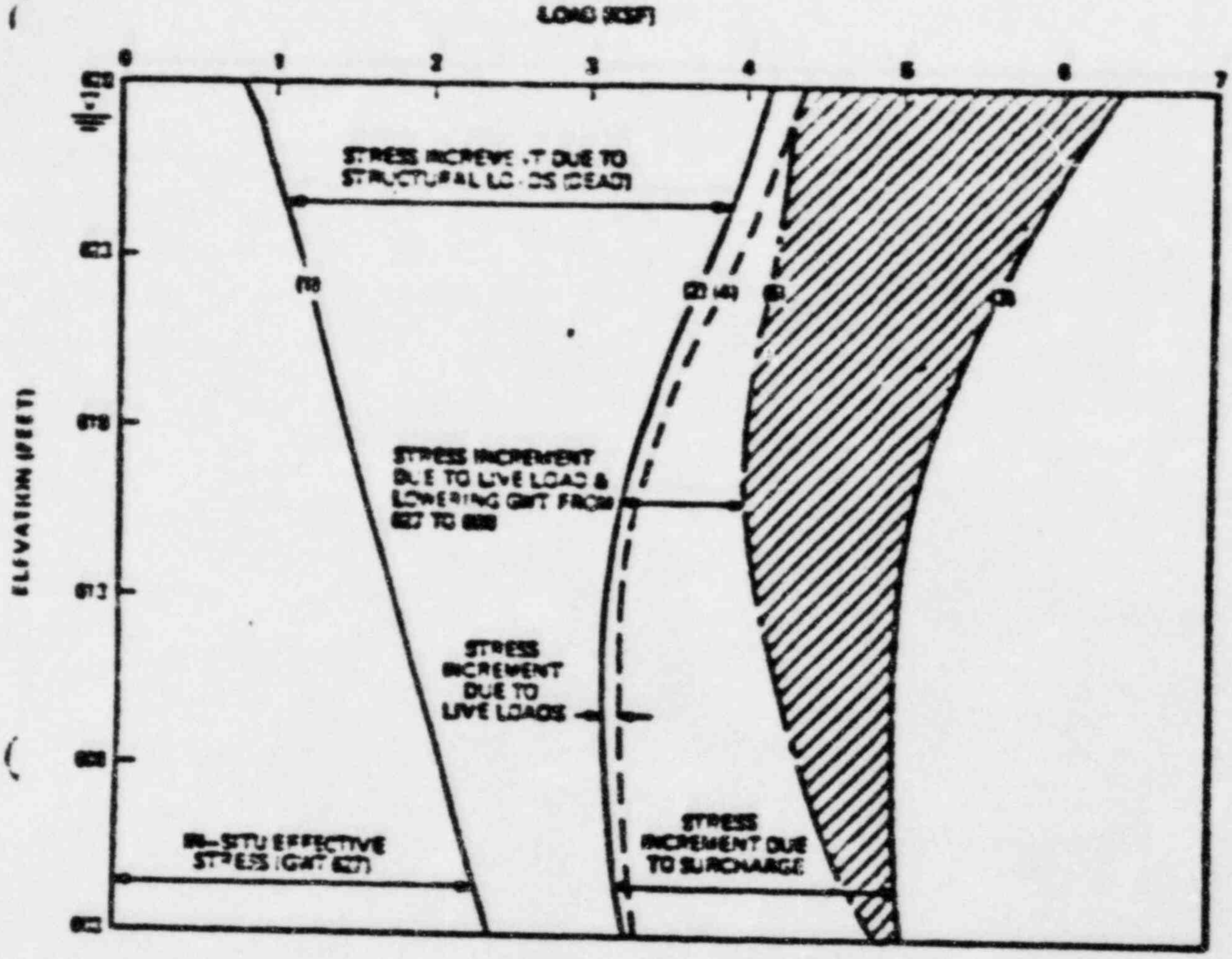


Figure 2
(See Reference 1)



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)

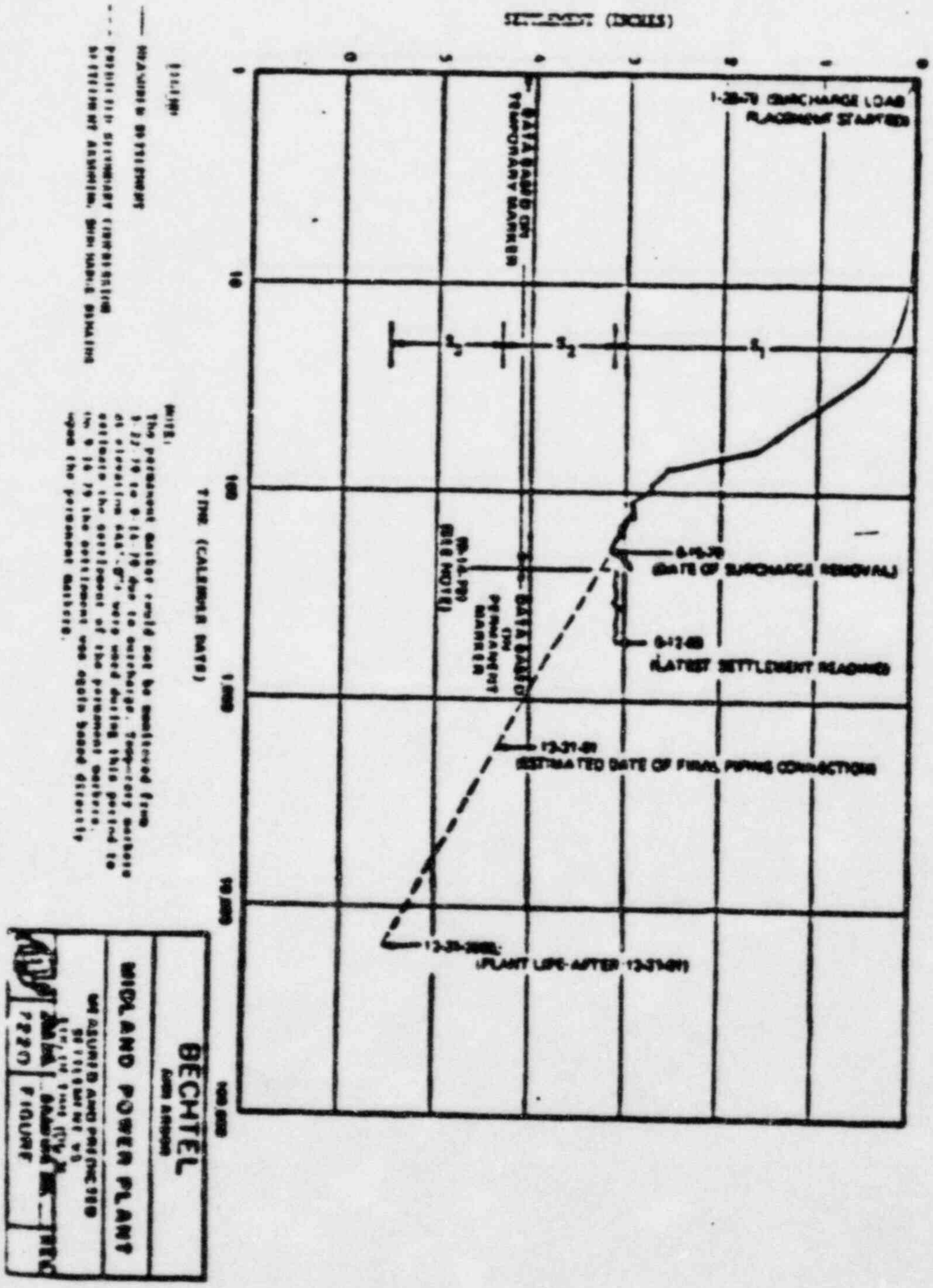


Figure 4
(See Reference 1)

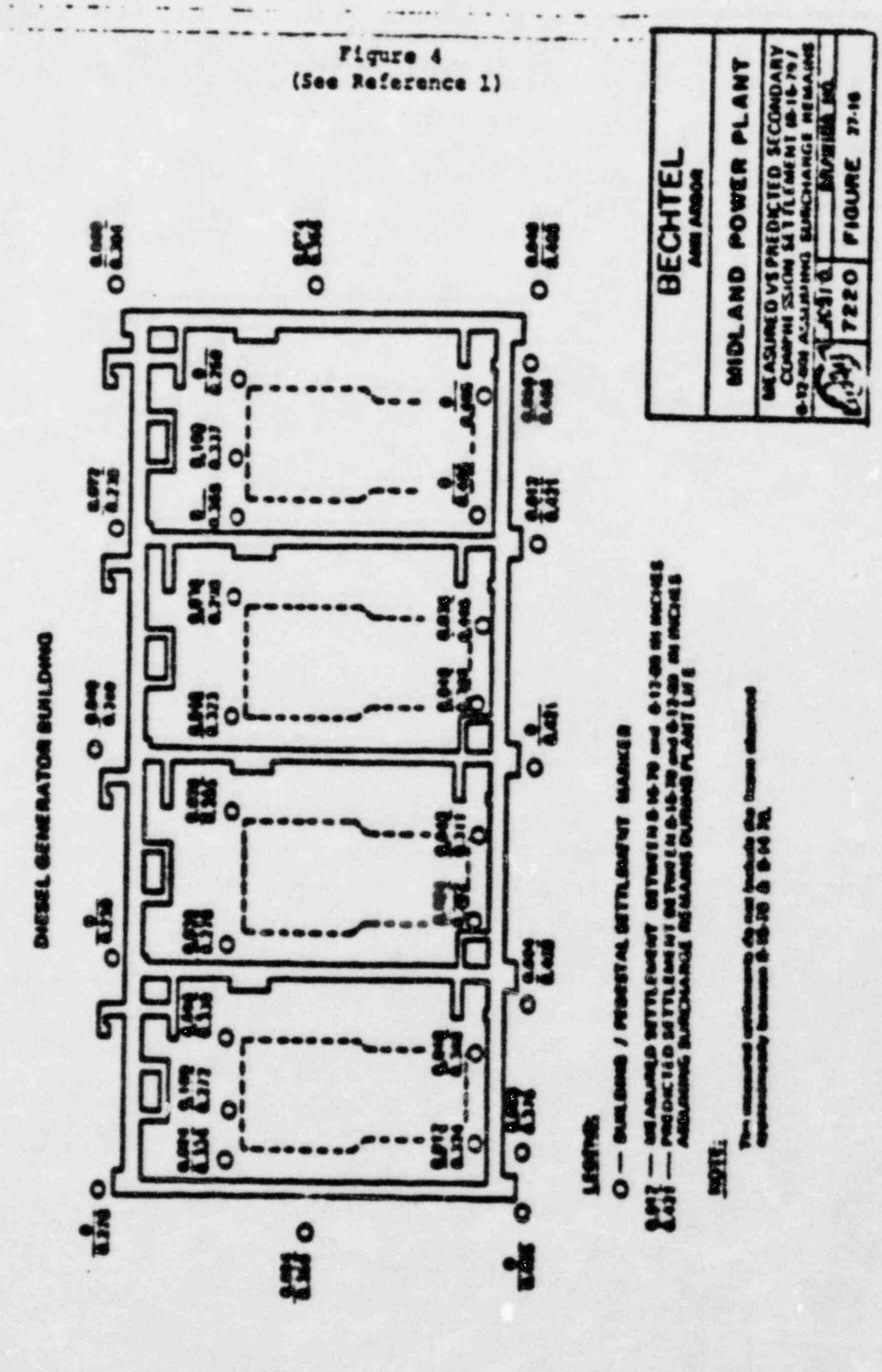
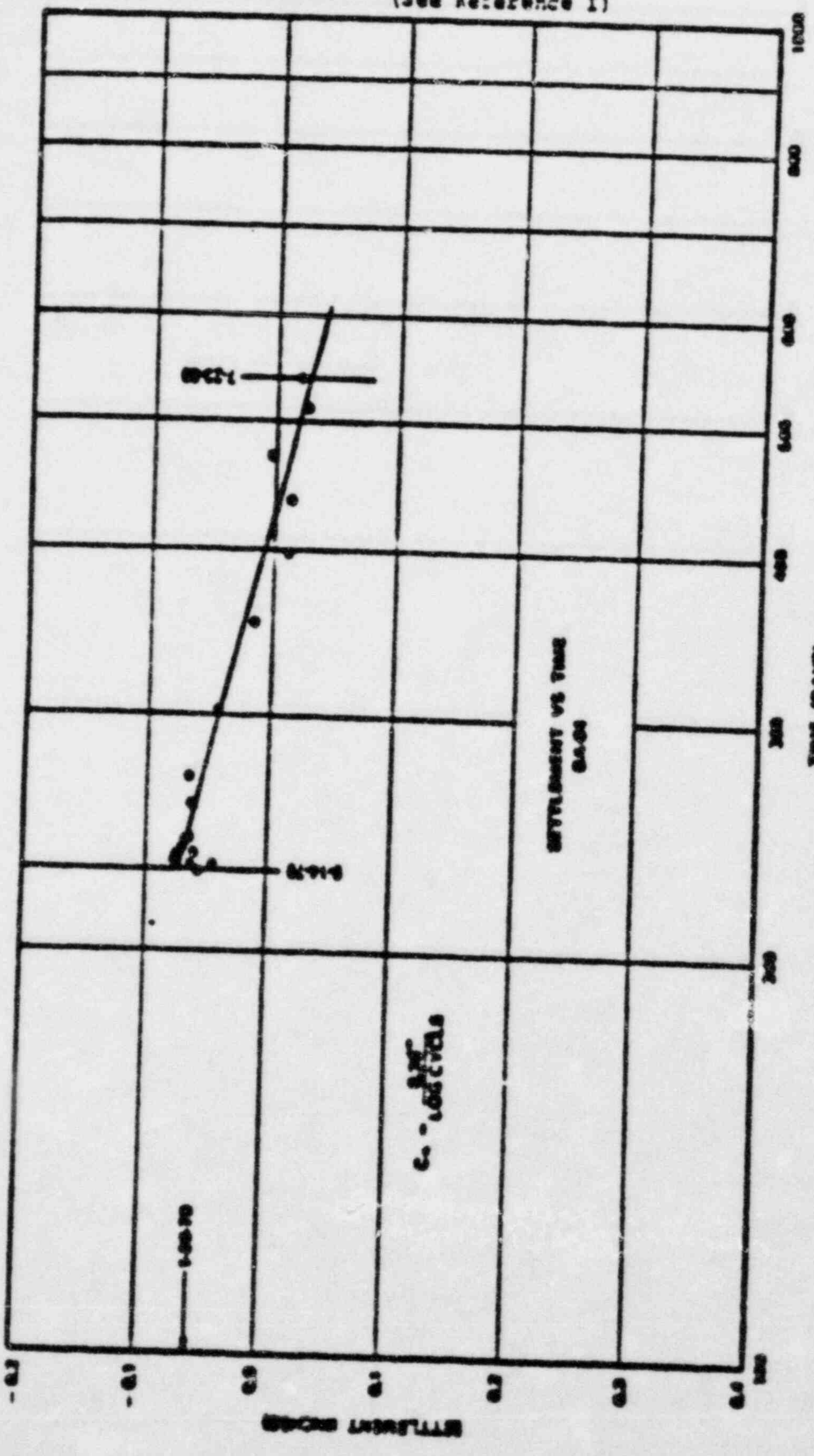
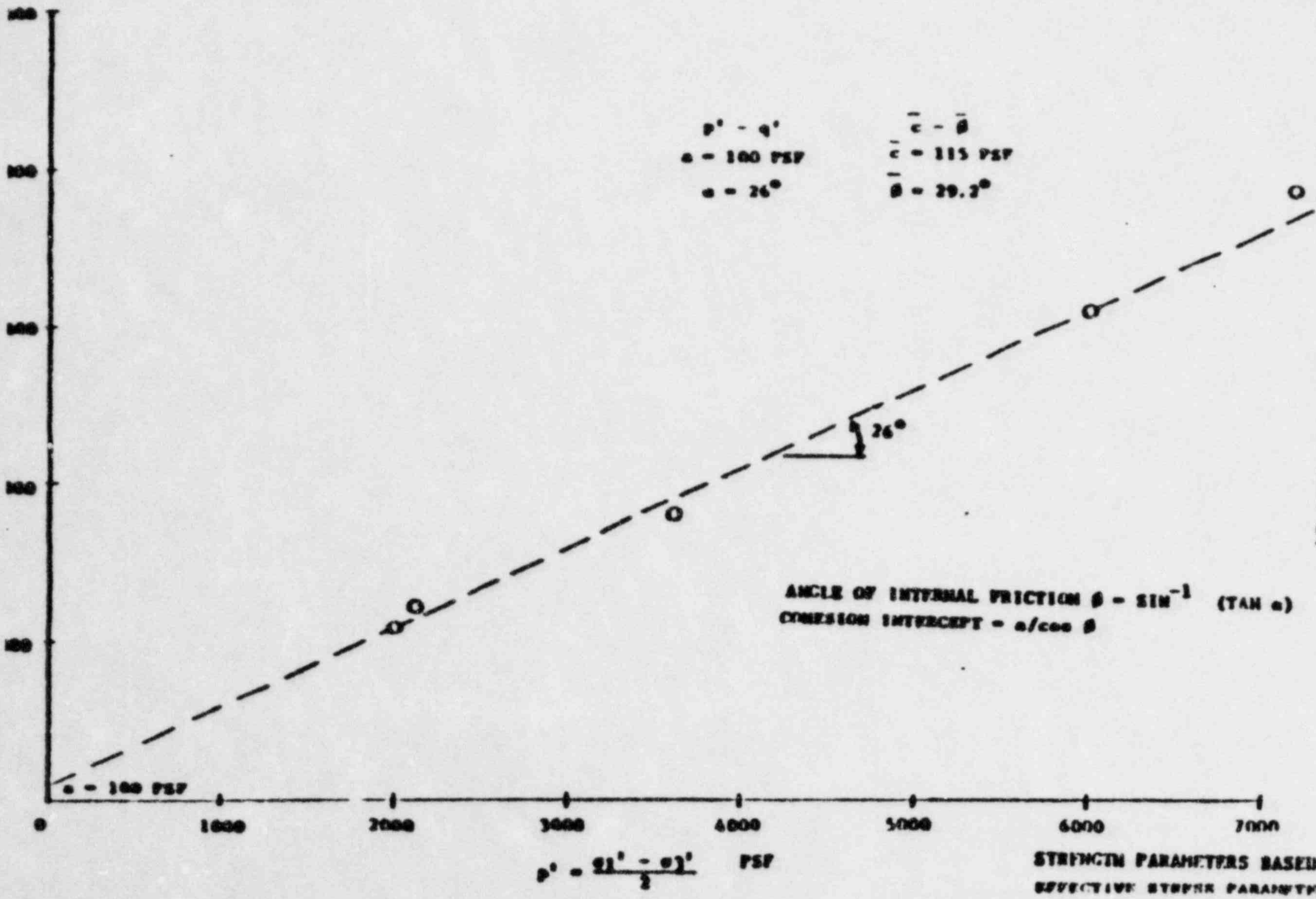


Figure 5
(See Reference 1)

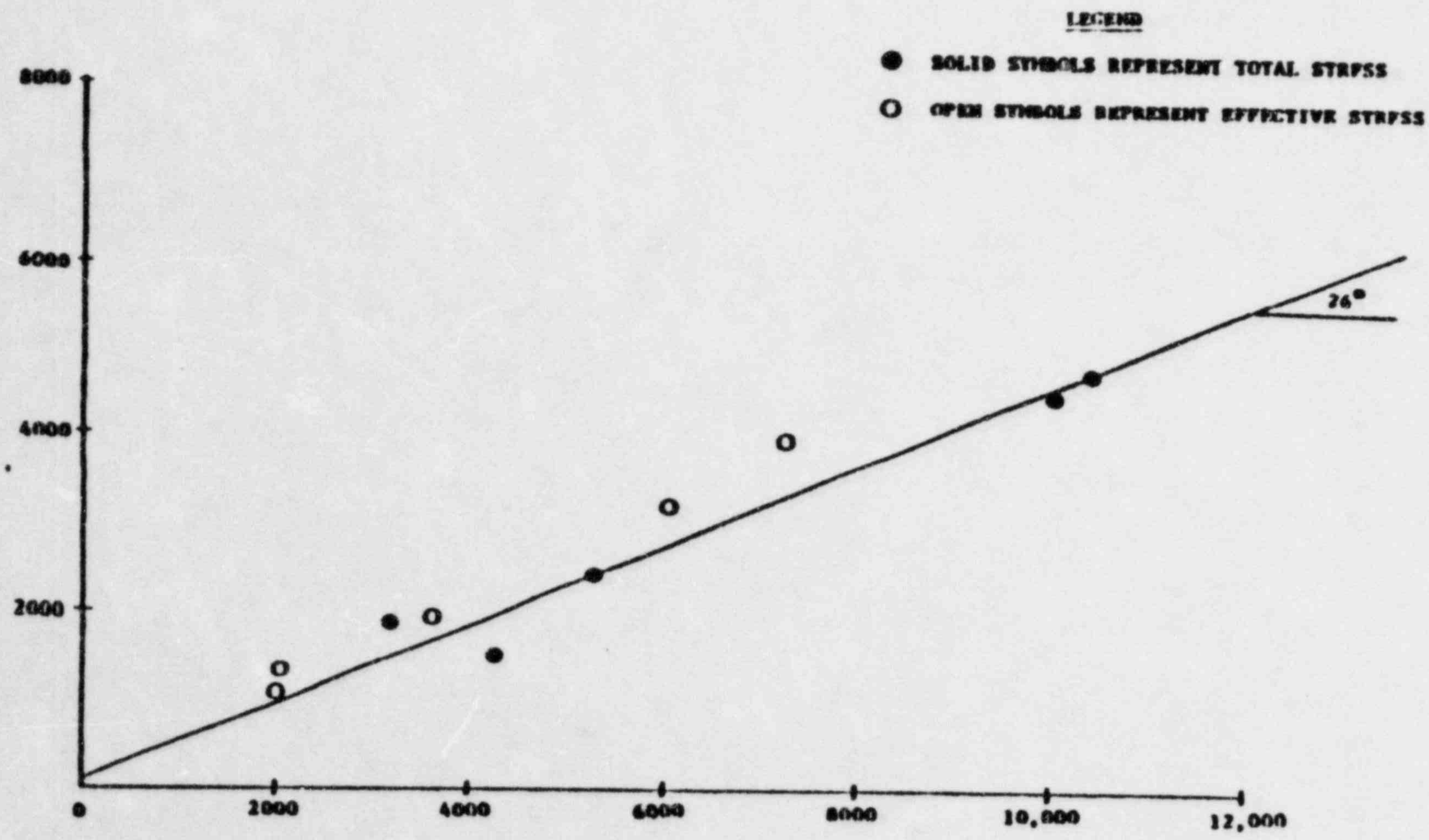




STRENGTH PARAMETERS BASED ON EFFECTIVE STRESS PARAMETERS

Figure 6
(See Reference 1)

Figure 7
(See Reference 1)



COMPARISON OF TOTAL AND EFFECTIVE
STRESS STRENGTH PARAMETERS

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

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

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

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

$= 28395$ psf

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

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

b). Use Vesic

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

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

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

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

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

Figure 8 (Sh 2 of 2)

B. BASED ON FIVE SAMPLES WITH LOWER DENSITIES

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

$\bar{c} = 114$ psf

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

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

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

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

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

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

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

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

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

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

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

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

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.

M. T. Davisson

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. Facht, 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.

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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.

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9. M. T. Davisson and S. Prakash, "A Review of Soil-Pole 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.

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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.

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

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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 to 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, MI.

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 M. 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, 125 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.

Consulting Experience Directly Applicable for the Design of Large Underground Chambers for Storage

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.

Alfred J. Hendron, Jr.

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 Power Co Exhibit #4
Hella Deposition 12/9/80

cc: SHH
JWC
GSK
TCC
DMB
BWM
WRB
MGK
Doc Control
Serial

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

JUN 3 0 1980

Docket Nos.: 50-329/330

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 PLANT FILL

We have reviewed your responses to our requests of November 19, 1979 regarding the quality of plant fill, effects and remedial actions resulting therefrom. Our review is being performed with the assistance of the U. S. Army Corps of Engineers. We and they find that the results of additional explorations and laboratory testing identified in Enclosure 1 (Request 37) are needed to support required geotechnical engineering studies. Details on the extent of these studies will be provided shortly by separate correspondence. Enclosure 1 is provided in order that you may initiate planning of the required explorations in a timely manner. However we suggest you await receipt of these further details prior to physically beginning the explorations. Enclosure 1 (Footnote 4 of Table 37-1) also includes requests for advanced notification of the availability of certain samples.

As noted in our Request 37 of Enclosure 1, your position in previous responses to Requests 5 and 35 not to complete additional explorations, sampling and laboratory testing after preloading continues to be unacceptable to us. So that you might better understand our position, we offer the following observations:

(1) The preload program as completed on the heterogeneous materials which were placed for the purpose of structural fill is not necessarily an improvement, nor does it necessarily produce foundation soils of more uniform engineering properties, compared to the soil performance which would have resulted if the material had been properly compacted to the original requirements established in the Midland PSAR.

(2) To develop reasonable assurance of plant safety, the required studies are needed to serve as an independent verification of the predictions of future settlements and the conclusions of the preload program.

~~8007180084~~

Sheriff

(3) The required studies will permit an estimate of total and differential settlement for involved structures and systems following drawdown with the proposed permanent dewatering system.

(4) Certain aspects of the preload program, such as the complication introduced by the simultaneous raising of the cooling pond reservoir, present difficulties in our full acceptance of your conclusion of the preload program.

Enclosure 1 also includes other requests for information which we and the U. S. Army Corps of Engineers need to continue our review.

We would appreciate your response to Enclosure 1 at your earliest opportunity. A partial reply based upon data already available should be submitted rather than to await the results of new borings and tests contained in parts of Enclosure 1. Should you require clarifications of these requests and positions, please contact us.

Sincerely,

A. Schwencer

A. Schwencer, Acting Chief
Licensing Branch No. 3
Division of Licensing

Enclosure:
As stated

cc: See next page

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

cc: Commander, Naval Surface Weapons Center
ATTN: P. C. Huang
G-402
White Oak
Silver Spring, Maryland 20910

Mr. L. J. Auge, Manager
Facility Design Engineering
Energy Technology Engineering Center
P. O. Box 1449
Canoga, Park, California 91304

Mr. William Lawhead
U. S. Corps of Engineers
NCEED - T
7th Floor
477 Michigan Avenue
Detroit, Michigan 48226

ADDITIONAL REQUESTS REGARDING PLANT FILL

36. We have reviewed your response to Request 24 and find that information from additional boring logs is needed.

Provide the boring logs for the following explorations:
a. Pull down holes PD-1 thru PD-27 (35 holes that include 8A, 20A, 20B, 20C, 15A, 15B, 15C and 27A)
b. LOW-1 thru LOW-14 (14 holes)
c. TW-1 thru TW-5 and PZ-1 thru PZ-48 (55 holes)
d. OW-1 thru OW-5 (5 holes)
e. TEW-1 thru TEW-8 (8 holes)

The logs should include date and method of drilling, the type and location of samples attempted. Also provide the locations, boring logs and available test data of any exploration completed in 1979 and 1980 which has not yet been submitted.

37. (RSP) Your position in previous responses to Requests 5 and 35 not to complete additional explorations, sampling and laboratory testing following the preload program continues to be unacceptable. We require that you complete as a minimum, the exploration and testing program indicated by Table 37-1.

38. Discuss the foundation design for any seismic safety-related piping and conduit connected to or located under the Radwaste Building and Turbine Building where piping and conduit have been placed on plant fill.

Table 37-1

Request for Additional Explorations, Sampling and Testing

Location 1/	Depth 2/	Sampling 3/	Lab Testing 4/	Anticipated Geotechnical 6/ Engineering Studies to be Required
Diesel Generator Building (6 holes along perimeter)	Thru fill and a minimum of 5' into natural glacial till soils	Classify samples according to Unified Soils Classification System	For cohesive soils C-D (Consolidated-Drained) C-U (Consolidated-Undrained) Consolidation 5/ For sands Drained Direct Shear on both loose & dense specimens Relative Density	Bearing Capacity Settlement Piping Distortion
Auxiliary Building (2 holes)	Same as above	Same as above	Same as above except add U-U (Unconsolidated-Undrained for cohesive soils	Caisson Foundation Design (Vertical and Lateral Load Support)
Service Water Pump (1 hole) Structure and Retaining Walls (? holes)	Same as above	Same as above	Same as above except consolidation testing would be limited to samples in retaining wall foundations.	Pile Foundation Design (Vertical and Lateral Load Support) Retaining Wall Stability Settlement.
Cooling Pond Embankments (7 holes along perimeter)	Extend thru fill and a minimum of 5' into natural residual soils except hole no. 5 which should extend to bottom elevation of cooling pond.	Same as above	For cohesive soils C-D (Consolidated-Drained) C-U (Consolidated-Undrained) U-U (Unconsolidated-Undrained)	Slope Stability Fill compaction adequacy

Have 2 cross holes that we have blow counts on.

Caisson goes to till

Consolidation test would only be applicable

Our position is to put no holes

No subsidence

NOTES: See page 2

Table 37-1 (continued)

NOTES:

1/ See attached Figs. 37-1 and 37-2 for approximate boring location. Holes to be accurately located in the field to avoid obstructions, underground piping and conduits and slurry trench area.

2/ No boring is to be terminated in loose or soft soils.

3/ Continuous split spoon sampling using SPT is required. Holes are to be held open using either casing or hollow stem auger. Additional borings to obtain representative undisturbed samples for detailed laboratory testing should be located at the completion and elevation of the split spoon sampling program. The groundwater level should be recorded at the completion of drilling in all borings once the level has stabilized.

4/ Normal classification (e.g., gradation, Atterberg Limits) unit weight and moisture content testing to be performed on representative samples from each significant foundation layer. This column pertains to lab testing in addition to the above mentioned tests. It is requested that at least one week notice be provided to the NRC before opening undisturbed samples to permit on site visual observation by Corps of Engineer representative.

5/ The maximum load should be great enough to establish the straight-line portion of the void ratio-pressure curve.

6/ Details on the extent of geotechnical engineering studies to be completed using the results of field and lab testing work will be provided in a separate letter.

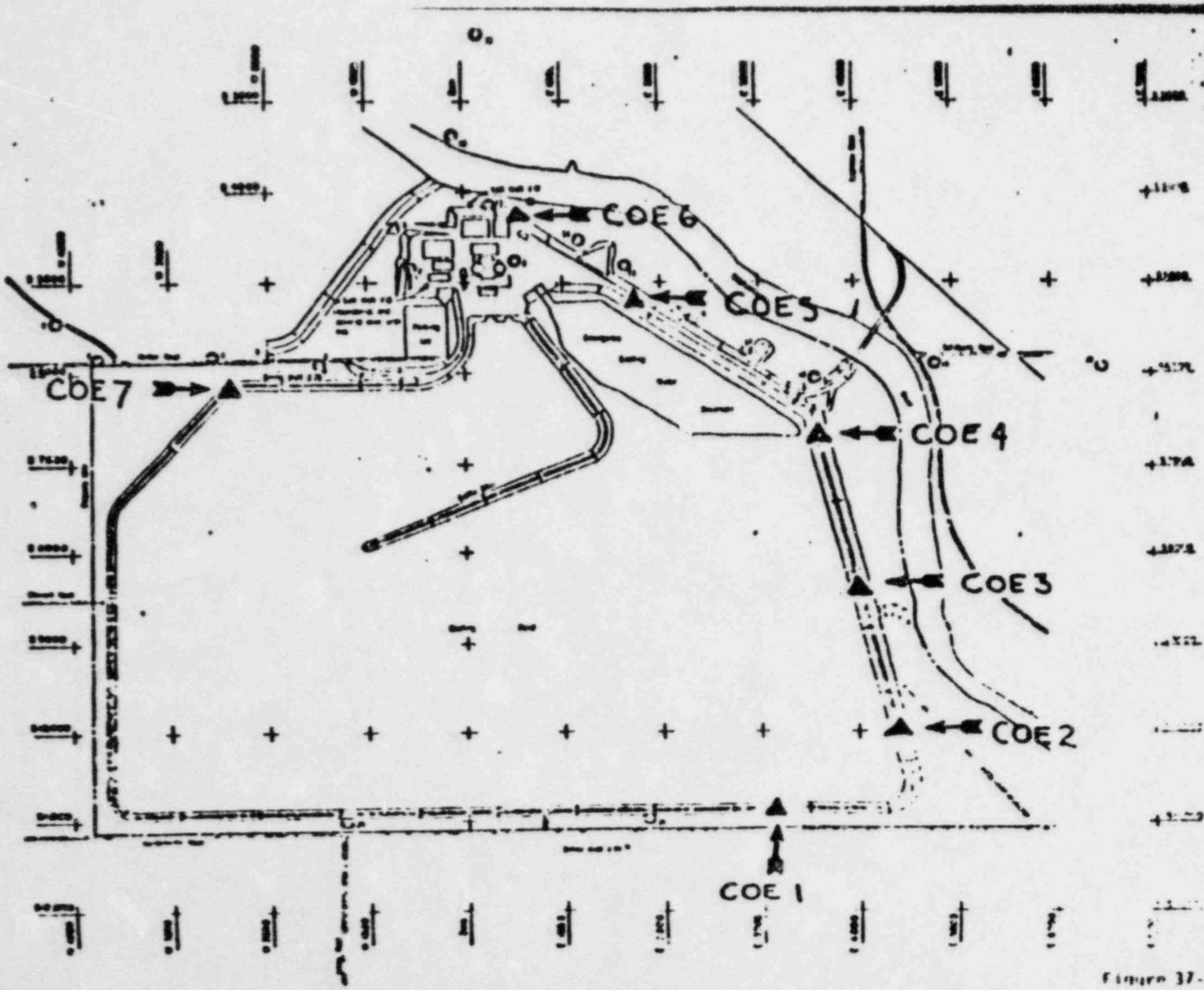
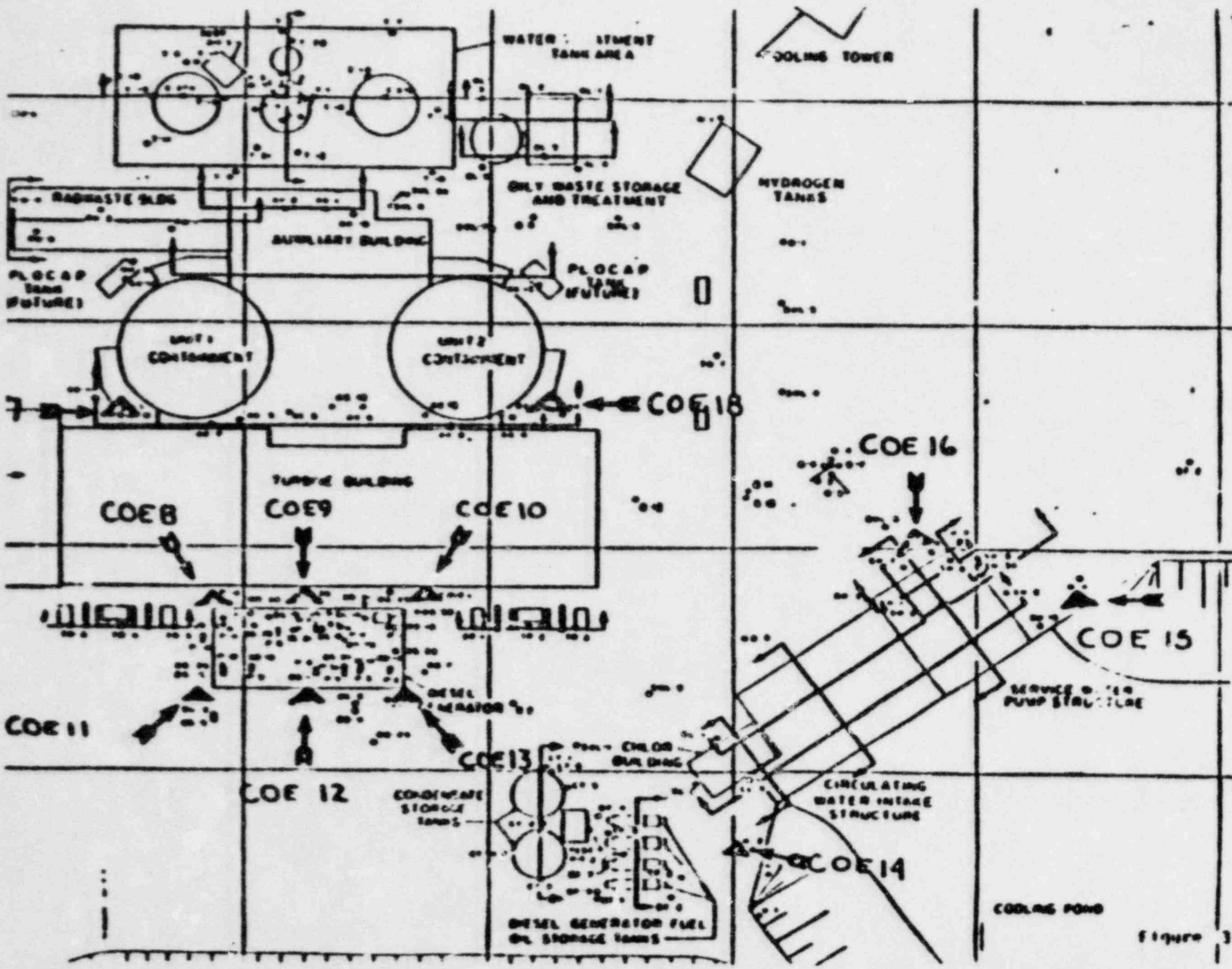


Figure 37-1


WATER TREATMENT TANK AREA
COOLING TOWER
RADWASTE BLDG
OILY WASTE STORAGE AND TREATMENT
HYDROGEN TANKS
AUXILIARY BUILDING
FLOCAP TANK (FUTURE)
FLOCAP TANK (FUTURE)
UNIT 1 CONTAINMENT
UNIT 2 CONTAINMENT
COE 18
COE 16
TURBINE BUILDING
COE8
COE9
COE10
COE 15
SERVICE WATER PUMP STRUCTURE
DIESEL GENERATOR
COE11
COE13
COE 12
CHLOR. BUILDING
CIRCULATING WATER INTAKE STRUCTURE
CONDENSATE STORAGE TANKS
COE14
COOLING POND
DIESEL GENERATOR FUEL OIL STORAGE TANKS
Figure 37-2

Power Co Exhibit #5 - Midland ... 10/...

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

AUG 4 1980

AUG 07 1980

Copies To: [illegible]

Docket Nos.: 50-329/330

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

Dear Mr. Cook:

SUBJECT: CORP OF ENGINEERS REPORT AND REQUEST FOR ADDITIONAL INFORMATION ON PLANT FILL

My letter of June 30, 1980 requested the results of additional explorations and laboratory testing needed to support certain geotechnical engineering studies on the Midland plant fill and associated remedial actions. That letter noted that details on the extent of these studies would be provided by separate correspondence. Enclosure 1 is a letter report of July 7, 1980 by our consultant, the U.S. Army Corps of Engineers, and is forwarded to this end.

Paragraph 4 of the Corps report identifies additional information needed to resolve specific problems identified in paragraph 3. For purposes of control, we have re-numbered the subparagraphs of paragraph 4 to be sequential with our prior requests on this matter. They have also been marked to reflect the results of NRR review. Your reply should reference the revised numbering system and should address the requests as marked to reflect our changes.

Subparagraph 4j of the Corps report entitled Liquefaction Potential, is not included in our re-numbering since it represents an evaluation rather than a request. We consider this evaluation to be tentative at this time since it is subject to the determination of suitable seismic design input for the site. We will address this matter shortly by separate correspondence.

8008270158

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

Sincerely,

A. Schwencer

A. Schwencer, Acting Chief
Licensing Branch No. 3
Division of Licensing

Enclosure:
COE Letter Report
dated 7/7/80

cc: See next page

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Bay City, Michigan 48706

DEPARTMENT OF THE ARMY

DETROIT DISTRICT, CORPS OF ENGINEERS
BOX 1027
DETROIT, MICHIGAN 48231

ENCLOSURE 1

REPLY TO
ATTENTION OF

7 JUL 1980

NCEED-T

SUBJECT: Interagency Agreement No. NRC-03-79-167, Task No. 1 - Midland Plant Units 1 and 2, Subtask No. 1 - Letter Report

THRU: Division Engineer, North Central
ATTN: NCDED-G (James Simpson)

TO: U.S. Nuclear Regulatory Commission
ATTN: Dr. Robert E. Jackson
Division of Systems Safety
Mail Stop P-314
Washington, D. C. 20555

1. The Detroit District hereby submits this letter report with regard to completion of subtask No. 1 of the subject Interagency Agreement concerning the Midland Nuclear Plant, Units 1 and 2. The purpose of this report is to identify unresolved issues and make recommendations on a course of action and/or cite additional information necessary to settle these matters prior to preparation of the Safety Evaluation Report.

2. The Detroit District's team providing geotechnical engineering support to the NRC to date has made a review of furnished documents concerning foundations for structures, has jointly participated in briefing meetings with the NRC staff, Consumers Power Company (the applicant) and personnel from North Central Division of the Corps of Engineers and has made detailed site inspections. The data reviewed includes all documents received through Amendment 78 to the operating license request, Revision 28 of the FSAR, Revision 7 to the 10 CFR 50.54(f) requests and MCAR No. 24 through Interim Report No. 8. Generally, each structure within the complex was studied as a separate entity.

3. A listing of specific problems in review of Midland Units 1 and 2 follows for Category I structures. The issues are unresolved in many instances, because of inadequate or missing information. The structures to be addressed follow the description of the problem.

a. Inadequate presentation of subsurface information from completed borings on meaningful profiles and sectional views. All structures.

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b. Discrepancies between soil descriptions and classifications on boring logs with submitted laboratory test results summaries. Examples of such discrepancies are found in boring T-14 (Borated water tank) which shows stiff to very stiff clay where laboratory tests indicate soft clay with shear strength of only 500 p.s.f. The log of boring T-15 shows stiff, silty clay, while the lab tests show soft, clayey sand with shear strength of 120 p.s.f. All structures.

c. Lack of discussion about the criteria used to select soil samples for lab testing. Also, identification of the basis for selecting specific values for the various parameters used in foundation design from the lab test results. All structures.

d. The inability to completely identify the soil behavior from lab testing (prior to design and construction) of individual samples, because in general, only final test values in summary form have been provided. All structures.

(1) Lack of site specific information in estimating allowable bearing pressures. Only textbook type information has been provided. If necessary, bearing capacity should be revised based on latest soils data. All structures on, or partially on, fill.

(2) Additional information is needed to indicate the design methods used, design assumptions and computations in estimating settlement for safety related structures and systems. All structures except Diesel Generator Building where surcharging was performed.

e. A complete detailed presentation of foundation design regarding remedial measures for structures undergoing distress is required. Areas of remedial measures except Diesel Generator Building.

f. There are inconsistencies in presentation of seismic design information as affected by changes due to poor compaction of plant fill. Response to NRC question 35 (10 CFR 50.54f) indicates that the lower bound of shear wave velocity is 500 feet per second. We understand that the same velocity will be used to analyze the dynamic response of structures built on fill. However, from information provided by the applicant at the site meeting on 27 and 28 February 1980, it was stated that, except for the Diesel Generator Building, higher shear wave velocities are being used to re-evaluate the dynamic response of the structures on fill material. Structures on fill or partially on fill except Diesel Generator Building.

4. A listing of specific issues and information necessary to resolve them.

39. a. Reactor Building Foundation

(1) Settlement/Consolidation. Basis for settlement/consolidation of the reactor foundation as discussed in the FSAR assumes the plant site would

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7 JUL 1980

NCEED-T
SUBJECT: Interagency Agreement No. NRC-03-79-167, Task No. 1 - Midland Plant Units 1 and 2, Subtask No. 1 - Letter Report

not be dewatered. Discuss and furnish computation for settlement of the Reactor Buildings in respect to the changed water table level as the result of site dewatering. Include the effects of bouyancy, which were used in previous calculations, and fluctuations in water table which could happen if the dewatering system became inoperable.

(2) Bearing Capacity. Bearing capacity computations should be provided and should include method used, foundation design, design assumptions, adopted soil properties, and basis for selecting ultimate bearing capacity and resulting factor of safety.

40. ~~f~~. Diesel Generator Building.

(1) Settlement/Consolidation. In the response to NRC Question 4 and 27, (10 CFR 50.54f), the applicant has furnished the results of his computed settlements due to various kinds of loading conditions. From his explanation of the results, it appears that compressibility parameters obtained by the preload tests have been used to compute the static settlements. Information pertaining to dynamic response including the amplitude of vibration of generator pedestals have also been furnished. The observed settlement pattern of the Diesel Generator Building indicates a direct correlation with soil types and properties within the backfill material. To verify the preload test settlement predictions, compute settlements based on test results on samples from new borings which we have requested in a separate memo and present the results. Reduced ground water levels resulting from dewatering and diesel plus seismic vibration should be considered in settlement and seismic analysis. Furnish the computation details for evaluating amplitude of vibration for diesel generator pedestals including magnitude of exciting forces, whether they are constant or frequency dependent.

(2) Bearing Capacity. Applicant's response to NRC Question 35 (10 CFR 50.54f) relative to bearing capacity of soil is not satisfactory. Figure 35-3, which has been the basis of selection of shear strength for computing bearing capacity does not reflect the characteristics of the soils under the Diesel Generator Building. A bearing capacity computation should be submitted based on the test results of samples from new borings which we have requested in a separate memo. This information should include method used, foundation design assumptions, adopted soil properties and basis for selection, ultimate bearing capacity and resulting factor of safety.

(3) Preload Effectiveness. The effectiveness of the preload should be studied with regard to the moisture content of the fill at the time of preloading. The height of the water table, its time duration at this level, and whether the plant fill was placed wet or dry of optimum would be all important considerations.

7 JUL 1980

NCEED-T
SUBJECT: Interagency Agreement No. NRC-03-79-167, Task No. 1 - Midland Plant Units 1 and 2, Subtask No. 1 - Letter Report

(a) Granular Soils.

When sufficient load is applied to granular soils it usually causes a reorientation of grains and movement of particles into more stable positions plus (at high stresses) fracturing of particles at their points of contact. Reorientation and breakage creates a chain reaction among these and adjacent particles resulting in settlement. Reorientation is resisted by friction between particles. Capillary tension would tend to increase this friction. A moisture increase causing saturation, such as a rise in the water table as occurred here, would decrease capillary tension resulting in more compaction. Present a discussion on the water table and capillary water effect on the granular portion of the plant fill both above and below the water table during and after the preload.

(b) Impervious and/or Clay Soils.

Clay fill placed dry of optimum would not compact and voids could exist between particles and/or chunks. In this situation SPT blow counts would give misleading information as to strength. Discuss the raising of the water table and determine if the time of saturation was long enough to saturate possible clay lumps so that the consolidation could take place that would preclude further settlement.

Discuss the preload effect on clay soils lying above the water table (7 feet ±) that were possibly compacted dry of optimum. It would appear only limited consolidation from the preload could take place in this situation and the potential for further settlement would exist.

Discuss the effect of the preload on clays placed wet of optimum. It would appear consolidation along with a gain in strength would take place. Determine if the new soil strength is adequate for bearing capacity.

~~Conclusion: Since the reliability of existing fill and compaction information is uncertain, additional borings and tests to determine void ratio (granular soils) relative density, moisture content, density, consolidation properties and strength (triaxial tests) would appear to be desirable in order to satisfactorily answer the above questions. Borings should be continuous push with undisturbed cohesive soil samples taken.~~ Deleted: Covered by 6/30/80 Letter

(4) Miscellaneous. A contour map, showing the settlement configuration of the Diesel Generator Building, furnished by the applicant at the meeting of 27 and 28 February 1980 indicates that the base of the building has warped due to differential settlements. Additional stresses will be induced in the various components of the structure. The applicant should evaluate these stresses due to the differential settlement and furnish the computations and results for review.

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41. f. Service Water Building Foundation.

(1) Bearing Capacity. A detailed pile design based upon pertinent soil data should be developed in order to more effectively evaluate the proposed pile support system prior to load testing of test piles. Provide adopted soil properties, reference to test data on which they are based, and method and assumptions used to estimate pile design capacity including computations. Provide estimated maximum static and dynamic loads to be imposed and individual contribution (DL, LL, OBE, SSE) on the maximum loaded pile. Provide factor of safety against soil failure due to maximum pile load.

(2) Settlements.

(a) Discuss and provide analysis evaluating possible differential settlement that could occur between the pile supported end and the portion placed on fill and glacial till. Describe the impact of failure on safety related features (e.g., diesel fuel oil storage tanks) behind or near the wall.

(b) ~~Present~~ Discuss~~ion~~ why the retaining wall adjacent to the intake structure is not required to be a Seismic Category I structure. Evaluate the observed settlement of both the service water pumphouse retaining walls and the intake structure retaining wall and the significance of the settlement including future settlement prediction on the safe operation of the Midland Nuclear Plant. This evaluation should address actual stresses induced by the settlement against allowable stresses permitted by approved codes.

(3) Seismic Analysis. Provided the proposed 100 ton ultimate pile load capacities are achieved and reasonable margin of safety is available, the vertical pile support proposed for the overhang section of the Service Water Pump Structure will provide the support necessary for the structure under combined static and seismic inertial loadings even if the soil under the overhang portion of the structure should liquefy. There is no reason to think this won't be achieved at this time, and the applicant has committed to a load test to demonstrate the pile capacity. The dynamic response of the structure, including the inertial loads for which the structure itself is designed and the mechanical equipment contained therein, would change as a result of the introduction of the piles. Therefore:

(a) Please summarize or provide copies of reports on the dynamic analysis of the structure in its old and proposed configuration. For the latter, provide detailed information on the stiffness assigned to the piles and the way in which the stiffnesses were obtained and show the largest change in interior floor vertical response spectra resulting from the proposed modification. If the proposed configuration has not yet been analyzed, describe the analyses that are to be performed giving particular attention to the basis for calculation or selection, of and the range of numerical stiffness values assigned to the vertical piles.

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

42. ~~f.~~ Auxiliary Building Electrical Penetration Areas and Feedwater Isolation Valve Pits.

(1) Settlement. Provide the assumptions, method, computation and estimate of expected allowable lateral and vertical deflections under static and seismic loadings.

(2) Provide the construction plans, and specifications for underpinning operations beneath the Electrical Penetration Area and Feedwater Valve Pit. The requested information to be submitted should cover the following in sufficient details for evaluation:

(a) Details of the temporary dewatering system (locations, depth, size and capacity of wells) including the monitoring program to be required, (for example, measuring drawdown, flow, frequency of observations, etc.) to evaluate the performance and adequacy of the installed system.

(b) Location, sectional views and dimensions of access shaft and drift to and below auxiliary building wings.

(c) Details of temporary surface support system for the valve pits.

~~(d)~~ Dewatering before underpinning is recommended in order to preclude differential settlement between pile and soil supported elements and negative drag forces.

(d) Provide adopted soil properties, method and assumptions used to estimate caisson and/or pile design capacities, and computational results. Provide estimated maximum static and dynamic load (compression, uplift and lateral) to be imposed and the individual contribution (DL, LL, OBE, SSE) on maximum loaded caisson and/or pile. Provide factor of safety against soil failure due to maximum pile load.

(e) Discuss and furnish computations for settlement of the portion of the Auxiliary Building (valve pits, and electrical penetration area) in respect to changed water level as a result of the site dewatering. Include the effect of bouyancy, which was used in previous calculations, and fluctuations in water table which could happen, if dewatering system becomes inoperable.

(f) Discuss protection measures to be required against corrosion, if piling is selected.

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(8) Identify specific information, data and method of presentation to be submitted for regulatory review at completion of underpinning operation. This report should summarize construction activities, field inspection records, results of field load tests on caissons and piles and an evaluation of the completed fix for assuring the stable foundation.

43. Borated Water Tanks.

(1) Settlement. The settlement estimate for the Borated Water Storage Tanks furnished by the applicant in response to NRC Question 31 (10 CFR 50.54f) is based upon the results of two plate load tests conducted at the foundation elevation (EL 627.00±) of the tanks. Since a plate load test is not effective in providing information regarding the soil beyond a depth more than twice the diameter of the bearing plate used in the test, the estimate of the settlement furnished by the applicant does not include the contribution of the soft clay layers located at depth more than 5' below the bottom of the tanks (see Boring No. T-14 and T-15, and T-22 thru T-26).

(a) Compute settlements which include contribution of all the soil layers influenced by the total load on the tanks. Discuss and provide for review the analysis evaluating differential settlement that could occur between the ring (foundations) and the center of the tanks.

(b) The bottom of the borated tanks being flexible could warp under differential settlement. Evaluate what additional stresses could be induced in the ring beams, tank walls, and tank bottoms, because of the settlement, and compare with allowable stresses. Furnish the computations on stresses including method, assumptions and adopted soil properties in the analysis.

(2) Bearing Capacity. Laboratory test results on samples from boring T-15 show a soft stratum of soil below the tank bottom. Consideration has not been given to using these test results to evaluate bearing capacity information furnished by the applicant in response to NRC Question 35 (10 CFR 50.54f). Provide bearing capacity computations based on the test results of the samples from relevant borings. This information should include method used, foundation design assumptions, adopted soil properties, ultimate bearing capacity and resulting factor of safety for the static and the seismic loads.

44. Underground Diesel Fuel Tank Foundation Design

(1) Bearing capacity. Provide bearing capacity computation based on the test results of samples from relevent borings, including method used, foundation design assumptions, adopted soil properties, ultimate bearing capacity and the resulting factor of safety.

(2) Provide tank settlement analysis due to static and dynamic loads including methods, assumptions made, etc.

(3) What will be effects of uplift pressure on the stability of the tanks and the associated piping system if the dewatering system becomes inoperable?

45. ~~6~~. Underground Utilities:

(1) Settlement

(a) Inspect the interior of water circulation piping with video cameras and sensing devices to show pipe cross section, possible areas of crackings and openings, and slopes of piping following consolidation of the plant fill beneath the imposed surcharge loading.

(b) The applicant has stated in his response to NRC Question 7 (10 CFR 50.54f) that if the duct banks remain intact after the preload program has been completed, they will be able to withstand all future operating loads. Provide the results of the observations made, during the preload test, to determine the stability of the duct banks, with your discussion regarding their reliability to perform their design functions.

(c) The response to Question 17 of "Responses to NRC Requests Regarding Plant Fill" states that "there is no reason to believe that the stresses in Seismic Category I piping systems will ever approach the Code allowable." We question the above statement based on the following:

Profile 26" - OHBC-54 on Fig. 19-1 shows a sudden drop of approx. 0.2 feet within a distance of only 20 feet. Using the procedure on p. 17-2,

$$\sigma_b = E(e) = E\left(\frac{D}{2R}\right) = E\,\frac{(D)}{2}\,\frac{(8\delta)}{L^2}$$

$$\sigma_b = 30000\ \frac{(26)}{2}\left[\frac{8(0.2)(12)}{(20\times12)^2}\right] = 130.0\ \text{KSI}$$

~~Furthermore, the Eq. 10(a) of Article NC-3652.3, Sec. III, Division 1, of the ASME code requires that some Stress Intensification Factor "i" be assigned to all computed settlement stresses.~~ Yet, Table 17-2 lists only 52.5 KSI as allowable stress for this pipe. This matter requires further review. Please respond to this apparent discrepancy and also specify the location of each computed settlement stress at the pipeline stationing shown on the profiles. More than one critical stress location is possible along the same pipeline.

(d) During the site visit on 19 February 1980, we observed three instances of what appeared to be degradation of rattlespace at penetrations of Category I piping through concrete walls as follows:

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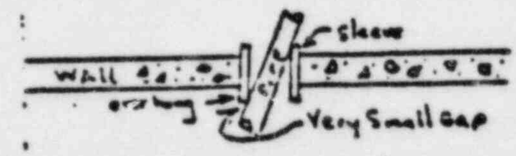
<u>West Borated Water Tank</u> - in the valve pit attached to the base of the structure, a large diameter steel pipe extended through a steel sleeve placed in the wall. Because the sleeve was not cut flush with the wall, clearance between the sleeve and the pipe was very small.



<u>Service Water Structure</u> - Two of the service water pipes penetrating the northwest wall of the service water structure had settled differentially with respect to the structure and were resting on slightly squashed short pieces of 2 x 4 placed in the bottom of the penetration. From the inclination of the pipe, there is a suggestion that the portions of the pipe further back in the wall opening (which was not visible) were actually bearing on the invert of the opening. The bottom surface of one of the steel pipes had small surface irregularities around the edges of the area in contact with the 2 x 4. Whether these irregularities are normal manufacturing irregularities or the result of concentration of load on this temporary support caused by the settlement of the fill, was not known.

These instances are sufficient to warrant an examination of those penetrations where Category I pipe derives support from plant fill on one or both sides of a penetration. In view of the above facts, the following information is required.

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

(2) Identify all those locations where a Category I pipe deriving support from plant fill penetrates an exterior concrete wall. Determine and report the vertical and horizontal rattlespace presently available and the minimum required at each location and describe remedial actions planned as a result of conditions uncovered in the inspection. It is anticipated that the answer to Question (1) can be obtained without any significant additional excavation. If this is not the case, the decision regarding the necessity to obtain information at those locations requiring major excavation should be deferred until the data from the other locations have been examined.

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(e) Provide details (thickness, type of material etc.) of bedding or cradle placed beneath safety related piping, conduits, and supporting structures. Provide profiles along piping, and conduits alignments showing the properties of all supporting materials to be adopted in the analysis of pipe stresses caused by settlement.

(f) The two reinforced concrete return pipes which exit the Service Water Pump Structure, run along either side of the emergency cooling water reservoir, and ultimately enter into the reservoir, are necessary for safe shutdown. These pipes are buried within or near the crest of Category I slopes that form the sides of the emergency cooling water reservoir. There is no report on, or analysis of, the seismic stability of post earthquake residual displacement for these slopes. While the limited data from this area do not raise the specter of any problem, for an important element of the plant such as this, the earthquake stability should be examined by state-of-the-art methods. Therefore, provide results of the seismic analysis of the slopes leading to an estimate of the permanent deformation of the pipes. Please provide the following: (1) a plan showing the pipe location with respect to other nearby structures, slopes of the reservoir and the coordinate system; (2) cross-sections showing the pipes, normal pool levels, slopes, subsurface conditions as interpreted from borings and/or logs of excavations at (a) a location parallel to and about 50 ft from the southeast outside wall of the service water pipe structure and (b) a location where the cross section will include both discharge structures. Actual boring logs should be shown on the profiles; their offset from the profile noted, and soils should be described using the Unified Soil Classification System; (3) discussion of available shear strength data and choice of strengths used in stability analysis; (4) determination of static factor or safety, critical earthquake acceleration, and location of critical circle; (5) calculation of residual movement by the method presented by Newmark (1965) or Makdisi and Seed (1978); and (6) a determination of whether or not the pipes can function properly after such movements.

46 ~~7~~. Cooling Pond.

(1) Emergency Cooling Pond. In recognition that the type of embankment fill and the compaction control used to construct the retention dikes for the cooling pond were the same as for the problem plant fill, we request reasonable assurance that the slopes of the Category I Emergency Cooling Pond (baffle dike and main dike) are stable under both static and dynamic loadings. We request a revised stability analysis for review, which will include identification of locations analyzed, adopted foundation and embankment conditions (stratification, seepage, etc.) and basis for selection, adopted soil properties, method of stability analysis used and resulting factor of safety with identification of sliding surfaces analyzed. Please address any potential impact on Category I pipes near the slopes, based on the results of this stability study. Recommendations for location of new exploration and testing have been provided in a separate letter.

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(2) Operating Cooling Pond. A high level of safety should be required for the remaining slopes of the Operating Cooling Pond unless it can be assured that a failure will not: (a) endanger public health and properties, (b) result in an assault on environment, (c) impair needed emergency access. Recommendations for locations of new borings and laboratory tests have been submitted in a separate letter. These recommendations were made on the assumptions that the stability of the operating cooling pond dikes should be demonstrated.

47 f. Site Dewatering Adequacy.

(1) In order to provide the necessary assurance of safety against liquefaction, it is necessary to demonstrate that the water will not rise above elevation 610 during normal operations or during a shutdown process. The applicant has decided to accomplish this by pumping from wells at the site. In the event of a failure, partial failure, or degradation of the dewatering system (and its backup system) caused by the earthquake or any other event such as equipment breakdown, the water levels will begin to rise. Depending on the answer to Question (a) below concerning the normal operating water levels in the immediate vicinity of Category I structures and pipelines founded on plant fill, different amounts of time are available to accomplish repair or shutdown. In response to Question 24 (10 CFR 50.54) the applicant states "the operating groundwater level will be approximately el 595 ft" (page 24-1). On page 24-1 the applicant also states "Therefore el 610' is to be used in the designs of the dewatering system as the maximum permissible groundwater level elevation under SSE conditions." On page 24-15 it is stated that "The wells will fully penetrate the backfill sands and underlying natural sands in this area." The bottom of the natural sands is indicated to vary from elevation 605 to 580 within the plant fill area according to Figure 24-12. The applicant should discuss and furnish response to the following questions:

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

(b) In the event the water levels in observation wells near Category I Structures or Pipelines supported on plant fill exceed those for normal operating conditions as defined by your answer to Question (a) what action will be taken? In the event that the water level in any of these observation wells exceeds elevation 610, what action will be taken?

(c) Where will the observation wells in the plant fill area be located that will be monitored during the plant lifetime? At what depths will the screened intervals be? Will the combination of (1) screened interval in cohesionless soil and (2) demonstration of timely response to changes in cooling pond level prior to drawdown be made a condition for selecting the observation wells? Under what conditions will the alarm mentioned on page 24-20 be triggered? What will be the response to the alarm? A worst case test of the completed permanent dewatering and groundwater level monitoring systems could be conducted to determine whether or not the time required to accomplish shutdown and cooling is available. This could be done by shutting off the entire dewatering system when the cooling pond is at elevation 627 and determining the water level versus time curve for each observation well. The test should be continued until the water level under Category I structure, whose foundations are potentially liquefiable, reaches elevation 610 (the normal water level) or the sum of the time intervals allotted for repair and the time interval needed to accomplish shutdown (should the repair prove unsuccessful) has been exceeded, whichever occurs first. In view of the heterogeneity of the fill, the likely variation of its permeability and the necessity of making several assumptions in the analysis which was presented in the applicant's response to Question 24g, a full-scale test should give more reliable information on the available time. In view of the above the applicant should furnish his response to the following:

If a dewatering system failure or degradation occurs, in order to assure that the plant is shutdown by the time water level reaches elevation 610, it is necessary to initiate shutdown earlier. In the event of a failure of the dewatering system, what is the water level or condition at which shutdown will be initiated? How is that condition determined? An acceptable method would be a full-scale worst-case test performed by shutting off the entire dewatering system with the cooling pond at elevation 627 to determine, at each Category I Structure deriving support from plant fill, the water level at which a sufficient time window still remains to accomplish shutdown before the water rises to elevation 610. In establishing the groundwater level or condition that will trigger shutdown, it is necessary to account for normal surface water inflow as well as groundwater recharge and to assume that any additional action taken to repair the dewatering system, beyond the point in time when the trigger condition is first reached, is unsuccessful.

(2) As per applicant response to NRC Question 24 (10 CFR 50.54f) the design of the permanent dewatering system is based upon two major findings: (1) the granular backfill materials are in hydraulic connection with an underlying discontinuous body of natural sand, and (2) seepage from the cooling pond is restricted to the intake and pump structure area, since the plant fill south of Diesel Generator Building is an effective barrier to the inflow of the cooling pond water. However, soil profiles (Figure 24-2 in the "Response to NRC Requests Regarding Plant Fill"), pumping test time-drawdown graphs (Figure 24-14), and plotted cones of influence (Figure 24-15) indicate that south of Diesel Generator Building, the plant fill material adjacent to

the cooling pond is not an effective barrier to inflow of cooling pond water. The estimated permeability for the fill material as reported by the applicant is 8 feet/day and the transmissivities range from 29 to 102 square feet/day. Evaluate and furnish for review the recharge rate of seepage through the fill materials from the south side of the Diesel Generator Building on the permanent dewatering system. This evaluation should especially consider the recovery data from PD-3 and complete data from PD-5.

(3) The interceptor wells have been positioned along the northern side of the Water Intake Structure and service water pump structures. The calculations estimating the total groundwater inflow indicate the structures serve as a positive cutoff. However, the isopachs of the sand (Figures 24-9 and 24-10) indicate 5 to 10 feet of remaining natural sands below these structures. The soil profile (Figure 24-2) neither agrees nor disagrees with the isopachs. The calculations for total flow, which assumed positive cutoff, reduced the length of the line source of inflow by 2/3. The calculations for the spacing and positioning of wells assumed this reduced total flow is applied along the entire length of the structures. Clarify the existence of seepage below the structures, present supporting data and calculations, and reposition wells accordingly. Include the supporting data such as drawdown at the interceptor wells, at midway location between any two consecutive wells, and the increase in the water elevations downstream of the interceptor wells. The presence of structures near the cooling pond appears to have created a situation of artesian flow through the sand layer. Discuss why artesian flow was not considered in the design of the dewatering system.

(4) Provide construction plans and specification of permanent dewatering system (location, depths, size and capacity of wells, filterpack design) including required monitoring program. The information furnished in response of NRC Question 24 (10 CFR 50.54f) is not adequate to evaluate the adequacy of the system.

(5) Discuss the ramifications of plugging or leaving open the weep holes in the retaining wall at the Service Water Building.

(6) Discuss in detail the maintenance plan for the dewatering system.

(7) What are your plans for monitoring water table in the control tower area of the Auxiliary Building?

(8) What measures will be required to prevent incrustation of the pipings of the dewatering system. Identify the controls to be required during plant operation (measure of dissolved solids, chemical controls). Provide basis for established criteria in view of the results shown on Table 1, page 23 of tab 1-7.

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(9) Upon reaching a steady state in dewatering, a groundwater survey should be made to confirm the position of the water table and to insure that no perched water tables exist.

Dewatering of the site should be scheduled with a sufficient lead time before plant start up so that the additional settlement and its effects (especially on piping) can be studied. Settlement should be closely monitored during this period.

Provide your plans for conducting this groundwater survey.

N/A j. Liquefaction Potential.

An independent Seed-Idriss Simplified Analysis was performed for the fill area under the assumption that the groundwater table was at or below elevation 610. For 0.19 g peak ground surface accceleration, it was found that blow counts as follows were required for a factor of safety of 1.5:

Elevation ft	Minimum SPT Blow Count*[1] For F.S. = 1.5
610	14
605	16
600	17
595	19

The analysis was considered conservative for the following reasons (a) no account was taken of the weight of any structure, (b) liquefaction criteria for a magnitude 6 earthquake were used whereas an NRC memorandum of 17 Mar 80 considered nothing larger than 5.5 for an earthquake with the peak acceleration level of 0.19 g's, (c) unit weights were varied over a range broad enough to cover any uncertainty and the tabulation above is based on the most conservative set of assumptions. Out of over 250 standard penetration tests on cohesionless plant fill or natural foundation material below elevation 610, the criteria given above are not satisfied in four tests in natural materials located below the plant fill and in 23 tests located in the plant fill. These tests involve the following borings:

SW3, SW2, DG-18, AX 13, AX 6, AX 15, AX 7, AX 5, AX 11, DG 19, DG 13, DG 7, DG 5, D 21, GT 1, 2.

Some of the tests on natural material were conducted at depths of at less than 10 ft before approximately 35 ft of fill was placed over the location. Prior to comparison with the criteria these tests should be multiplied by a factor of about 2.0 to account for the increase in effective overburden pressure that results from the placement and future dewatering of the fill.

[1]*For M = 7.5, blow counts would increase by 30%.

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Of the 23 tests on plant fill which fail to satisfy the criteria, most are near or under structures where remedial measures alleviating necessity for support from the fill are planned. Only 4 of the tests are under the Diesel Generator Building (which will still derive its support from the fill) and 3 others are near it. Because these locations where low blow counts were recorded are well separated from one another and are not one continuous stratum but are localized pockets of loose material, no failure mechanism is present.

In view of the large number of borings in the plant fill area and the conservatism adopted in analysis, these few isolated pockets are no threat to plant safety. The fill area is safe against liquefaction in a Magnitude 6.0 earthquake or smaller which produces a peak ground surface acceleration of 0.19 g or less provided the groundwater elevation in the fill is kept at or below elevation 610.

48. ~~4~~. Seismic analysis of structures on plant fill material.

(1) Category I Structures. From Section 3.7.2.4 of the FSAR it can be calculated that an average V_s of about 1350 ft/sec was used in the original dynamic soil structure interaction analysis of the Category I structures. This is confirmed by one of the viewgraphs used in the 28 February Bechtel presentation. Plant fill V_s is clearly much lower than this value. It is understood from the response to Question 13 (10 CFR 50.54f) concerning plant fill that the analysis of several Category I structures are underway using a lower bound average V_s = 500 ft/sec for sections supported on plant fill and that floor response spectra and design forces will be taken as the most severe of those from the new and old analysis. The questions which follow are intended to make certain if this is the case and gain an understanding of the impact of this parametric variation in foundation conditions.

(a) Discuss which Category I structures have been and/or will be reanalyzed for changes in seismic soil structure interaction due to the change in plant fill stiffness from that envisioned in the original design. Have any Category I structures deriving support from plant fill been excluded from reanalysis? On what basis?

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

(c) Is it the intent to analyze the adequacy of the structures and their contents based upon the envelope of the results of the old and new analyses? For each structure analyzed, please show on the same plot the old, new, and revised enveloping floor response spectra so the effect of the