DEPARTMENT OF THE ARMY
DETROIT DISTRICT, CORPS OF ENGINEERS
BOX 1027
DETROIT, MICHIGAN 48231

27 MAR 1980

NCEED-T

SUBJECT: NRC Midland Project, Request for Additional Borings and Existing Soil Data

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

Dear Dr. Jackson:

1. The Detroit District Corps of Engineers in providing geotechnical assistance to the Nuclear Regulatory Commission concerning the Midland Nuclear Plant requires additional soil borings and related soil test data as described in Inclosure 1.

2. The requested borings and related soil test data should be provided as soon as possible. Delays in receipt of this information would delay completion of the interagency agreement subtasks.

FOR THE DISTRICT ENGINEER

P. McCallister

1 Incl
As stated

P. McCALLISTER
Chief, Engineering Division

This letter superseded by resubmitted letter of 16 April 1980 from P. McCallister

INCLOSURE 1

1. It is requested that the applicant furnish the boring logs listed below indicating when and how these were taken, the type of sampling, and samples taken:

Pull down holes PD-1 thru PD-27* (35 holes)
LOW-1 thru LOW-13 & W-1 thru W-4 (18 Holes)
TW-1 thru TW-5 & PZ-1 thru PZ-48 (53 holes)
OW-1 thru OW-3 & OL-1 thru OL-6 (9 holes)
TEW-1 thru TEW-7 & Q-1 thru Q-12 (19 holes)
*Includes 8A, 20A, 20B, 20C, 15A, 15B, 15C, & 27A.

2. Locations, boring logs and test data from any other drill holes taken in 1979 and 1980 are also requested.

3. Dutch cone penetrometer data from holes P-1 thru P-13 must also be provided.

4. Information is requested on all piezometers that were installed to monitor problems related to plant fill. The information should include the number and location, the time of installation, the type of filter around the piezometer, the installed depth, and the type of piezometer.

5. All piezometer readings for each installation with dates and times are required.

6. The data and information requested in paragraphs 1 thru 5 above is needed to verify the applicant's computations and conclusions and to make any needed computations for the dewatering analysis, the seismic analysis and the settlement analysis.

7. A need exists for additional borings, since random exploratory borings throughout the plant site have revealed pockets of soft clay subject to settlement and or consolidation and loose sands subject to liquefaction. A need also exists to check the results of the proposed remedial measures of surcharge loading at the Diesel Generator Building and the dewatering plan.

a. In the case of the Diesel Generator Building, check borings must be made in the vicinity of borings which identified low "N" values in the clay and sand fill. The proposed borings shall be carried into the glacial till and all samples including those in the glacial till tested as indicated below.

The boring locations are as indicated on the attached map. All soil for the full depths of the borings shall be classified according to the Unified Soil Classification System. Any tests necessary to classify the soil shall be accomplished. Unit weight and moisture content of all samples should also be determined. The samples obtained from any cohesive strata shall be tested. The tests for cohesive material shall be an unconsolidation triaxial shear test and a consolidation test with restraining load equal to the load in place at the strata depth the sample represents [redacted]. The sands shall be tested in direct shear for a loose and dense condition and the relative density of the sand in situ determined.

b. Where piling or caissons are proposed to underpin the Service Water Building and Auxiliary Building - feed water valve pits which are located on fill, the load bearing capacity of the bearing strata must be determined. The capability to resist lateral shearing stresses that could be induced in low "N" value soil subjected to seismic action must also be determined. The same tests required for soil samples obtained from the new borings at the Diesel Generator Building shall also be made on soil samples from new borings for these buildings.

c. The questionable site area fill may have a counterpart in the cooling pond embankmen which was constructed contemporaneously with the site fill. It is requested that exploratory continuous drive borings be taken at a number of points along the north and east embankments, omitting the slurry trench cutoff areas which are positively sealed. The approximate boring locations are as indicated on the attached map of the cooling pond. The tests on the soil samples obtained from the borings in the embankments shall include the following tests, unconsolidated, undrained triaxial shear tests, Atterberg limits and all soils classified according to the Unified Soil Classification System. The borings shall be sampled every 2-1/2 feet using a standard split spoon sampler. The hole shall be held open using a hollow stem auger or casing. Particular attention shall be paid to ground water conditions during and after completion of drilling. In the case of Hole 5, the boring should be drilled to the depth of the cooling pond bottom while the remaining borings need penetrate only 5 feet into underlying residual soils unless soft ground indicates a need for further hole penetration.

8. Summary of Requested Drilling

a. Diesel Generator Building - 4-6 holes around the perimeter of the building. Samples of all stratas from ground surface into the glacial till (Holes 8-13).

b. Auxiliary Building - Take two borings around the proposed support piling or caisson for remedial grouting of loose sands and soft clays adjacent to pile or caisson to stiffen piles and adjoining ground against lateral loading. Borings need to penetrate to glacial till. (see attached map for boring locations - Holes 4 & 5.)

c. Service Water Building - A boring (Hole 16) shall be made as indicated on the attached map to and into the glacial till. All samples obtained shall be classified according to the Unified Soil Classification System also unconfined, undrained triaxial compression tests made on cohesive soil samples and direct shears for a loose and dense condition shall be made on all granular soil samples.

d. Plant Area Borings - Some borings must be taken under the Radwaste and Turbine Buildings to determine if unwatered pockets exist or persist. Suggested boring locations would be as indicated on the attached map. Further investigation could be needed after the results of these borings are obtained. No borings presently exist in these areas. The borings should be cased or hollow stem auger borings with drive samples every 2-1/2 feet through the fill should be taken and converted to dewatering holes or used for piezometers (Holes 1, 2, 3, 6 & 7).

e. The site visit of 27 or 28 February 1980 turned up two differential settlement points on the retaining wall adjacent to the Service Water Pump Structure. Two borings, Holes 14 and 15 as indicated on the attached map shall be taken to investigate this problem. Tests required are consolidation tests, triaxial compression tests, Atterberg limits and gradation tests made on cohesive soils, and direct shear for loose and dense conditions and gradation tests made on granular soils.

f. In all new borings made, the water table shall be determined.

Consumers Exhibit #5 (Kane)

J. Kane

DISPOSITION FORM

Rec'd 2/22/80 from L. Heller

For use of this form, see AR 340-15, the proponent agency is The Adjutant General's Office.

REFERENCE OR OFFICE SYMBOL	SUBJECT
NCEED-T	Geotechnical Engineering Assistance to NRC Orientation Meeting at the Bethesda, Maryland 7-8 November 1979

TO	FROM	DATE	CMT 1
NRC File	Kubinski	1 Feb 80 KUBINSKI/vw/6786	

1. The purpose of this trip was orientation in nature. It was made to acquaint R. Erickson and J. Kubinski with the NRC Organization, staff, project requirements, and facilities available at their main office in Bethesda, Maryland.

2. The meetings took place on the 7-8 November 1979. I will refer to the meeting that took place on the 7th as Meeting I, and the meeting that took place on the 8th as Meeting II.

3. The following are significant items discussed at the respective meetings:

a. Meeting I: This meeting was primarily orientation in nature. NCE personnel were introduced to the NRC staff, their organizational elements and in general their function as a review agency. Dave Lynch of NRC gave a concise presentation on the general mission, and referencing specifically Bailly Nuclear Generating Station near Gary, Indiana. He also covered elements in the normal review process giving an indication as to general requirements. Later, he covered the more technical aspects and problems in existence at the site.

b. Meeting II: This meeting was also of orientation nature, with the emphasis placed on the Midland Nuclear Facilities. This meeting was very similar in nature to the one on Bailly, but was conducted with emphasis on the Midland site.

4. The following people were involved in these meetings:

a. Meeting I:

Bob Jackson (NRC)
Lyman Heller (NRC)
Dave Lynch (NRC)
J. Kubinski (NCE)
R. Erickson (NCE)

b. Meeting II:

Lyman Heller (NRC)
Darl Hood (NRC)
Dan Gillen (NRC)
J. Kubinski (NCE)
R. Erickson (NCE)

DA FORM 2496, 1 FEB 62 — REPLACES DD FORM 96, EXISTING SUPPLIES OF WHICH WILL BE ISSUED AND USED UNTIL 1 FEB 63 UNLESS SOONER EXHAUSTED. * U.S. GPO: 1974-540-841/8656

5. The items discussed are listed below:

a. Meeting I:

I. This meeting was of orientation nature and a good introduction to the entire program was given by Dave Lynch, Project Manager, NRC, Bailly Nuclear Generating Station.

II. The purpose of NRC's mission with respect to review is to insure radiological safety and containment of all possible danger. It is not NRC's concern to see that OASHA standards or safety in general are observed.

III. The issue at Bailly is concerned with piles supporting primary containment facilities. It is a rigid structure and, therefore, no displacement can be tolerated. Dynamic operations result in displacement and this displacement must be monitored so that the entire structure is adjusted accordingly. There is a very well-defined load/deflection analysis for the entire facility.

IV. The containment facility cannot fail. It may have to be politically safe which implies a greater than necessary safety factor to be technically safe.

V. The Safety Evaluation Report (SER) has not yet been written for the Bailly plant.

VI. It is necessary to defend any technical judgments before the Advisory Committee for Reactor Safety (ACRS). At the Bailly site it will be necessary to defend as built conditions.

VII. The term "Intervener" is defined as follows: An intervener must live within 50 miles of the proposed facility (the State in which the facility exist can act as an intervener); the interveners may hire firms or individuals to represent them in obtaining information concerning the construction or operation of nuclear facilities.

VIII. The normal review process consists of the following items:

- Applicant submits PSAR (Preliminary Safety Analysis Report)

- NRC writes Safety Evaluation Report (SER). This SER is a concise picture of NRC staff's review.

- NRC submits SER to Advisory Committee on Reactor Saftey (ACRS). The ACRS can form subcomittees in which their members and/or their consultants can evaluate the specific issues.

- ACRS evaluates SER/PSAR and letter on the safety of the plant is written.

- Public hearings are generated only if the license is thought to be able to be granted. This is a construction license.

- The Construction Permit, issued by NRC, but license is granted by the Chairman of the Commission.

- The review of deviations from the PSAR, SER and CP must be reported by the applicant to the Nuclear Regulatory Commission Office of Inspection and Enforcement (I&E). The I&E Office sends this information to the review office for review, and a new license or amended license is usually issued.

NOTE: The following is a list of items concerning the Bailly plant.

IX. The construction permit for Bailly Plant consist of non-displacement high capacity piles which go to bedrock or glacial till and support a concrete mat foundation. They are embedded in concrete approximately three feet.

X. A brief driving history for the piles is as follows. In driving the piles, stiffening occurred at 55 feet. Blow counts from 200 to 300 blows per inch were experienced. The till material is at about 110 feet and bedrock is at 120 feet. Above a very stiff clay deposit which is wedge shaped in profile, intermittent sands and clays are the overburden material. This stiffening occurs in a very dense sand above this larger clay deposit.

XI. In May 1974 the construction permit called for a test pile program which indicated significant problems in driving. Shortly after that, NIPSCo came in with a short pile proposal. In September 1977 an alternate proposal to jet long piles was submitted. A test program was initiated and in February 1978, the NRC issued an order to jetting the piles. In jetting the piles, the soil reacted similar to a giant wash boring (1,000 gallons per minutes at 300 PSI). The area of disturbance was much too large and the pile was actually loose near the surface. The nature of the structure which was to be supported by these piles demanded that the piles have uplift capacity. Because of the disturbance and lack of uplift capacity, the short pile concept is once against an issue as of March 1978. These piles would develope end bearing and friction. The applicant was allowed to drive 100 piles as indicators to determine capacities and applicability of using the short pile concept. A cluster was driven to observe heave within the piles. This brings us to the current state of the issue.

XII. It is now the task of the NRC review to look at all of the above submittals and reconsider the entire issue. They must also determine if construction restrictions are required or further load test are required. The jetting procedures have made soft spots which encompress almost five percent of the area of the foundation. These loosened areas must be densified and a technique developed to insure that they develop all lateral capacities as well as uplift capacities.

XIII. The Advisory Committee on Reactor Safety (ACRS) has already indicated that nothing was substantially wrong with use of short piles to provide substantial foundation. That is, that there is no deflection in the piles and that all the disturbed areas due to the jetting procedures are densified.

XIV. It is apparent that now it is necessary to look at the PSAR and become fully familiar with it as well as considering the groundwater affect on the foundation.

XV. NCE will have to prepare the entire Safety Evaluation Report (SER) and not just assist in its preparation. A sample Saftey Evaluation Report is available from NRC and will be transmitted.

NOTE: The last item is of general nature.

XVI. The hearing of a Hearing Board process can be described as follows. Administrative law judge act as the Chairman. Engineer Scientists and some technical people drawn from university staff act as part of the committee. The commission delegates authority to the Board, the Board inturn can dictate policy. The Board can question any item and the interveners' attorney can question around items brought up by the Board. It is, therefore, necessary to minimize any questions the Board may have by clear concise presentations.

XVII. NCE will meet with Newmark, Hall and Davison at Champagne (University of Illinois) concerning the piling issue sometime in January or February.

b. Meeting II:

This meeting was of a briefer nature than Meeting I. At this meeting Joe Kane (NRC) and Darl Hood (NRC Project Manager) presented an introduction concerning issues at the Midland Nuclear Facility.

I. As a preliminary to the meeting, the following items were discussed. A brief discussion on what safe shutdown earthquake (SSE) or an operating base earthquake (OBE) were head. Appropriate volumes of the Preliminary Safety Analysis Report (PSAR) were to be sent to NCE as soon as possible. The applicant, Consumers Power Company (CPCo), must still respond to original I&E questions on the interim report and on 10CFR 50.54(f). There is apparently a report or a paper on the dewatering system.

II. The I&E Office (Inspection and Enforcement) is investigative in nature and generally goes to the NRR (Nuclear Regulatory Review) for support. The I&E Office considered the overall performance of the applicant as well as the technical adequacy of any field changes. The viability of the Quality Assurance Program is also investigated by this group.

III. The current state of the review is one in which the construction permit should be suspended, modified or revoked by the Commission. One of these actions is necessary to take concerning the quality assurance breakdown at the Midland site as well as the inadequate fill in support of Category I structures.

IV. Questions of a non-policy nature can go directly to the applicant. No commitment is considered to be binding between NCE and the applicant. Once these questions are established and they are addressed to the applicant, they should be documented especially when they are relatively significant.

V. Construction inspections or visits to the site are necessary in performing the mission. NCE must be able to reply (we saw) in reference to a specific issue if possible.

VI. More than one visit is in most cases necessary, since sequential events will be occurring in the fixing of unstable conditions at the site.

VII. The NRC Office of Inspection and Enforcement has a fulltime man at the site, and he can be contacted concerning observing any action at the site.

VIII. Meeting concluded with two immediate items of major concern:

a. Should the existing license be modified, suspended or revoked.

b. A list of visits and times sequentially established in the future.

6. These meetings were of orientation in nature and it is difficult to establish any conclusions. The actions to be taken in the future are ones concerning scheduling field trips and site visits, carrying out orientation procedures with all documents transmitted, assuring that all documents have been transmitted and then beginning the review process and making either recommendations, comments, or conclusions regarding the situations at both facilities.

J Kubinski
J. KUBINSKI
Technical Branch

CONCURRENCE:

____
R. Erickson

____
L. Heller (NRC)

CONSUMERS
EXHIBIT #6 - (KANE)

7/25/80

MIDLAND — Understanding of NRC Position (In Anticipation of Report)

Establish the following:

— The problems which have developed at the Midland site were not caused by NRC actions.

— These problems now require a more intense scrutiny by the NRC staff than is normally covered in our reviews. This increased level of staff review is needed to permit the staff to FULLY understand the proposed solutions and to be in a position where we can either defend acceptance of the solutions or CLEARLY identify our concerns so that they may be addressed & resolved.

— We suggest CPCo carefully consider the intent of our questions and request for information. If the information being requested is the type of information that CPCo's consultants or contractors must have originated and evaluated to come to a conclusion on adequacy or margin of safety, then there is no reasonable basis for CPCo to object to the staff's request for this information

Consumers Exhibit # 7 (Kane)

-- Preparation of a Safety Evaluation Report (SER) input which describes the evaluation of the design of the applicants' safety related (and some non-safety related) systems.

-- Attend meetings with the Advisory Committee on Reactor Safeguards (ACRS) and public hearings to assist the staff in explaining bases for conclusions and positions reached in the SER.

-- Preparation of input to SER Supplements which further clarify and document systems evaluations in the SER based upon review by the ACRS.

The geotechnical engineering aspects of proposed nuclear plant facilities to be evaluated generally include the stability and settlement of safety related structures, emergency cooling water reservoirs, appurtinent, safety-related structures such as earth embankments and rock fill dams, canals, weirs, intake and discharge structures, and pipelines, under both static and dynamic conditions, including the subjection of dams, etc., to the Safe Shutdown and Operating Basis Earthquakes. The evaluation typically consists of:

1. A review of the site investigation program, both field and laboratory, to assure that an adequate determination of all subsurface conditions has been achieved including consideration of borrow sources. This may require recommendations for additional investigations to obtain the required data;

2. Evaluations and recommendations pertaining to the proposed design criteria;

3. A review of the stability and settlement analysis performed by the applicant and, in many cases, the performance of independent stability analysis. A determination that the applicant has presented adequate bases to support the design parameters used in his analysis;

4. An evaluation of stabilization techniques proposed by applicants to solve site foundation problems. In many cases, the contractor will be asked to provide recommendations for stabilization;

5. In regard to most cases, field trips by contractor personnel will be necessary to inspect the site, to observe sampling and testing of soil and rock, and to evaluate the adequacy of techniques and equipment.

Specific Work Requirements

Task 1 - Midland Plant Units 1 and 2

Technical Monitor: J. Kane

Estimated Manpower: 3 Man-years

The contractor shall review the FSAR (with amendments and documents related to the 10 CFR 50.54 (f) request regarding plant fill which have been submitted to NRC on the subject plant for the purpose of obtaining an OL.

This review shall include an evaluation of all the information included in Section 2.5, 3.7 and 3.8 of the FSAR and 10 CFR 50.54 (f) documents which address the adequacy of soil and rock mechanics, earthquake engineering and foundation engineering design and construction aspects in order to assure the safe siting and operation of all seismic category safety-related structures and conduits. The review should be conducted in accordance with NRC Standard Review Plans Sections 2.5.1, 2.5.2 and 2.5.4. Specific guidance on design methods which are acceptable to the NRC staff that have been made available to applicants in their designs include Regulatory Guides 1.132, 1.138 and 1.70 (Section 2.5).

Subtasks	Estimated Completion Date
1. Review and evaluate the information contained in the above NRC 10 CFR 50.54 (f) documents regarding plant fill in accordance with acceptance criteria outlined in the related Standard Review Plans. Meet with the NRC staff and applicant as required. Make site visits to observe remedial methods and procedures. Prepare a letter report identifying any unresolved issues with recommendations on a course of action to be taken during construction to resolve these issues.	12/79
2. Review and evaluate the information contained in the above FSAR Sections in accordance with acceptance criteria outlined in the related Standard Review Plans. Meet with the NRC staff as required, prepare a draft SER identifying any unresolved issues. Participate in approximately ten meetings with the applicant and the NRC staff to resolve the issues identified in the above draft SER.	1/80
3. Prepare a final SER. This SER may contain open issues or describe areas in which the contractor and staff continue to differ with an applicant.	3/80
4. Participate at a maximum of six ACRS meetings, prepare testimony for and appear at Licensing Board Hearings as required.	6/80
5. Review and evaluate any unresolved or open issues identified in the SER, or issues raised at ACRS meetings and in hearings. Participate in a maximum of five meetings with the applicant and the NRC staff to resolve any outstanding issues. Prepare inputs to SER supplements and Technical Specifications to complete the resolution of all outstanding issues, as required.	8/80

Specific Work Requirements

Task 2 - Bailly Generating Station - Nuclear 1

Technical Monitor: L. Heller

Estimated Required Manpower: 1 Man-year

The contractor shall review the FSAR with amendments and documents related to the pile foundations which have been submitted to the NRC on the subject plant for the purpose of obtaining an OL.

This review shall include an evaluation of all the information included in Section 2.5, 2.7 and 3.8 of the FSAR which address the adequacy of soil and rock mechanics, earthquake engineering and foundation engineering design and construction aspects in order to assure the safe siting and operation of all seismic Category I safety-related structures and conduits. The review should be conducted in accordance with NRC Standard Review Plans Sections 2.5.1, 2.5.2 and 2.5.4. Specific guidance on design methods which are acceptable to the NRC staff that have been made available to applicants in their designs include Regulatory Guides 1.132, 1.138 and 1.70 (Section 2.5).

Subtasks	Estimated Completion Date
1. Review and evaluate the information contained in the documents regarding plant pile foundation in accordance with acceptance criteria outlined in the related Standard Review Plans. Meet with the NRC staff, NRC consultants and applicant as required. Prepare a letter report identifying any unresolved issues with recommendations on a course of action to be taken during construction to resolve these issues.	12/79
2. Review and evaluate the information contained in the above FSAR Sections in accordance with acceptance criteria outlined in the related Standard Review Plans. Meet with the NRC staff as required, prepare a draft SER identifying any unresolved issues. Participate in approximately six meetings with the applicant and the NRC staff to resolve the issues identified in the above draft SER.	3/80
3. Prepare a final SER. This SER may contain open issues or describe areas in which the contractor and staff continue to differ with an applicant.	5/80
4. Participate at a maximum of five ACRS meetings, prepare testimony for and appear at Licensing Board Hearings as required.	6/80
5. Review and evaluate any unresolved or open issued identified in the SER, or issues raised at ACRS meetings and in hearings. Participate in a maximum of five meetings with the	9/80

applicant and the NRC staff to resolve any outstanding issues. Prepare inputs to SER supplements and technical specifications to complete the resolution of all outstanding issues, as required.

Reporting Requirements

1. Upon a completion of each subtask of each task the contractor will provide the cognizant NRC branch chief with a letter report which includes, as appropriate, safety evaluation report input testimony and supplemental safety report input.

2. A bi-monthly business letter report shall be submitted by the 20th of the month to the cognizant branch chief with a copy to the Director, Division of Systems Safety (Attn: B. L. Grenier). These reports will contain:

-- A listing of any efforts completed during the period; milestones reached, or if missed, an explanation provided;

-- The amount of funds expended during the period and cumulative to date;

-- Any problems or delays encountered or anticipated;

-- A summary of the progress to date;

-- Plans for the next reporting period;

-- The first bi-monthly letter report should contain the planned monthly rate of expenditure based upon review schedule established.

Note: These reports are not to be technical in nature.

Meetings and Travel

The contractor will attend approximately 10 meetings with the staff and applicants or with ACRS over the period of performance in Bethesda or Washington, D.C. and approximately 10 at each of the plant sites, A/E, or utility offices. These meetings will usually be of one or two days duration, however, on-site review meetings and observation of practices may extend over a longer period.

NRC Furnished Materials

Documents needed for review will be forwarded to the contractor under separate cover. Some of this material may contain propriety information, as marked, and must be kept in confidence by the contractor.

Billing Requirements

Vouchers submitted for payment should list expenditures for manpower and any other major items of expenditures for each task, i.e., for each separate plant.

Consumers Exhibit #8 (Kane)

DISCUSSION OF THE APPLICANT'S POSITION

ON THE NEED FOR ADDITIONAL BORINGS

FOR

MIDLAND PLANT UNITS 1 AND 2

CONSUMERS POWER COMPANY

DOCKET NUMBERS 50-329 AND 50-330

Report Date: September 14, 1980

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

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

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

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

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

1. That the additional borings are not necessary, and

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

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

A. DIESEL GENERATOR BUILDING

1. Settlement

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

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

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

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

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

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

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

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

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

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

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

2. Bearing Capacity[1]

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

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

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

where

C = cohesion intercept

N_c, N_q, N_γ = bearing capacity factors

γ = effective soil unit weight

D_f = foundation embedment depth

B = foundation width

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

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

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

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

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

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

B. SERVICE WATER STRUCTURE

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

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

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

DH/R

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

C. AUXILIARY BUILDING

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

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

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

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

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

D. COOLING POND DIKE

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

E. RETAINING WALL

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

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

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

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

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

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

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

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

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

REFERENCES

1. NRC Meeting, 8/29/80, Midland, Michigan

2. Responses to NRC Requests Regarding Plant Fill, Volume 3, Tab 7, letter from A.J. Hendron to S.S. Afifi, 10/23/78

3. Responses to NRC Requests Regarding Plant Fill, Volume 3, Tab 12, Bechtel Meeting Notes No. 882, 11/7/78

4. Responses to NRC Requests Regarding Plant Fill, Volume 4, Tab 75, letter from R.B. Peck to S.S. Afifi, 7/23/79

5. Responses to NRC Requests Regarding Plant Fill, Question 9

6. NRC letter to Consumers Power Company, Docket No. 50-329/330, 7/30/80; Table 37-1, Item 3

7. Responses to NRC Requests Regarding Plant Fill, Question 27

8. NRC Meeting, 7/31/80, Washington, D.C.

9. Responses to NRC Requests Regarding Plant Fill, Volume 3, Tab 70, letter from Mssrs. Peck, Hendron, Davisson, Loughney, and Gould to S.S. Afifi, 7/2/79

10. Responses to NRC Requests Regarding Plant Fill, Volume 3, Tab 57, letter from S.S. Afifi to Mssrs. Davisson and Hendron, 5/22/79

11. FSAR Subsection 2.5.4.3.2

12. NRC Meeting, 2/28/80 and 2/29/80, Midland, Michigan

13. Responses to NRC Requests Regarding Plant Fill, Volume 3, Tab 55, Meeting Notes, 5/10/79

14. Responses to NRC Requests Regarding Plant Fill, Volume 4, Tab 79, letter from C.H. Gould to S.S. Afifi, 8/3/79

15. Responses to NRC Requests Regarding Plant Fill, Question 12

16. FSAR Subsection 2.5.6.4

17. NRC Midland Site Meeting, Dike Tour, 8/28/80

18. Consumers Power Company letter to NRC, Serial 9697, 9/12/80, Settlement Update

TABLE 1

LABORATORY TEST DATA

SUMMARY OF SOIL PROPERTIES

TO DETERMINE p' - q' RELATIONSHIP

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

NOTES:

γ_d = dry unit weight

w = water content

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

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

Figure 1
(See Reference 1)

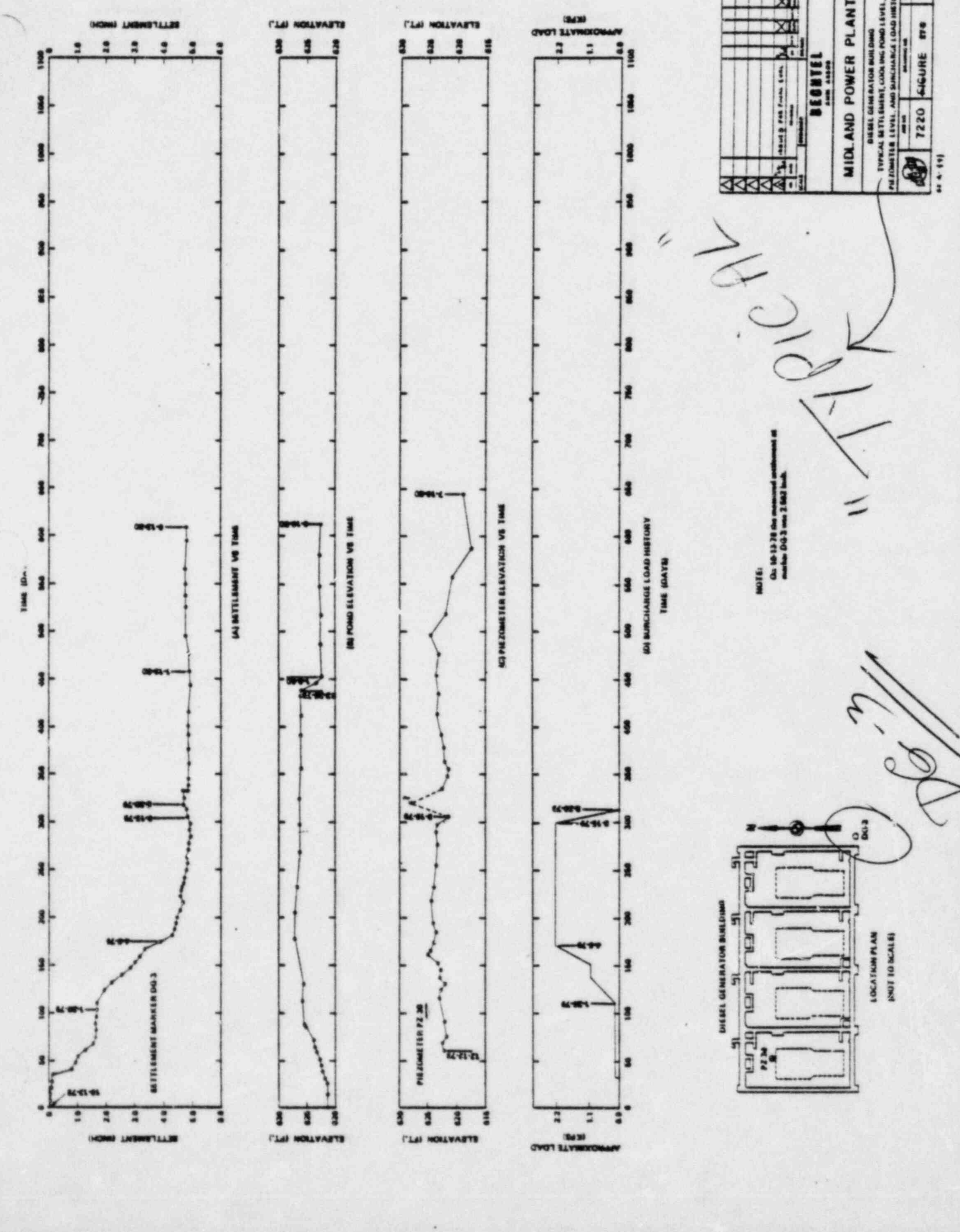
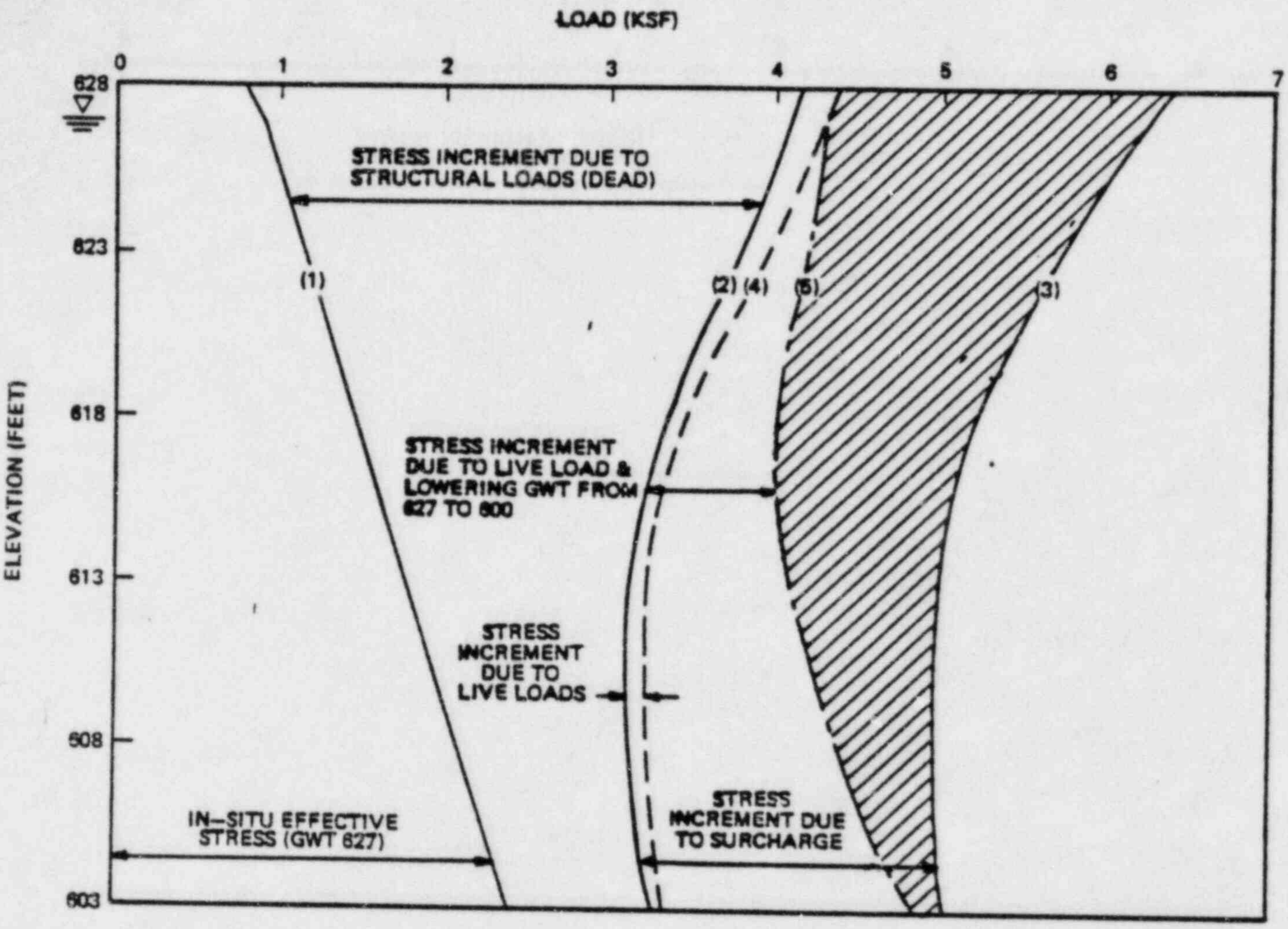


Figure 2
(See Reference 1)



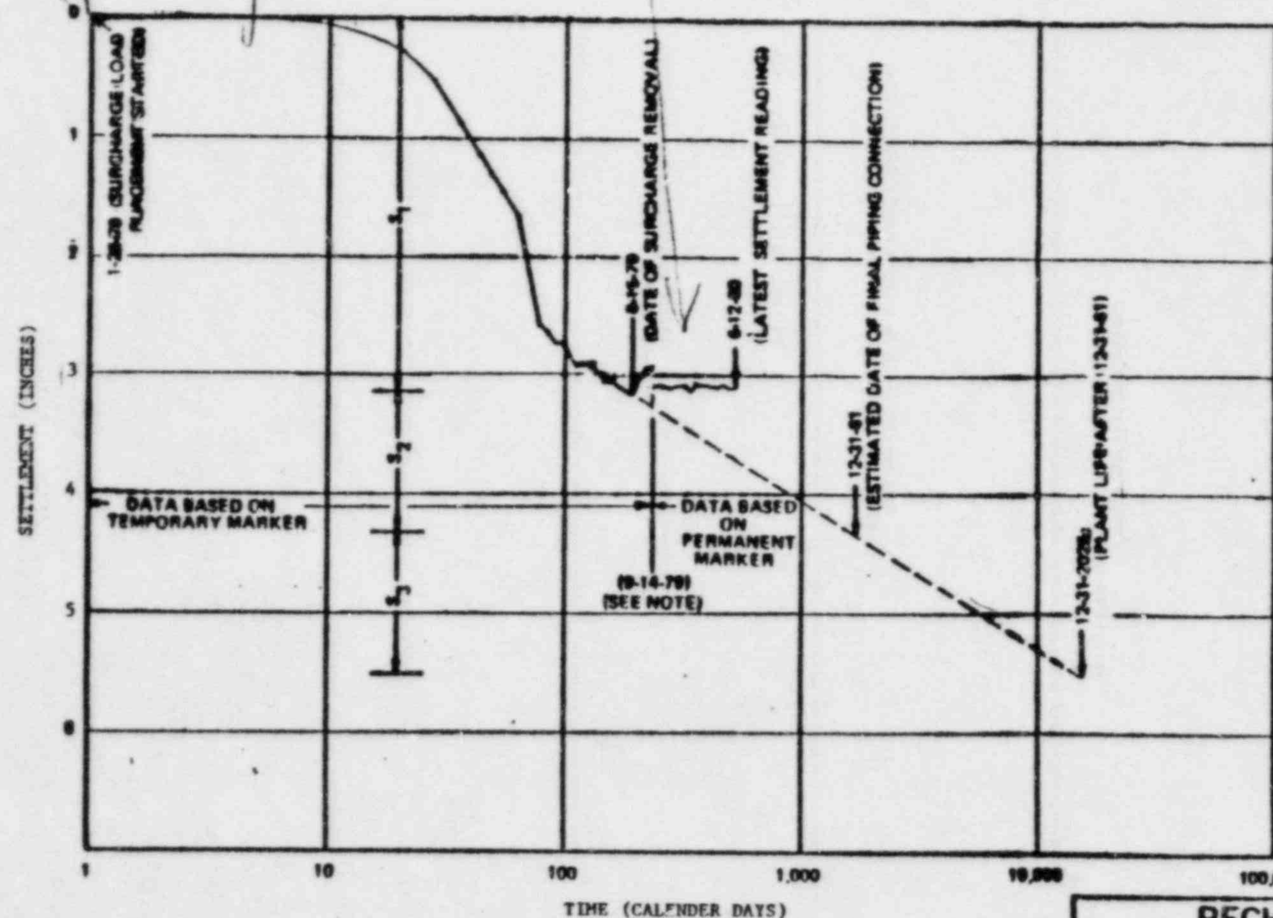
NOTES:

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

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

SOUTHWEST CORNER OF DIESEL GENERATOR BUILDING

Figure 3
(See Reference 1)



LEGEND

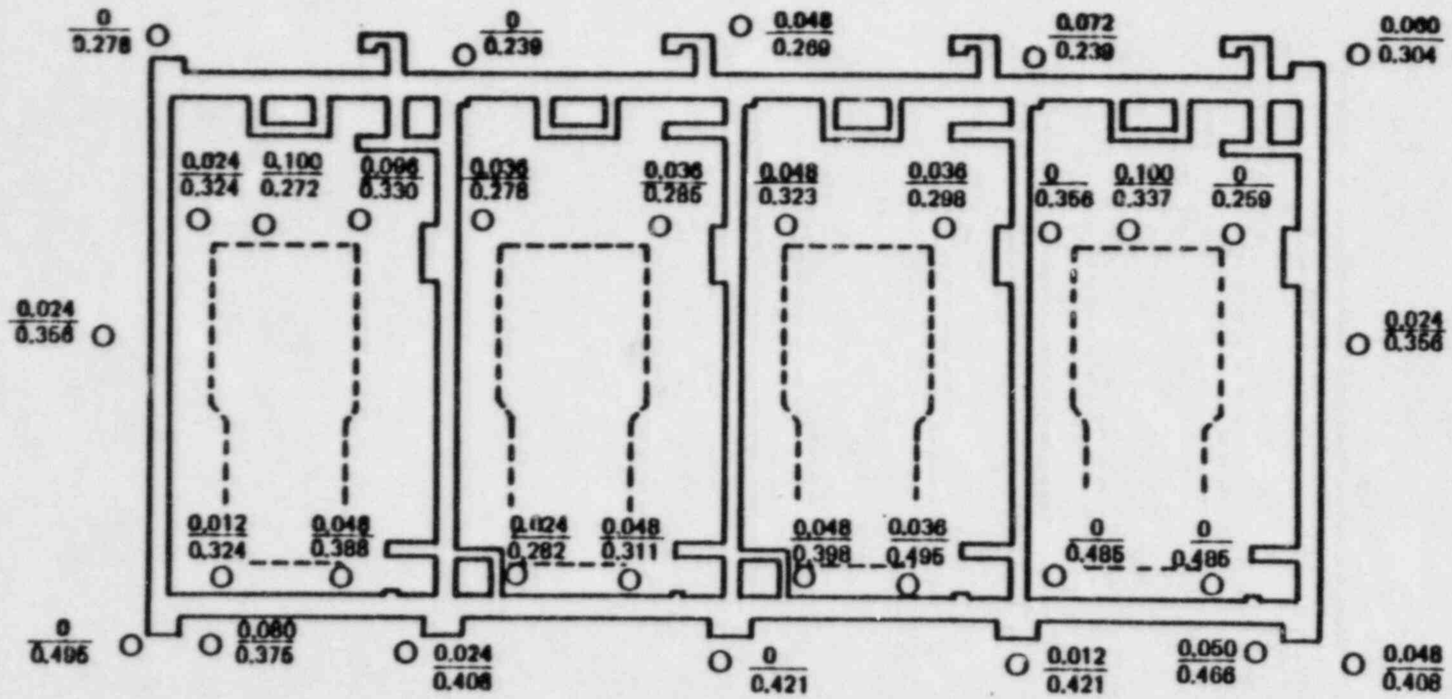
—— MEASURED SETTLEMENT

- - - PREDICTED SECONDARY COMPRESSION SETTLEMENT ASSUMING SURCHARGE REMAINS

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

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

DIESEL GENERATOR BUILDING



LEGEND:

O — BUILDING / PEDESTAL SETTLEMENT MARKER

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

NOTE:

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

Figure 4
(See Reference 1)

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

Figure 5
(See Reference 1)

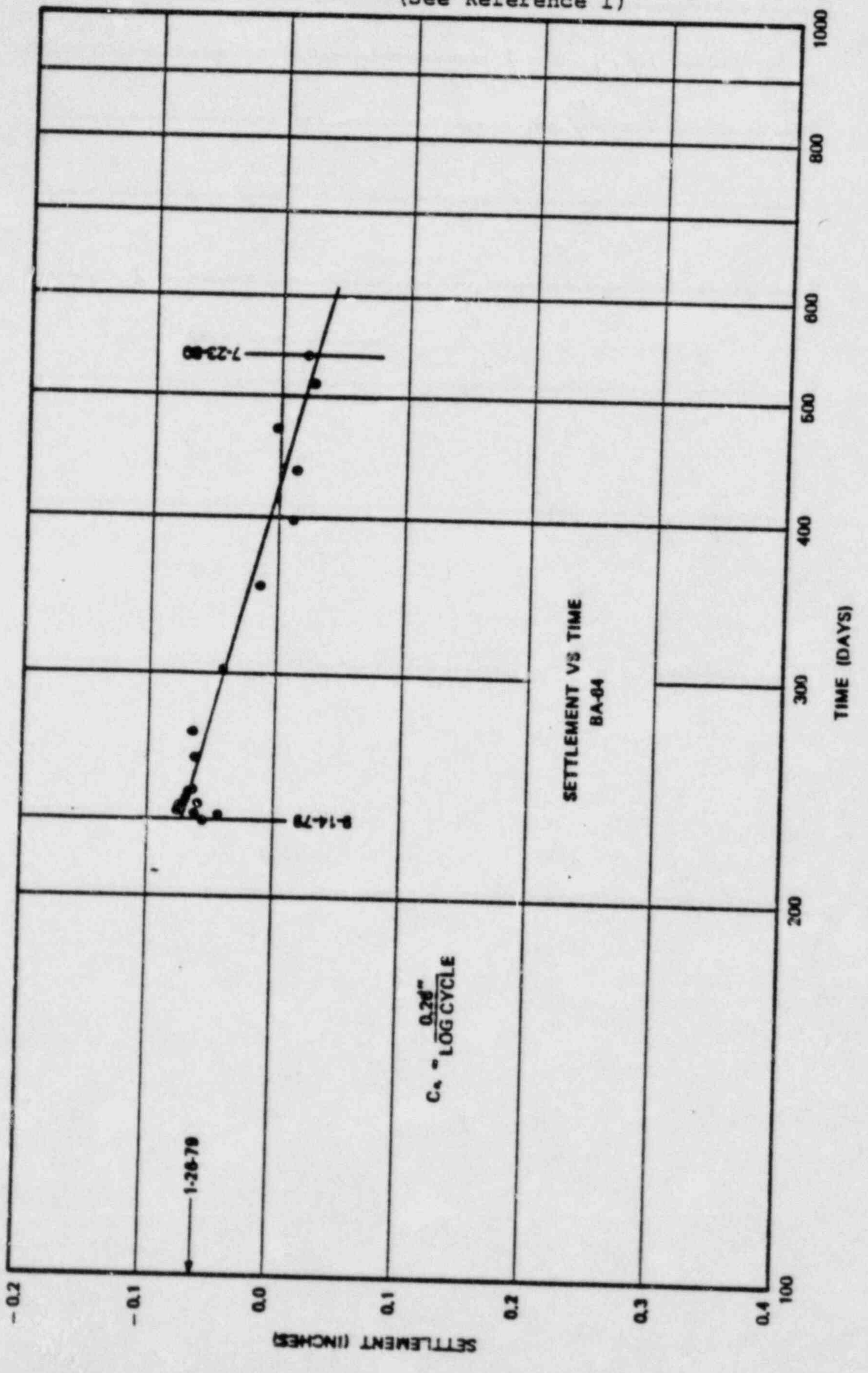


Figure 6
(See Reference 1)

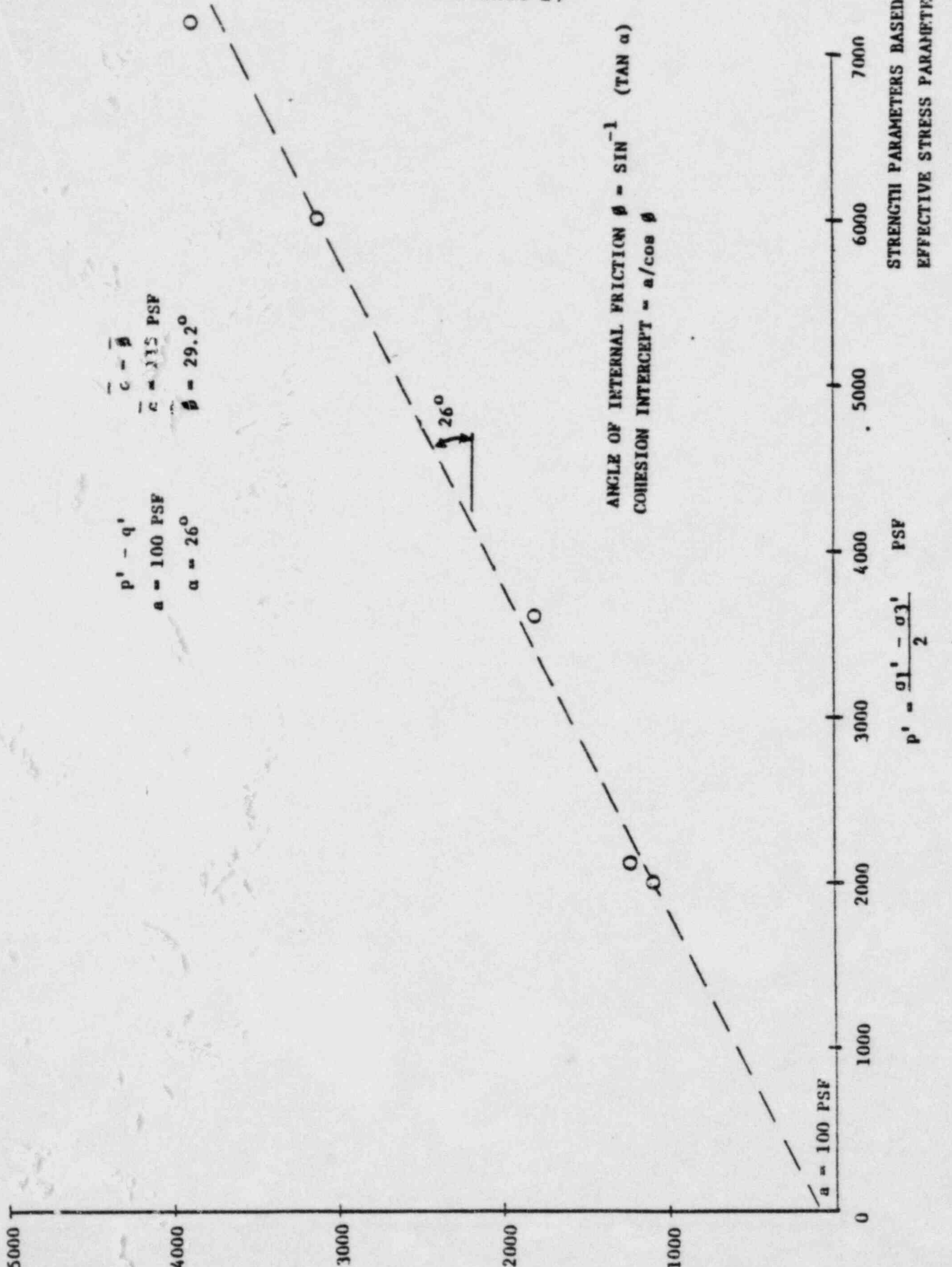
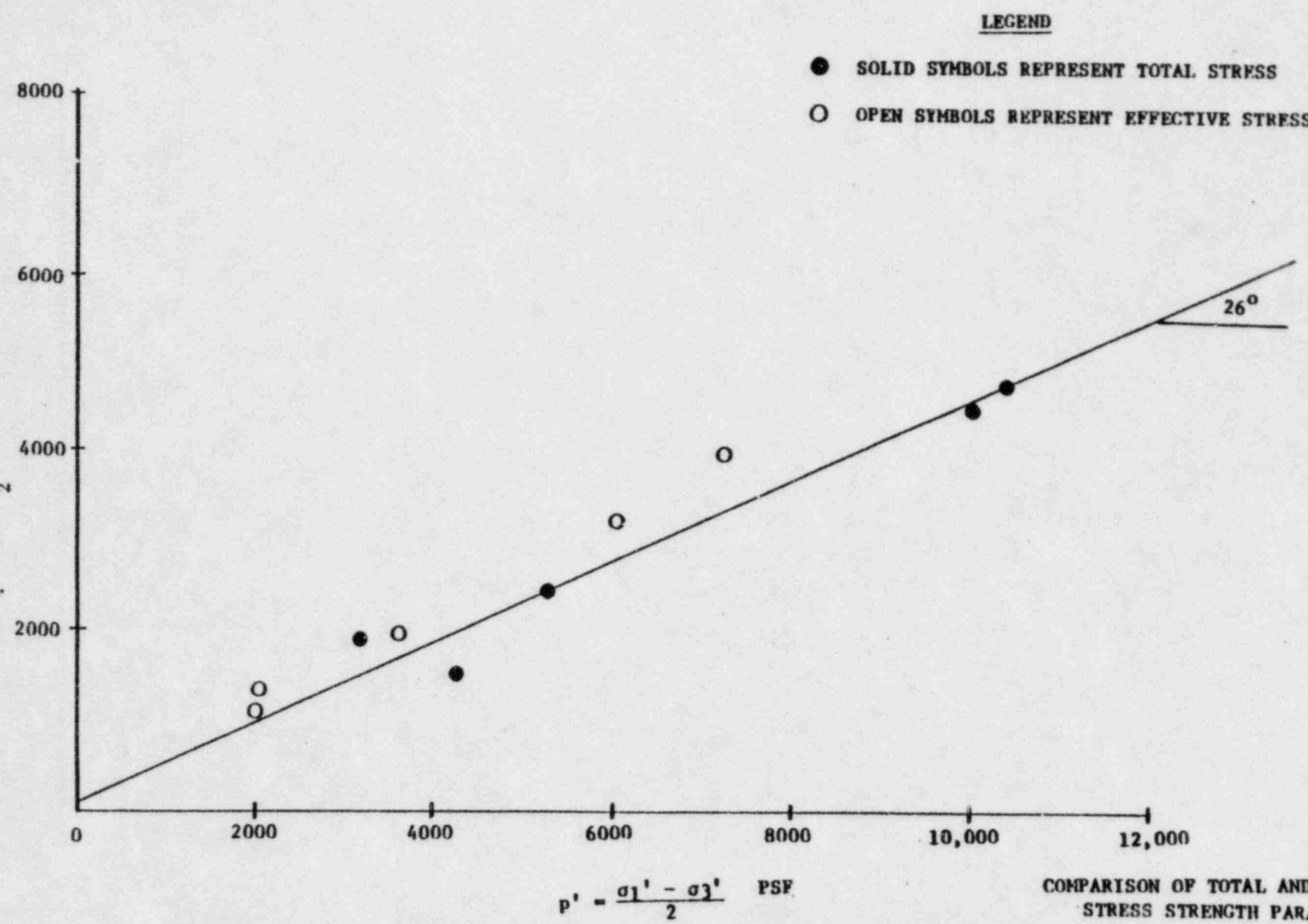


Figure 7
(See Reference 1)



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

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

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

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

$= 28395$ psf

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

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

b). Use Vesic

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

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

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

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

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

B. BASED ON FIVE SAMPLES WITH LOWER DENSITIES

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

$\bar{c} = 114$ psf

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

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

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

$= 24,453$ psf

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

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

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

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

$= 12,000 + 9,375$

$= 21,375$ psf

$(q_d)_{net} = 20,625$ 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 exper nce 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.

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

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

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

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

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

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

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

13.. Davisson, discussion of: "Buckling of Long, Unsupported Timber s," by E. J. Klohn and G. T. Hughes, Proceedings, ASCE, Vol. 91, SM4, July 1965, p. 224.

14.. Davisson, T. R. Maynard and V. G. Koile, "Static and Dynamic vior of Sands in One-Dimensional Compression," Technical Report AFWL-TR-65-29, AFWL, Kirtland Air Force Base, December 1965.

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

16.. Davisson, "Design of Deep Foundations for Tall Buildings Under ral Load," Proceedings, Structural Engineering In Modern Building gn, Illinois Structural Engineering Conference, Chicago, 1966, 57-174.

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

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

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

20. Davisson, discussion of: "Skin Friction for Steel Piles in " by Harry M. Coyle and I. H. Sulaiman, Proceedings, ASCE, 5, No. SM1, January 1969, pp. 373-374.

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

22. Davisson, "Static Measurements of Pile Behavior," Proceedings, ence on Design and Installation of Pile Foundations and ar Structures, Lehigh University, Bethlehem, April 1970, -164.

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

24. M. Davisson and J. R. Salley, "Model Study of Laterally Loaded P Proceedings, ASCE, Vol. 95, No. SM5, September 1970, P15-1627.

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

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

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

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

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

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

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

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

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

Professional Background and Experience

Name: Alfred J. Hendron, Jr.

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

Date of Birth: October 4, 1937

Marital Status: Married with 2 children

Citizenship: Natural Born - U.S.

Education

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

Positions Held

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

Alfred J. Hendron, Jr.

Offices held and other services to professional societies

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

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

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

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

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

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

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

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

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

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

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

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

Alfred J. Hendron, Jr.

Examples of Foundation Engineering and Earthquake Engineering Experience

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

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

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

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

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

6. Consultant, as an associate of Dr. N. M. Newmark, to the Divisions of Reactor Licensing and Reactor Safety of the Atomic Energy Comission, on the adequacy of nuclear reactor foundations 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 H. E. Auld, pp. 29-47, Proceedings of the International Symposium on Wave Propagation and Dynamic Properties of Earth Materials, University of New Mexico Press, 1968. "Mechanical Properties of Rock," Chapter 2, pp. 21-53, of the book "Rock Mechanics in Engineering Practice," edited by K. G. Stagg and O. C. Zienkiewicz, published by John Wiley & Sons, London, 1968, 442 pg.

Alfred J. Hendron, Jr.

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

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

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

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

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

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

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

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

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

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

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

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

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

NAME: Ralph B. Peck

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

D.C.E.
Rensselaer Polytechnic Institute

Post-doctoral studies, Engineering
Harvard University

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

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

EXPERIENCE and QUALIFICATIONS:

Summary

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

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

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

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

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