Response to NRC Review Questions

Prepared for

LaCrosse Boiling Water Reactor
Dairyland Power Cooperative
Genoa, Wisconsin 54632

Dames & Moore

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July 11, 1980

LaCrosse Boiling Water Reactor
Dairyland Power Cooperative
Post Office Box 135
Genoa, Wisconsin 54632

Attention: Mr. R. E. Shimshak
Plant Superintendent

Re: Response to NRC Review Questions

Gentlemen:

We are submitting ten copies of the report "Response to NRC Review Questions, Seismic Hazard and Liquefaction Potential at LACBWR Site, near Genoa, Vernon County, Wisconsin" for your use. This report contains answers to all the questions raised by the Nuclear Regulatory Commission (NRC) in their letter of April 25, 1980, to Dairyland Power Cooperative (DPC).

The answer to Question 1 was prepared by technical personnel from DPC, and the remaining answers were prepared by Dames & Moore (D&M). The various answers were provided to DPC and NRC in a draft form on different dates as per an accepted schedule prepared by DPC and D&M.

The contents of this report have further reinforced our earlier findings regarding the low seismic hazard associated with the LACBWR site and a lack of liquefaction potential from an SSE producing a maximum ground surface acceleration of 0.12 g at the site.

DAMES & MOORE

Dairyland Power Cooperative
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June 30, 1980

The scope of services for this report was developed through consultation with Mr. Richard E. Shimshak of DPC.

If you have any questions, please do not hesitate to call.

Very truly yours,

DAMES & MOORE

Harch Singh

Harch Singh, Ph.D.
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HS/MN:spf
Enclosures

I. BACKGROUND

The seismic hazard and the liquefaction potential at the LaCrosse Boiling Water Reactor (LACBWR) site are under continuous review under the Systematic Evaluation Program (SEP) initiated by the Nuclear Regulatory Commission (NRC) in 1978. Dames & Moore (D&M) has prepared several reports since 1973 addressing the issues of seismic hazard and liquefaction potential at the LACBWR site (references I-1 (Oct. 1973), I-2 (Mar. 1979), I-3 (Sept. 1979), I-4 (Nov. 1979), and I-5 (Mar. 1980)).

NRC, after reviewing the last of the D&M reports (reference I-5), posed some review questions (reference I-6). The answers were provided to NRC in draft form between May 16 and June 13, 1980, as per a previously accepted schedule.

This report is a final compilation of all the answers, prepared for the sake of documentation. The numbering system followed in this report corresponds to the system followed in the NRC review questions. A list of the show cause order review questions from NRC is given below.

1. The Response focuses on the containment building. The turbine building is also important and may be more vulnerable. All structures and components critical to safe shutdown need to be identified and evaluated to conclude that mitigative measures are unnecessary. If some structures are excluded, due to alternate safe shutdown capability, these structures and the alternate safe shutdown capability should be identified.
2. The Response states that the density and earth pressure coefficient of the soils beneath and around the reactor foundation have been significantly affected by the driving of piles. An undocumented reference to a Dames & Moore project was raised to justify this statement.
   a. Provide data from the above referenced Dames & Moore project used to justify this statement.

b. Provide data from other case histories which reflect on these conditions. Provide, reference, and discuss any reports, if known, which do not support assumed increases in SPT blow counts and overconsolidation ratio.

c. Provide data that substantiates that in situ material behaves as if it were at an overconsolidation ratio of 4.

d. Provide site specific data to substantiate the N_1* data listed in Table 1 to the Response.

3. Provide any observations of heave or settlement during excavation and pile driving during site construction.

4. Provide a basis for the increase of 3 lb/ft^3 assumed for under the reactor vessel. Provide a basis for any increase in density under other structures supported by driven piles.

5. Provide a tabulation of all N_1 and depth values for each boring and plot the results on a figure with N_1 as the abscissa, and depth as the ordinate. Show the location of the FS=1.0 line.

6. Provide data to substantiate that the effect on an oiled rope is to increase (sic)* N by 20%.

7. The Response has characterized the SSE as corresponding to a "very low seismic risk." Accordingly, the Response states that "using the designated seismic parameters should lead to conservative conclusions." In view of the implications in your response that margin may exist relative to the existing specification of the SSE, investigate whether a lower SSE may be justified and if so, provide the basis for such a position.

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*NRC meant to use "decrease" instead of increase and the question was answered on the basis of the word "decrease."

References

I-1 Geotechnical Investigation of Geology, Seismology, and Liquefaction Potential, LaCrosse Boiling Water Reactor (LACBWR) near Genoa, Vernon County, Wisconsin (Dames & Moore, October 1973).

I-2 Review of Liquefaction Potential, LaCrosse Boiling Water Reactor (LACBWR) near Genoa, Vernon County, Wisconsin (Dames & Moore, March 20, 1979).

I-3 Liquefaction Potential at LaCrosse Boiling Water Reactor (LACBWR) Site near Genoa, Vernon County, Wisconsin (Dames & Moore, September 28, 1979).

I-4 Preliminary Report, Proposed Measure to Mitigate the Potential for Liquefaction at LACBWR Plant Site near Genoa, Wisconsin (Dames & Moore, November 29, 1979).

I-5 Response to NRC Concerns on Liquefaction Potential at LaCrosse Boiling Water Reactor (LACBWR) Site near Genoa Vernon County, Wisconsin (Dames & Moore, March 21, 1980).

I-6 Personal Communication with Dennis Ziemann of NRC (April 25, 1980).

II. ANSWERS TO REVIEW QUESTIONS

Question 1

Answer:

Reactor shutdown is accomplished using the control rod drives or the boron injection systems. The control rod drive system can accomplish its shutdown function without utilizing any components or power supply located outside the reactor containment building. The boron injection system requires the operation of certain valves, one of two high pressure core spray pumps. One of two emergency diesel generators or an offsite 69 KV power supply is necessary to operate the valves or power the pump. Additionally, some nuclear instrumentation would be required to indicate successful reactor shutdown. These instruments indicate in the control room and are powered by two sets of batteries, one of which would be required. These batteries are located in the electrical equipment room below the control room in the turbine building and in the 1B emergency diesel generator building. The shutdown on the reactor (including post shutdown instrumentation) requires components in the reactor containment building, either offsite power (with power supply cables passing through the turbine building) or an emergency diesel generator and battery bank (one battery bank in the turbine building and the other in the 1B emergency diesel generator building). Shutdown of the reactor and the post shutdown monitoring (neutron monitoring, reactor pressure and reactor water level) can be accomplished using fixed and dedicated portable equipment located in the reactor containment building. With the addition of an indicator/control panel in the 1B emergency diesel generator building, reactor shutdown could also be accomplished by using equipment found only in the reactor building and the 1B emergency diesel generator building. (NOTE: If containment can be entered, this panel is not needed, since local control and instrumentation can be used).

In addition to the achievement of a safe reactor shutdown, another consideration is decay heat removal. Heat removal can be accomplished by several methods, all of which involve equipment in the reactor containment building. Use of the main condenser/circulating water system requires equipment in the turbine building and crib house as well as an off-site power supply. Use of the decay heat removal system requires equipment in the turbine building and crib house. Use of the shutdown condenser requires use of equipment in the turbine building for eventual water makeup, instrumentation and valve operation (cycling).

In the event heat removal is not attainable through methods discussed above, the following two methods are available as ultimate back-ups. Use of the high pressure core spray for heat removal in a discharge and makeup mode requires either the 1A or 1B emergency diesel generator for power and instrumentation, the turbine building for post shutdown indication and eventual water makeup. Removal of water for pressure and temperature control can be accomplished by use of the decay heat blowdown system discharging to the main condenser in the turbine building or operation of the Manual Depressurization System within the containment building controlled from the control room or alternately from a future addition of an indicator/control panl in the 1B emergency diesel generator building or by safety valves, also located inside the reactor containment building.

Use of the alternate core spray system in a discharge and makeup mode or as a backup water supply for the shutdown condenser or overhead storage tank requires equipment in the turbine building and the crib house, as well as post shutdown instrumentation capability powered by either battery bank with indication in the turbine building or 1B emergency diesel generator building. Removal of water can be accomplished by the same methods used for the high pressure core spray system.

Emergency core cooling systems, which may be required to mitigate the effects of various break size LOCA's, are the high pressure core spray system, the alternate core spray system and the Manual Depressurization System. These systems and their locations have been discussed above as alternate decay heat removal systems.

The reactor containment building has been analyzed and will remain functional for a 0.12 G seismic event. The 1B emergency diesel generator building is a recent addition to the plant and was designed to survive a 0.12 G seismic event. The turbine building and the crib house are currently being analyzed as part of the NRC's Systematic Evaluation Program, and it is anticipated that they will survive a 0.12 G seismic event or can be structurally modified to survive.

If liquefaction must also be considered in addition to the 0.12 G seismic event, the turbine building and the crib house may not remain functionally intact. In order to provide alternate safe shutdown capability, a new and separate cooling water supply using a fixed or portable pump capable of taking water from the river and providing long-term makeup to the shutdown condenser and overhead storage tank would be required. A remote panel of shutdown instrumentation/controls located in the 1B emergency diesel generator building would also be required as a mitigative measure in addition to the alternate supply of water indicated previously for the shutdown condenser and overhead storage tank.

A final consideration for safe shutdown is containment integrity following a 0.12 G seismic event. Assuming all containment isolation devices have operated, the only remaining area of concern is whether the piping penetrations can survive relative motion between the containment building and the turbine building. Preliminary analysis shows that these penetrations can remain intact under relative displacements up to three inches. A displacement of three inches is not expected to occur with a 0.12 G earthquake unless liquefaction is also assumed. If the latter occurs, special structural modifications to the piping penetrations will be required to insure containment integrity.

As a mitigative measure, with a 0.12 G earthquake and liquefaction, structural bracing would have to be installed at containment to provide hinge points for plastic deformation of piping thereby reducing the displacement loads applied to the containment penetrations to acceptable values.

Question 2a

Answer:

As explained in the Response dated March 21, 1980 (reference I-5), a recent D&M project (reference 2a-1) documented a study in which the increases in SPT (Standard Penetration Test) blow counts due to the effects of pile driving and placement of fill were predicted and subsequently verified through a test boring program. A relation was assumed between confining pressure, relative density, and overconsolidation ratio as explained in the Response. Pile driving was assumed to increase the at-rest earth pressure coefficient and the relative density of the soil. Based on these assumptions and on SPT data from the site before original construction, predictions of increased N values were made. A field program was then carried out in which blow counts taken close to the building compared well with the predicted values, especially in clean sands.

Attached are plates from reference 2a-1, showing normalized SPT data close to the structure (figure 2a-1), mean and standard deviation values of normalized SPT data close to the structure (figure 2a-2), and comparison of average SPT results as predicted and measured (figure 2a-3).

Question 2b

Answer:

In addition to the D&M project discussed in answering Question 2a, various other projects have been documented in which driving of piles resulted in increases of SPT blow counts. Brief descriptions of several such projects and data summarizing the blow count increases are presented in answer to Question 2b.

2b-1 Treasure Island, California

Treasure Island was constructed in the San Francisco Bay by hydraulically placing sand fill over dense sand and soft-to-medium stiff silty clay (see figure 2b-1-1). The 30 feet of loose-to-medium dense fill was composed of clean sands in the fine-to-medium sand size range. To densify the soil to increase the safety factor against liquefaction under a proposed office building, sand compaction piles were installed. The installation procedure required driving a 14-inch diameter, hollow steel mandrel to the required depth, filling the mandrel with sand, applying 100 pound/square inch air pressure to the top of the sand column, and withdrawing the false-bottomed mandrel. Piles initially were placed on 6- and 7-foot centers, and ultimately on 3- and 4-foot centers to obtain the specified 75-percent relative density.

Comparisons of standard penetration tests and densities before and after densification by compaction piles at the different spacings are presented in figures 2b-1-2 through 2b-1-5. These plots demonstrate that compaction piles were an effective means of achieving the required densification of the sand fill at this site.

2b-2 Oil Tank Foundations in Kyushu, Japan

One of the many Japanese projects using compaction piles for densification of foundations soils is the 100,000-ton oil tank foundations constructed on reclaimed land in Kyushu. The upper 8 meters (26 feet) of the site consisted of loose, clean sand fill in the fine-to-medium sand size range. The densification effect of sand compaction piles was evaluated by installing them at spacings of 2.0, 2.5, and 3.0 meters in a triangular layout. For each pile a hollow mandrel was advanced by a vibratory hammer to the required depth, and sand was compressed by air pressure into the cavity as the mandrel was withdrawn. The completed compaction pile was generally 60-70 centimeters in diameter compared to the mandrel with a diameter of 45-50 centimeters.

Figure 2b-2-1 illustrates the soil profile and the increases in SPT blow counts after treatment by sand compaction piles at various spacings.

2b-3 Steel Mill in Tokyo Bay, Japan

Ground stabilization of an ore yard of a steel mill constructed on reclaimed land in Tokyo Bay involved installation of sand compaction piles. At this site the densification by the piles served to stabilize the underlying soft marine clay as well as the loose silty sands near the surface. Compaction pile diameters and the technique of using a vibratory hammer to drive the casings were similar to those at the Kyushu site, discussed in 2b-2. Spacings between pile centers varied between about 2 and 3.5 meters.

Figure 2b-2-1 shows the soil profile and comparison of SPT blow counts before and after densification by sand compaction piles.

2b-4 U.S. Army Corps of Engineers Pile-Driving Effects Test Program

A test program to evaluate the effects of pile driving was undertaken by the U.S. Army Corps of Engineers at Lock and Dam No. 26 on the Mississippi River at Alton, Illinois. Timber piles were driven into the alluvial and outwash deposits at the site. SPT blow counts and static cone penetration resistance were recorded in the vicinity of two pile groups before and after pile driving. The plan of the pile groups and boring locations is shown in figure 2b-4-1. Figures 2b-4-2 and 2b-4-3 illustrate the increases in penetration resistance resulting from driving of the piles.

2b-5 Silo at Kobe, Japan

A silo constructed on fill at the seashore of Kobe was to be founded on pedestal (compaction) piles of 51-centimeter diameter on centers of 1.46 meters. However, driving of the piles became progressively more difficult as the driven casings displaced soil and increased densities and blow counts. Original SPT data and soil profile of the site appears in figure 2b-5-1, and the pile-driving sequence is shown in figure 2b-5-2.

Based upon the known local correlation between void ratio e and blow count N, the decreased void ratios resulting from soil displacement by the piles were calculated, and new N values were predicted as shown in table 2b-5-1. A boring between rows 4 and 5 in figure 2b-5-1, where driving had become impossible, yielded the blow counts indicated in figure 2b-5-3, with similar results in further borings. These results clearly substantiated the predicted increases from original blow counts in the sand gravel fill.

2-b-6 Bailly Generating Station

A test program was carried out in 1978 at the site of the proposed Bailly Nuclear Generating Station in Indiana, to evaluate the densification effect of the driven piles. H-piles, 90 feet long, were driven close together in two rows separated by about 5 feet, with SPT borings drilled between the two rows before and after pile driving. Plots of the blow counts before and after driving appear in figure 2b-6-1, with substantial improvement occurring in the sand soils as a result of the pile driving. (It should be noted that the H-piles are not displacement piles, and if displacement piles were driven the densification effect would be more pronounced.)

Concluding Remarks for Question 2b

The selected case histories cited above are a representative few which serve to demonstrate increases in density and penetration resistance resulting from driving of piles in loose sandy soils. Although it is likely to be a contributing factor, the densifying effect of the vibration during driving is less well-defined than that of simple displacement of the soil by the driven piles, and it was solely this latter factor that was considered in the estimates of densification described in reference I-5 and in the discussion of Review Question 4.

We are not familiar with any projects in which driving of piles into loose sands resulted in further loosening and decreasing the density of the soils. Dense and very dense sands, however, tend to loosen when displacement piles are driven into them. Under such circumstances there would invariably be visual evidence of surficial heave.

Question 2c

Answer:

2c Assumption of Increased OCR

There are many variables involved in the densification of a loose sand deposit by pile driving that make it difficult to quantitatively predict the magnitudes of lateral stresses generated by the pile driving. However, among foundation engineers, it is an accepted premise that with empirical substantiation, as discussed for Question 2b, the soil displacement and vibration attendant to pile driving generally result in substantial increases in lateral stresses in the immediate vicinity of the driven piles. These increases in lateral stresses are expressed as an increased coefficient of earth pressure K_o.

Overconsolidation in a sand deposit generally is produced by removal of an overburden pressure or by fluctuation in groundwater level. These decreases in vertical stresses result in an increase in the earth pressure coefficient. Conversely, if lateral stresses are increased by an event such as pile driving, the earth pressure coefficient is increased and, by correlation, a condition of effective overconsolidation is induced. Several investigators (see references) have proposed quantitative correlations between K_o and OCR; an OCR of 4 corresponding to a K_o of 1.0 is conservative, as described in reference I-5. The increase in K_o from 0.4 or 0.5 in a normally consolidated deposit to about 1.0 near driven piles is an assumption which appears to be in keeping with assumptions of the various investigators and available empirical data (e.g., reference 2a-1). Therefore, in the context of the LACBWR site, the condition of overconsolidation should be viewed as a phenomenon which causes the soil to behave as having higher lateral stresses than under the normal depositional conditions, rather than having been caused by removal of previously applied vertical stresses. The different sources of the overconsolidation condition are not distinguishable in terms of the effect of increased penetration resistance.

Question 2d

Answer:

2d-1 General

The N_1* data presented in table 1 of the D&M Response dated March 21, 1980 (reference I-5), represent corrected blow counts to account for the following: 1) the increased density (therefore, increased relative density) due to pile driving, and 2) the effect of increased horizontal stresses due to pile driving. (The effect of increased horizontal stresses normally is expressed in terms of either an increased coefficient of lateral earth pressure or a behavior similar to that of an "overconsolidated" sediment.) In the D&M Response of March 21, a procedure based on data in published literature was used to quantify the increase in measured

SPT-N values due to pile driving. A case history where such a procedure was used to predict the increased N values and how such a prediction was verified by actual field measurements also was described in the Response of March 21.

Table 1 of reference I-5 focuses on SPT-N values between the depths of 30 and 45 feet below the plant grade (elevation +639 feet) to represent conditions under the containment which rests on some 230 piles. To substantiate the N_1* data presented in table 1, new SPT-N values under the containment would be required. SPT-N values would have to be obtained by drilling through the containment between the existing piles between the depths of 30 to 45 feet below plant grade. Obviously, such a drilling program is not feasible. Therefore, any substantiation of the N_1* data presented by D&M would have to be accomplished by some indirect means which can be considered reasonably applicable to the conditions of the LACBWR plant site.

One practical way of achieving the above objective is to simulate conditions existing under the containment and measure SPT-N values. However, this too is not simple and may not be economically feasible, because a true duplication of existing conditions cannot be achieved unless the entire stress history is reproduced in the same sequence. This might be done, for example, by excavating some 30 feet, driving a pile cluster (using the same type of piles, pile dimensions, spacings, pile-driving equipment, and driving procedures), loading the piles with loads similar to those from the containment, and then obtaining SPT-N values between piles. Even under these reproduced conditions, duplicating the effect of time and various other factors such as water table fluctuations which might have contributed to the present condition would be impossible.

Another simpler approach would be to study case histories which present data on site conditions, which are similar to those at the LACBWR plant site, and SPT data before and after driving of piles. Such an exercise was performed, and data from six case histories, in addition to details on the D&M project at South San Francisco Medical Center site, were provided (see answers to questions 2a and 2b). If an indirect substantiation of the N_1* data is acceptable to NRC, such a substantiation already has been made in the case histories provided.

2d-2 Available Alternatives for SPT's under the Existing Structures

All the potential areas where access to a drill rig could be provided for substantiation of N values under existing structures were examined and five locations were identified (see figure 2d-2-1). Location 1 is a relatively open area within the turbine building with sufficient head room and easy access for a drill rig. (Even a truck mounted rig could be used at this location.) A 4-foot thick concrete floor would have to be penetrated before SPT's could be performed at Location 1.

Location 2 is a relatively small free area in the turbine building with difficult access conditions and some restrictions to working. A very light skid rig with small overall dimensions (such as the Acker-Ace with approximate dimensions of 3½ feet x 6 feet and approximate weight of a ton, with a motor and a 15-foot boom that can be easily separated from the rig) can be hoisted from Location 1 and moved parallel to the roof and brought down through the hatch at Location 2. Certain light equipment in the area will require shifting during the drilling period. As in Location 1, penetrating through the concrete floor is necessary before SPT's can be performed.

Location 3 in the turbine building is the most difficult drilling location with restrictions on head room (about 14 feet), lack of flexibility in the drill hole location, and the need to go through a high-radiation area. The drill hole location is inflexible because of tightly spaced equipment in the tunnel through which the hole has to advance. An 8-inch diameter hole in the floor of the machine shop (Location 3) provides access in between the various pieces of equipment in the tunnel below. However, this location is too close to a pile cluster and there is a possibility of hitting one of the piles. (No details of the pile cap were available for this location, and if a pile cap does exist, then the possibility of hitting the pile cap is very high.) A few alternatives for gaining access to the drill rig are available. One of the alternatives is to follow the same procedure as for Location 2 and move the disassembled skid rig through a wide door into the machine shop and assemble it again. It may be possible to bring the disassembled drill rig through a couple of wide doors directly into the machine shop from the grade level. If the boom height poses a problem, a modification may be required--a pulley and rope system could be hung from the I-beam and could be used for the SPT's.

The drilling complexities increase as we go from Location 1 to Location 3. Work at Location 3 is likely to obstruct normal operations of the plant and will involve a greater number of complications.

The first two locations identified above are likely to provide direct information on the existing conditions under the turbine building under free-field conditions, since these locations are free from piles. The last location is in the turbine building near the containment and is likely to be influenced by the presence of piles. (The machine shop is to be considered part of the turbine building.)

Two other locations, Locations 4 and 5, were identified outside the buildings. Location 4 is on the pile cap of the stack where easy access for a small drill rig will be available. After about 4 feet of reinforced concrete pile cap has been cored through, SPT's can be performed within the pile cluster under the stack. The piles under the stack are at slightly greater spacings than the piles under the containment. Data from SPT's performed here may provide information which may indirectly subtantiate the assumed conditions within pile groups at similar depths.

Location 5 is a free area outside all structures where a small-scale pile-driving program may be undertaken. With SPT values obtained before and after the pile driving, a demonstration could be made of how the initial N values may have increased due to densification and increased horizontal stresses. Also, settlement or heave observations could be made during pile driving. One possible arrangement of the demonstration pile cluster could consist of 9 piles at 3½-foot centers driven to a depth of 45 feet. Tapered wood piles with dimensions similar to those under the containment could be used. SPT's could be performed in two locations within the pile cluster before operations and two other locations within the pile cluster after the driving operations. Vibrations during pile driving are likely to pose some problems to normal operations of the plant and therefore may impose a temporary shutdown. The exact location of the pile cluster should be decided after checking for any underground pipes or other obstructions.

It is necessary to keep in mind the following information before a drilling program is finalized and implemented:

1) The general plant grade is +639 feet.

2) The bottom of the containment is +610 feet.

3) The containment rests on 232 tapered piles.

4) The average tip elevation of the piles in (3) is +580 feet.

5) The turbine building rests on a 4-foot thick structural floor and several configurations of pile caps (pile caps are roughly 3 feet in thickness).

6) There are several different configurations of pile clusters under the turbine building with a total of 310 cast-in-place concrete piles of the step-taper design of Raymond.

7) The average tip elevation of piles in (6) is +569.5 feet.

8) The concern for liquefaction potential expressed by NRC is between the water table elevation (+629 feet average) and elevation +599 feet.

9) The piles under the containment are founded roughly 20 feet into dense sand, considered non-liquefiable under the design SSE.(*)

10) The piles under the turbine building are founded roughly 30 feet into dense sand, considered non-liquefiable under the design SSE.(*)

2d-3 Schedule

A minimum of 1 week and a maximum of 2 weeks of field operations will probably be required to perform any or all of the tests discussed above. One week's notice for field mobilization will be required and a 2 to 4 week period is estimated to be required for documenting and analyzing the data.

2d-4 Summary

Among the several review questions posed by NRC, question 2d was considered to be the key to the resolution of existing technical differences of opinion on the liquefaction issue at the LACBWR plant site. Therefore, a final decision regarding the level of effort required to perform satisfactory field work to obtain site-specific data will be made after the NRC review of all answers prepared by DPC/D&M and technical discussions among NRC staff and DPC/D&M.

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*Soils below elevation +599 feet are considered non-liquefiable by all, including the NRC and their consultant WES.

References

2a-1 Liquefaction Potential Study, South San Francisco Medical Center, South San Francisco, California, for Kaiser Foundation Hospitals (Dames & Moore, August 11, 1978).

2b-1-1 Basore, C. E., and J. D. Boitano, "Sand Densification by Piles and Vibroflotation," Placement and Improvement of Soils to Support Structures, ASCE Specialty Conference Proceedings (August 1968).

2b-2-1 Nakayama, J., E. Ichimoto, H. Kamada, S. Taguchi, "On Stabilization Characteristics of Sand Compaction Piles," Soils and Foundations, Vol. 13, No. 3 (September 1973).

2b-4-1 Woodward-Clyde Consultants, Results and Interpretation of Pile-Driving Effects Test Program, Existing Lock and Dam No. 26, Mississippi River, Alton, Illinois," Report to U.S. Army Corps of Engineers.

2b-5-1 Endo, M., "Relation Between Design and Construction in Soil Engineering--Deep Foundations," Caissons and Pile Systems, Proceedings of Specialty Session No. 3 of 9th International Conference of Soil Mechanics and Foundation Engineering (1977).

2b-6-1 Dames & Moore Report No. 05676-008-07 for Sargent & Lundy Engineers (1978).

2c-1 Ishihara, K., and J. Takatsu, "Effects of Overconsolidation and K_o Conditions on the Liquefaction Characteristics of Sands," Soils and Foundations, Vol. 19, No. 4 (December 1979).

2c-2 Schmertmann, J. S., "Measurement of In Situ Shear Strength," Proceedings of the Conference on In-Situ Measurements of Soil Properties, Specialty Conference of the Geotechnical Engineering Division, ASCE, Vol. II (1975).

2c-3 Seed, H. B., and W. H. Peacock, "Test Procedures for Measuring Soil Liquefaction Characteristics," Journal of the Soil Mechanics and Foundations Division, ASCE, Vol. 97, No. SM8 (August 1971).

2c-4 Sherif, M. A., I. Ishibashi, and D. E. Ryden, Coefficient of Lateral Earth Pressure at Rest, Soil Engineering Report No. 9, University of Washington (1974).

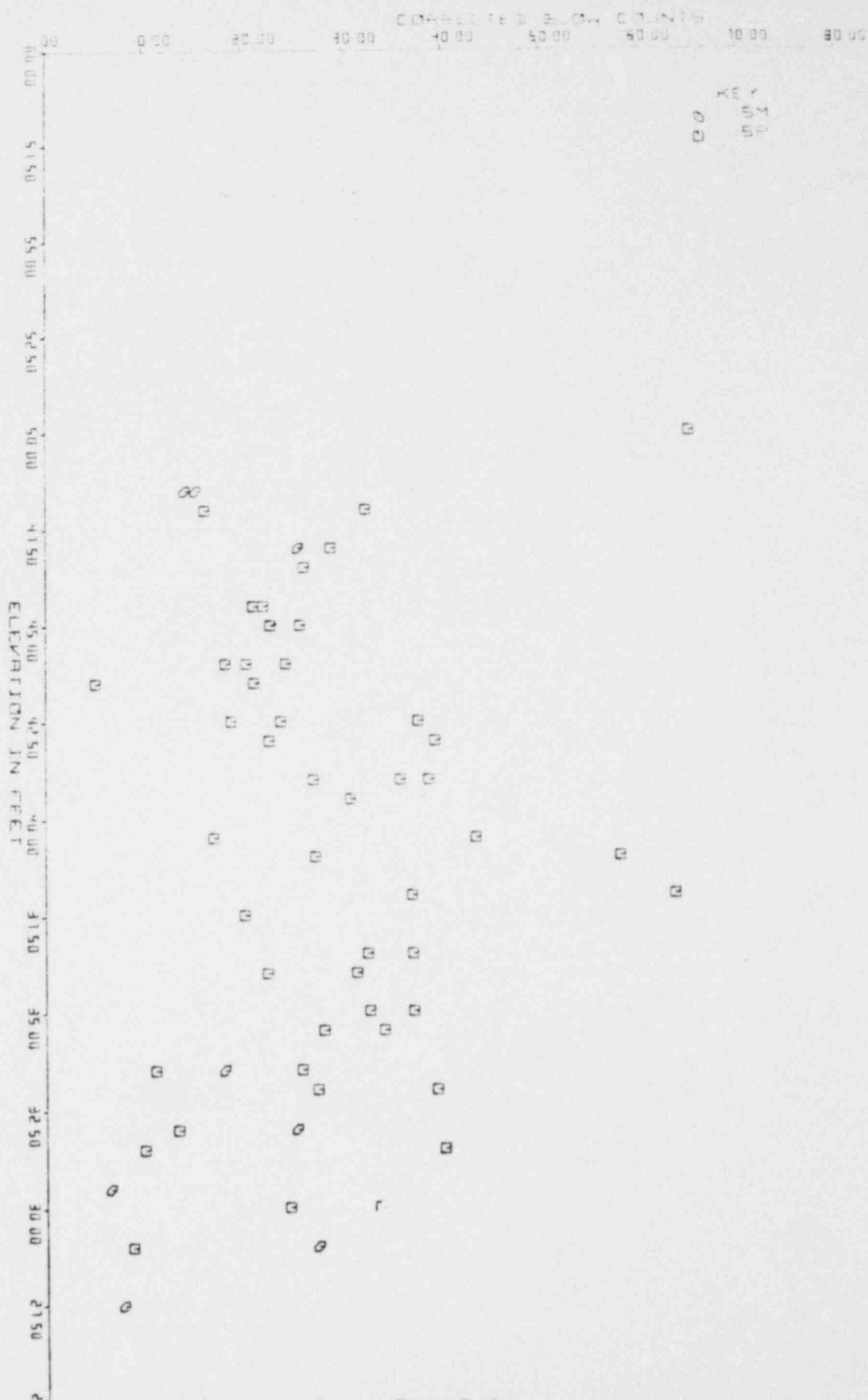

Figure 2a-1
Standard Penetration Resistance Data Close to Existing Hospital, Normalized to 1 ton/ft^2 Overburden

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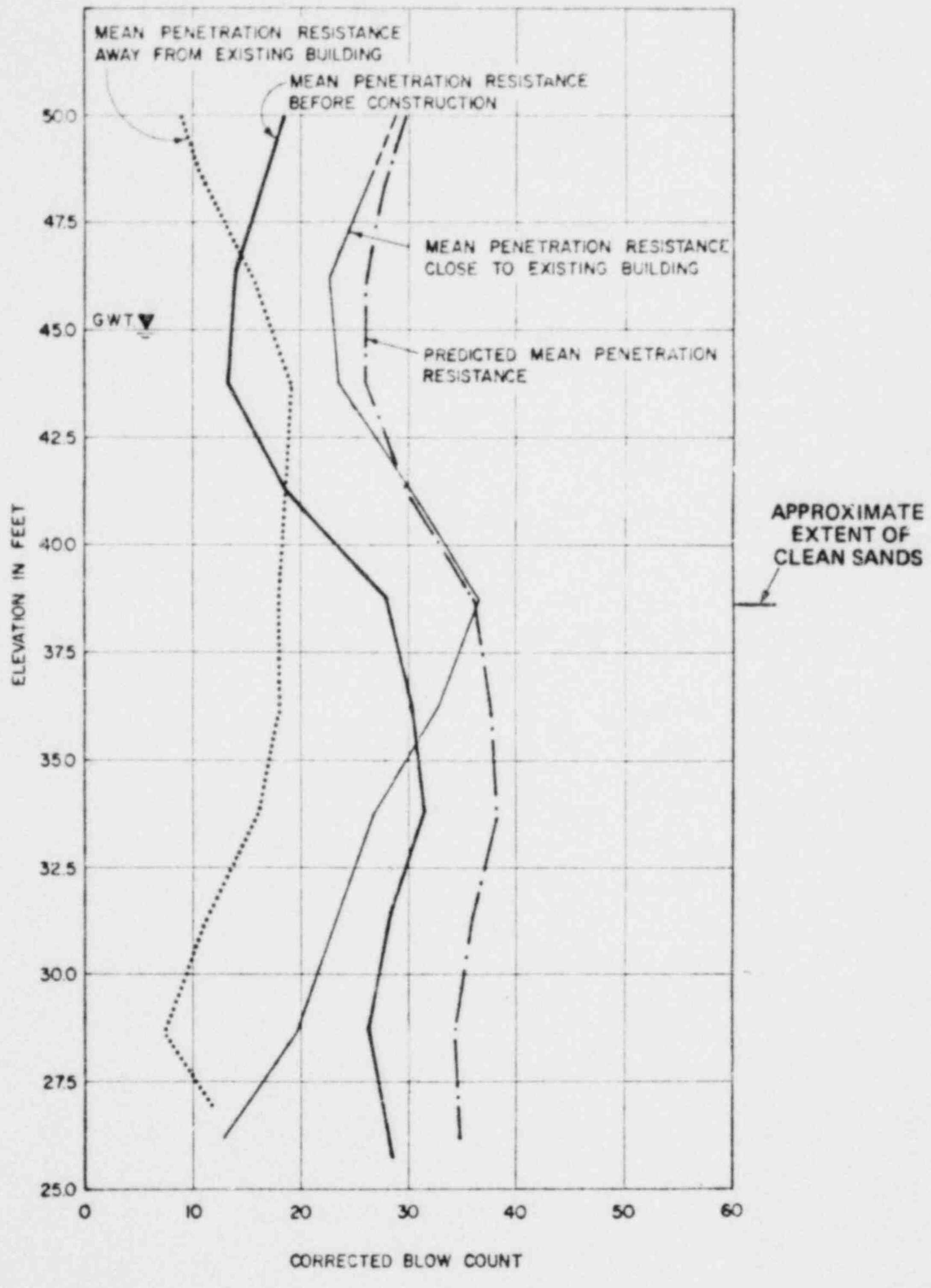

Figure 2a-3
Comparison of Average Standard Penetration Resistance Results

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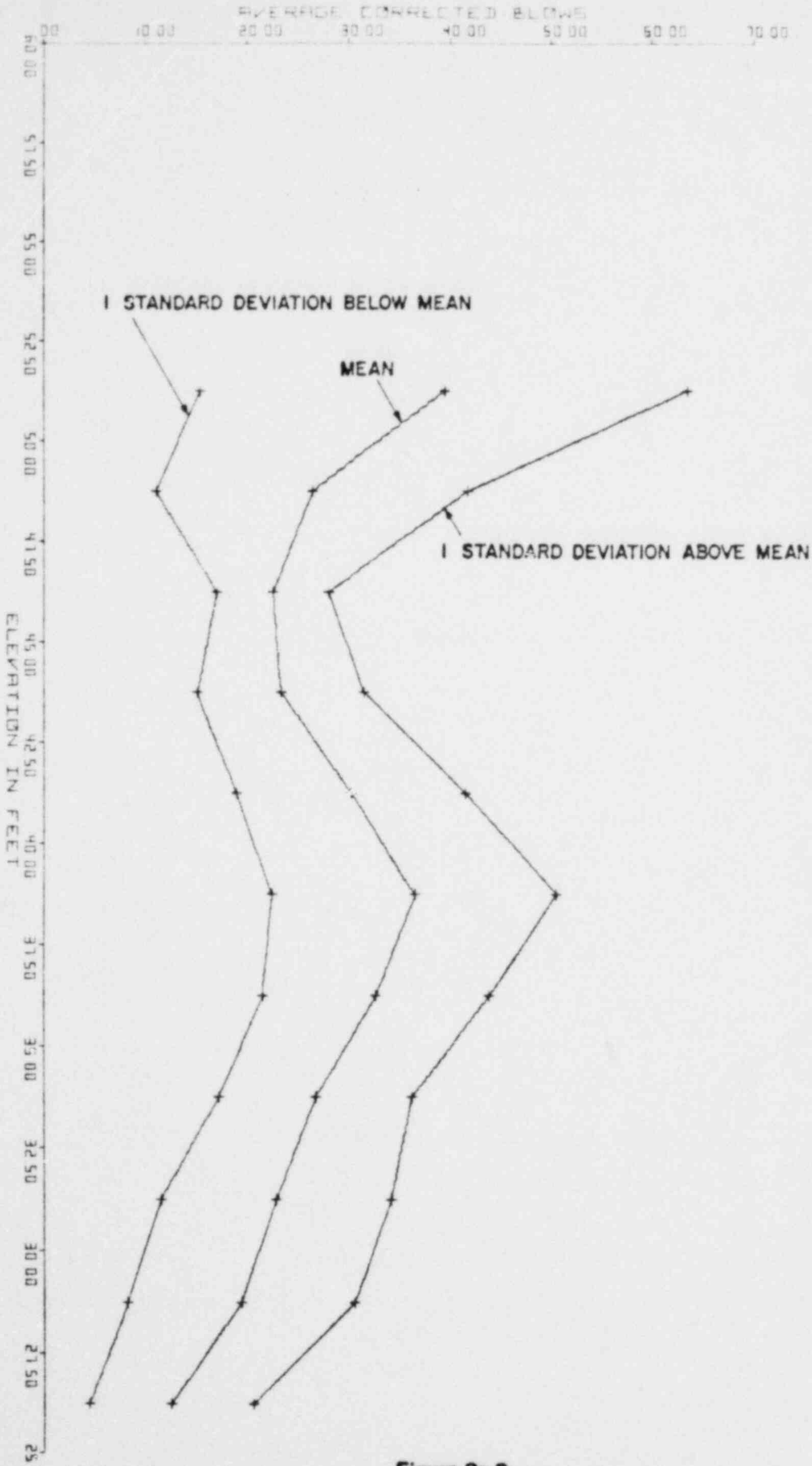

Figure 2a-2
Standard Penetration Resistance
(mean and standard deviation values of normalized blow counts shown on plate 2a-1, data close to existing building)

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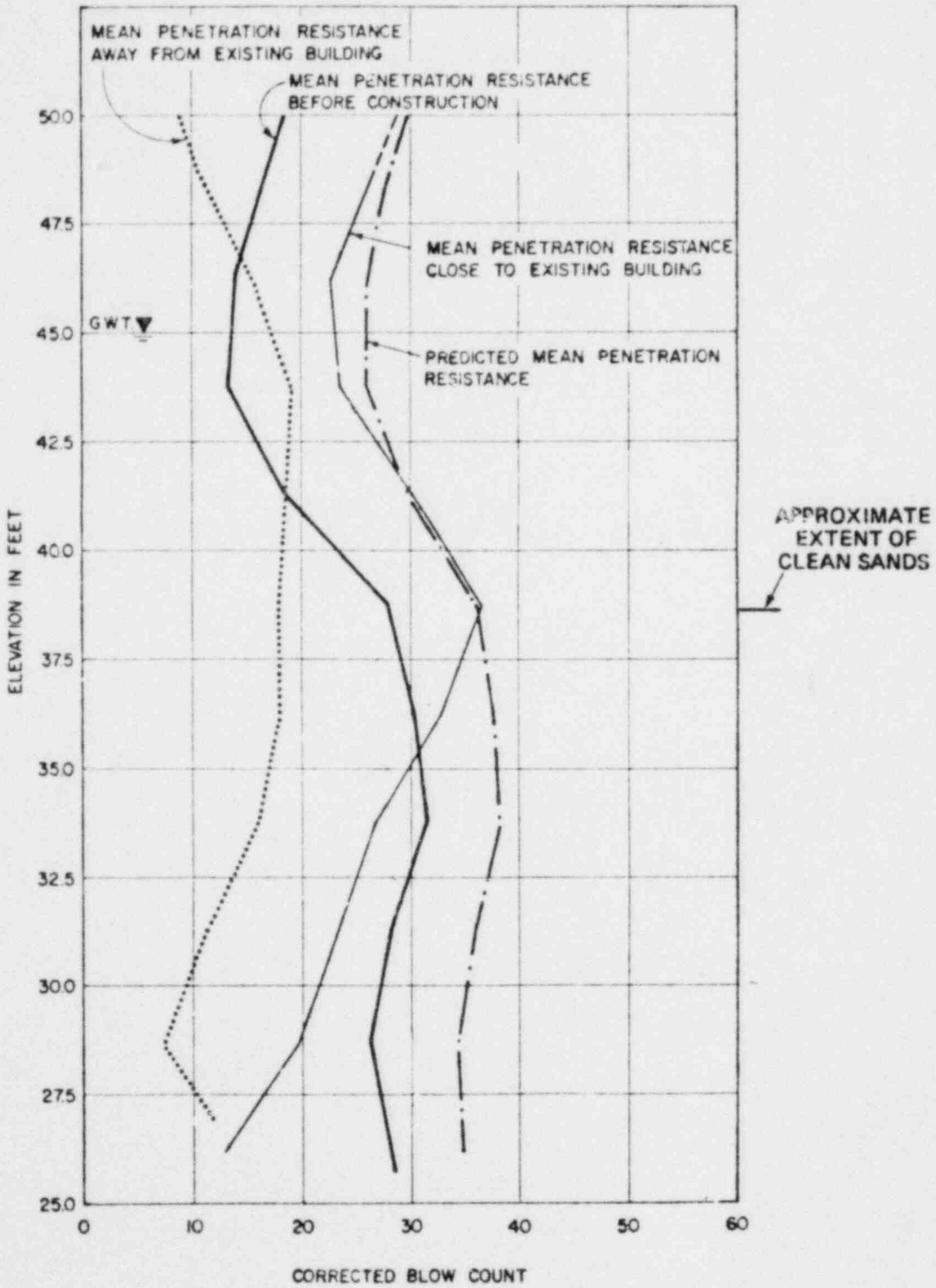

Figure 2a-3
Comparison of Average Standard Penetration Resistance Results

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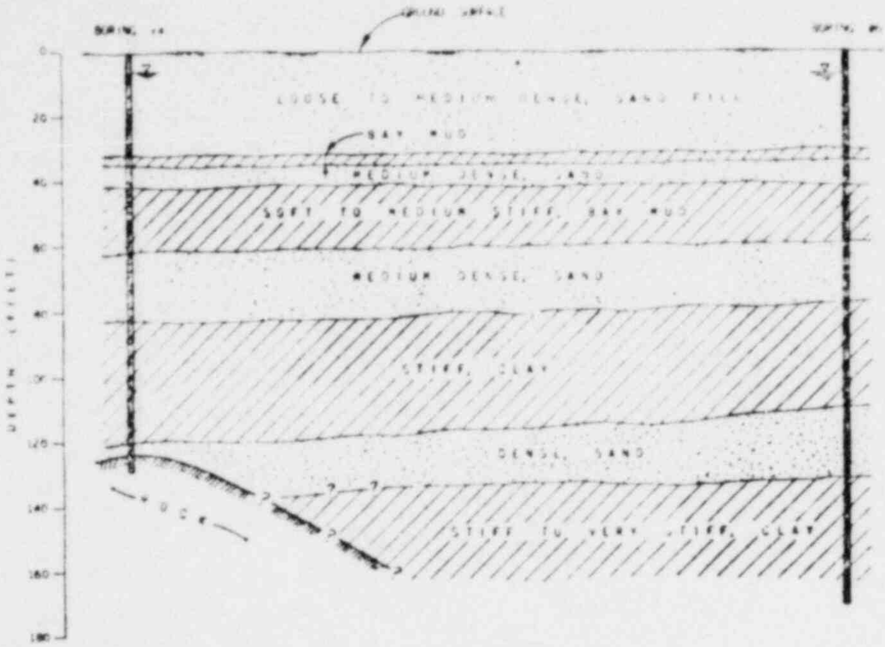


Figure 2b-1-1
Generalized Soil Profile Across Building Site

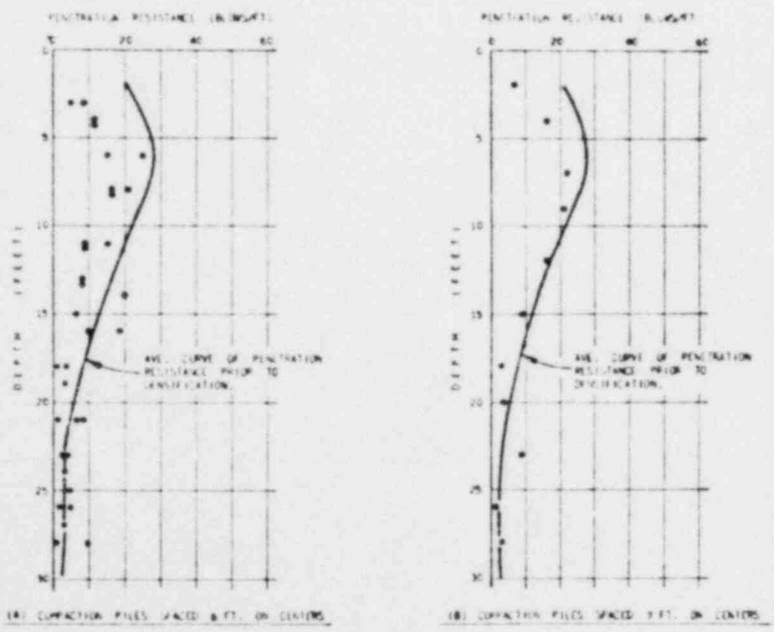


Figure 2b-1-2
Comparison of Standard Penetration Tests Before and After Densification by Compaction Piles

DAMES & MOORE

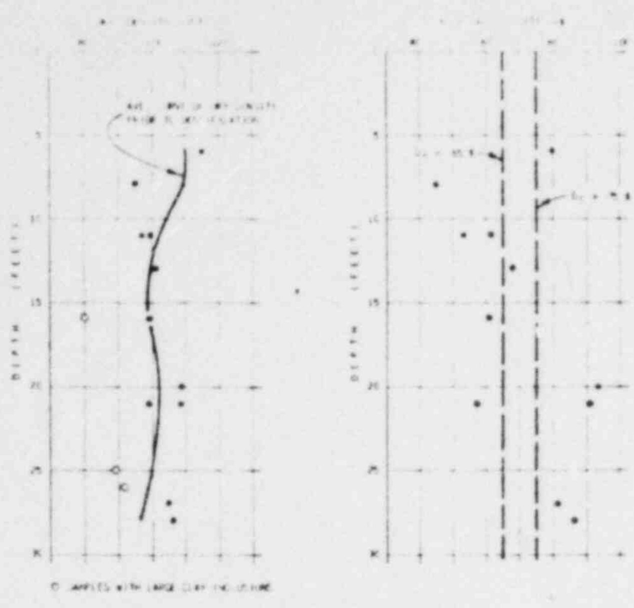


Figure 2b-1-3
Dry and Relative Sand Fill Density After Compaction Piles Spaced 6 Feet on Centers

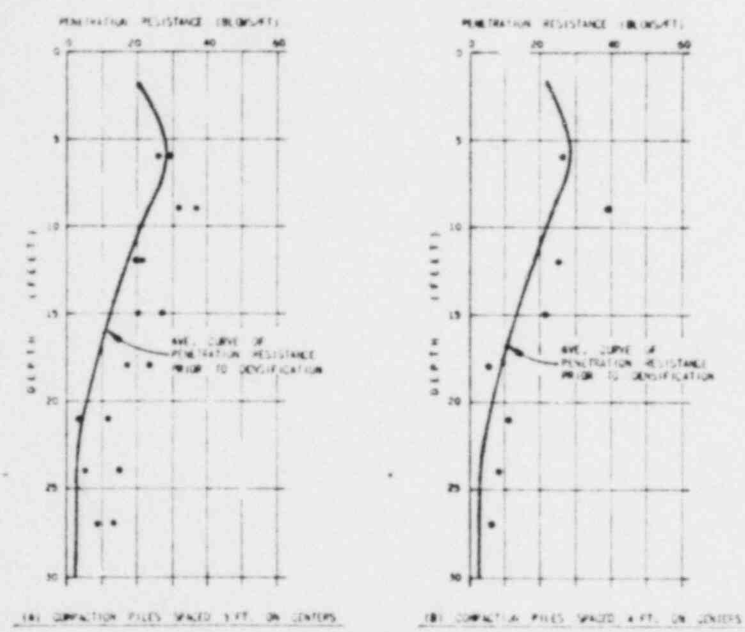


Figure 2b-1-4
Comparison of Standard Penetration Tests Before and After Densification by Compaction Piles

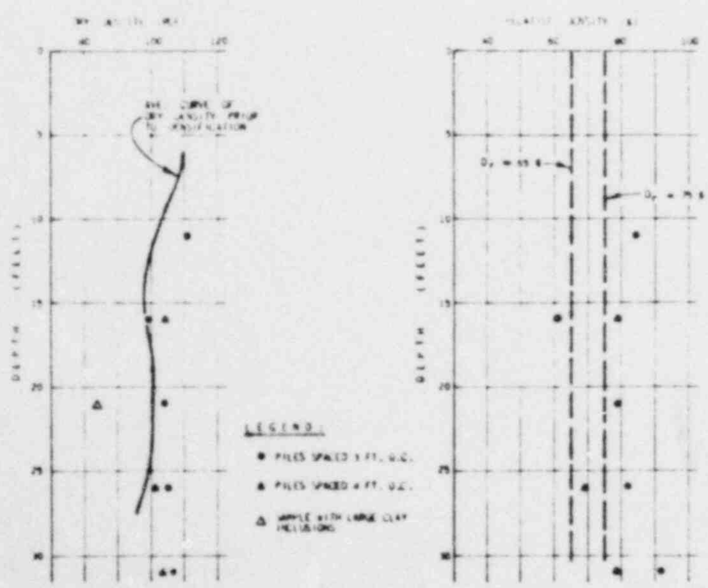


Figure 2b-1-5
Dry and Relative Sand Fill Density After Compaction Piles Spaced 3 Feet and 4 Feet on Centers

DAMES & MOORE

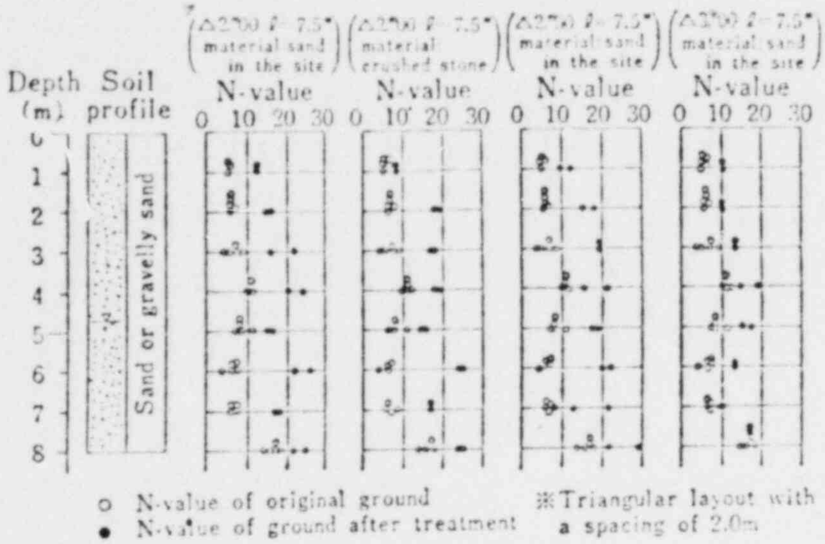


Figure 2b-2-1
Soil Profile and Comparison of N-Value Before and After Treatment

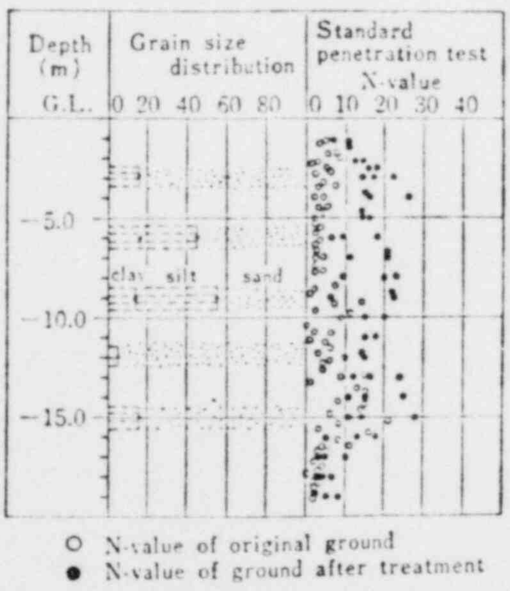


Figure 2b-3-1
Soil Profile and Comparison of N-Value Before and After Treatment

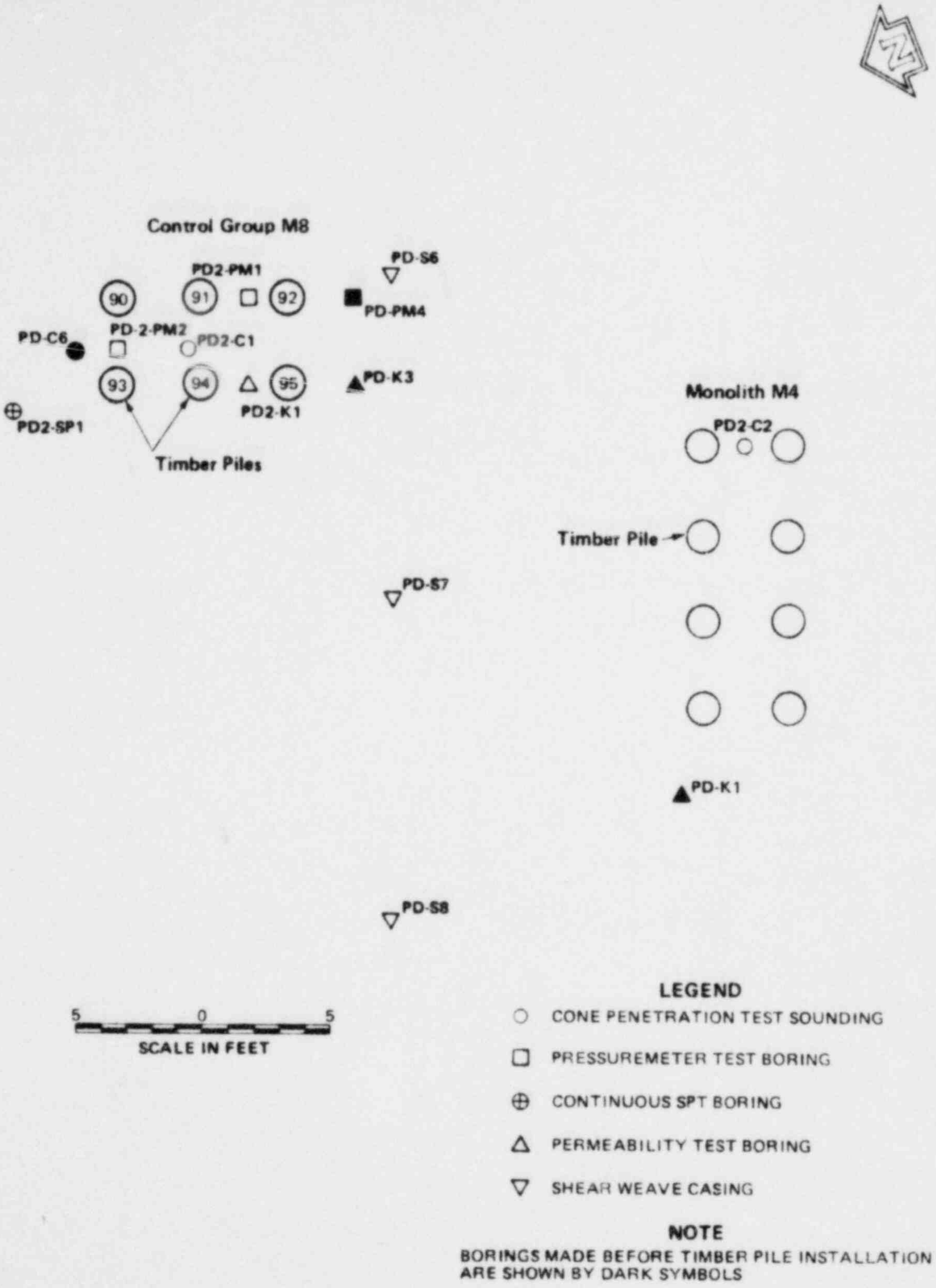


Figure 2b-4-1
Boring Location Plan After Timber Pile Installation
Pile Driving Effects Test Program by
Woodward-Clyde Consultants for St. Louis District, Corps of Engineers

DAMES & MOORE

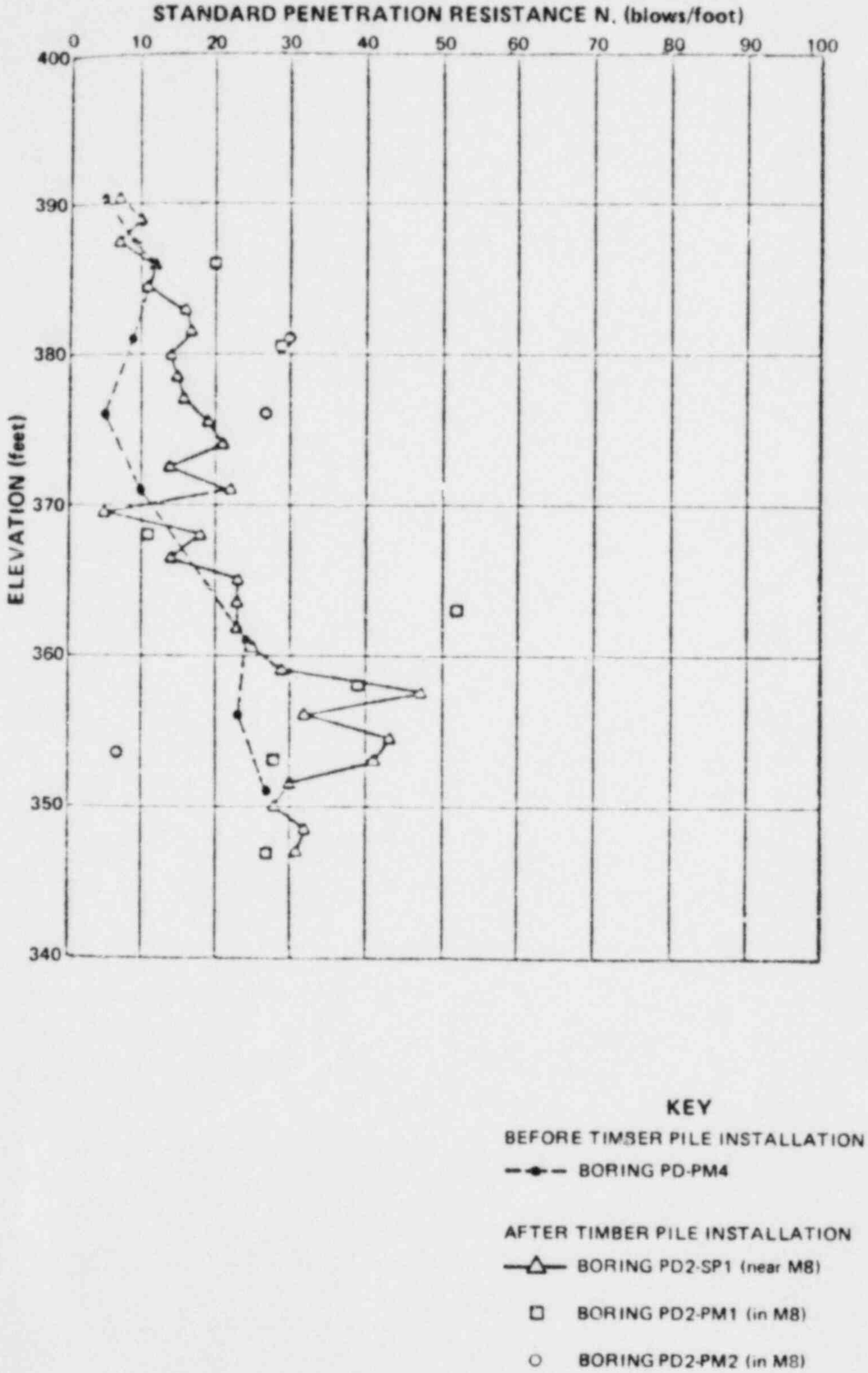
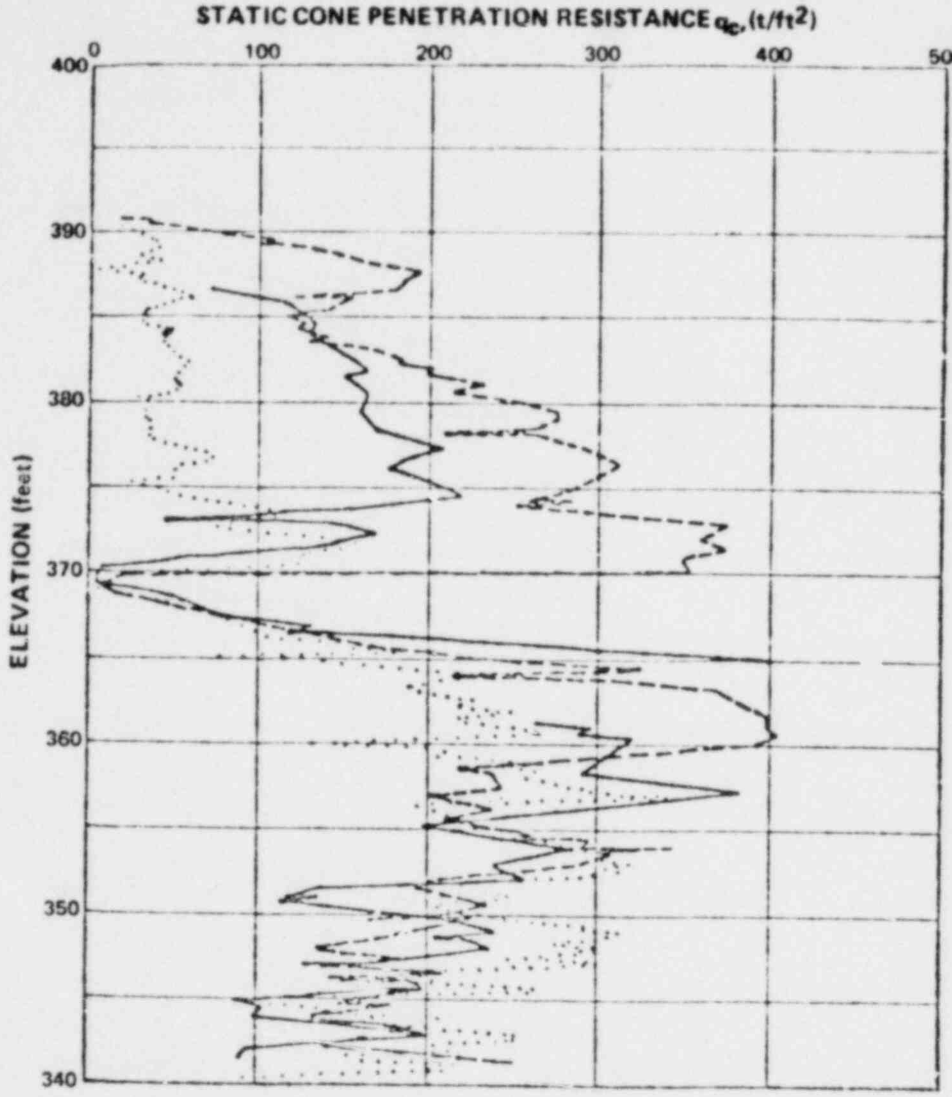


Figure 2b-4-2
Standard Penetration Resistance Profiles After Timber Pile Installation
Pile Driving Effects Test Program by
Woodward-Clyde Consultants for St. Louis District, Corps of Engineers

DAMES & MOORE



KEY

BEFORE TIMBER PILE INSTALLATION

·········· BORING PD-C6

AFTER TIMBER PILE INSTALLATION

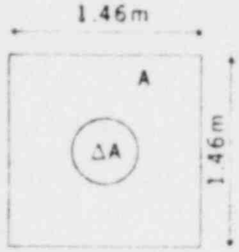
– – – – BORING PD2-C1

——— BORING PD2-C2

Figure 2b-4-3
Static Cone Penetration Resistance Profiles After Timber Pile Installation
Pile Driving Effects Test Program by
Woodward-Clyde Consultants for St. Louis District, Corps of Engineers

DAMES & MOORE

Table 2b-5-1
Variation in N Value Before and After Pile Driving Considered From Variation in Void Ratio



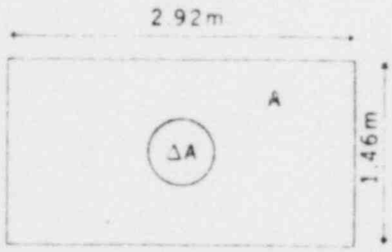
$A = 2.1316\ (m^2)$

$\Delta A = 0.2043\ (m^2)$

$\frac{\Delta A}{A} = 0.0958$

$\Delta e = 0.0958(1 + e_0)$

before pile driving	N value	30	20	15	10	5	4
	e_0	0.828	0.861	0.895	0.942	1.045	1.100
after pile driving	Δe	0.175	0.178	0.182	0.186	0.196	0.201
	e_1	0.653	0.683	0.713	0.756	0.849	0.899
	N value	N >50	N >50	N >50	N >50	23	15



$A = 4.2632\ (m^2)$

$\Delta A = 0.2043\ (m^2)$

$\frac{\Delta A}{A} = 0.0479$

$\Delta e = 0.0479(1 + e_0)$

before pile driving	N value	30	20	15	10	5	4
	e_0	0.828	0.861	0.895	[illegible]	1.045	1.100
after pile driving	Δe	0.088	0.089	0.091	0.093	0.098	0.101
	e_1	0.740	0.772	0.804	0.849	0.947	0.999
	N value	N >50	N>50	41	23	10	7

DAMES & MOORE

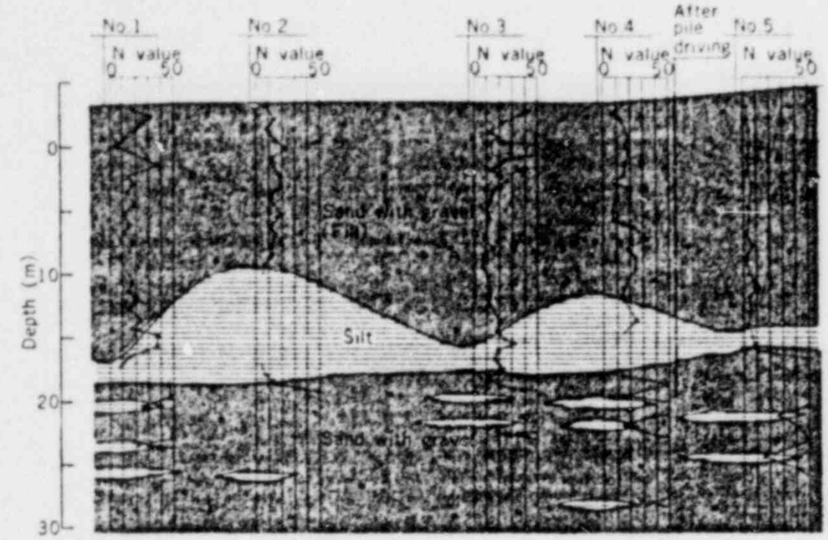


Figure 2b-5-1
Soil Strata and Properties of Construction Site of T Silo, Kobe
(before start of construction)

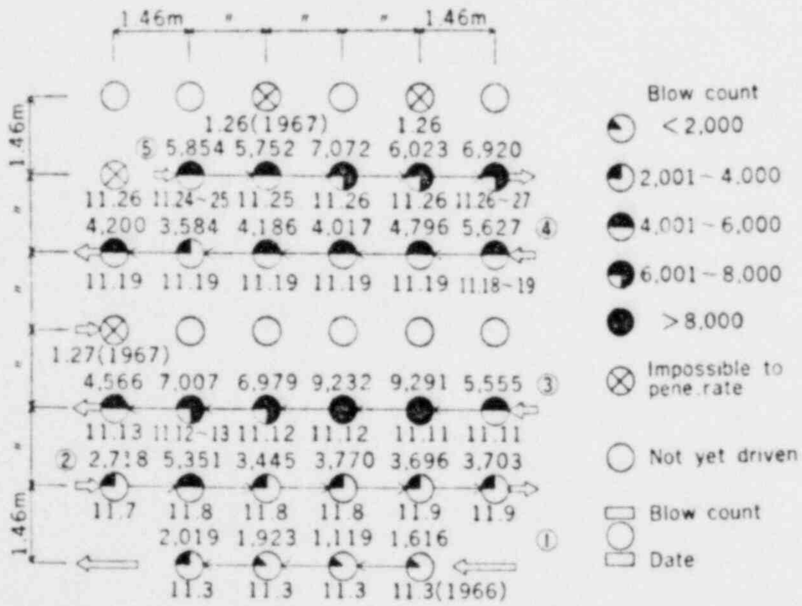


Figure 2b-5-2
Driving of Pedestal Piles (T Silo, Kobe)

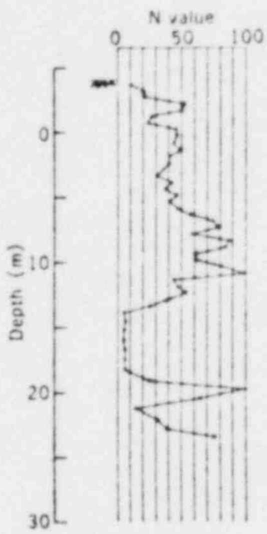


Figure 2B-5.3
Properties of Construction Site Soil Strata at T Silo, Kobe (after pile driving)

DAMES & MOORE

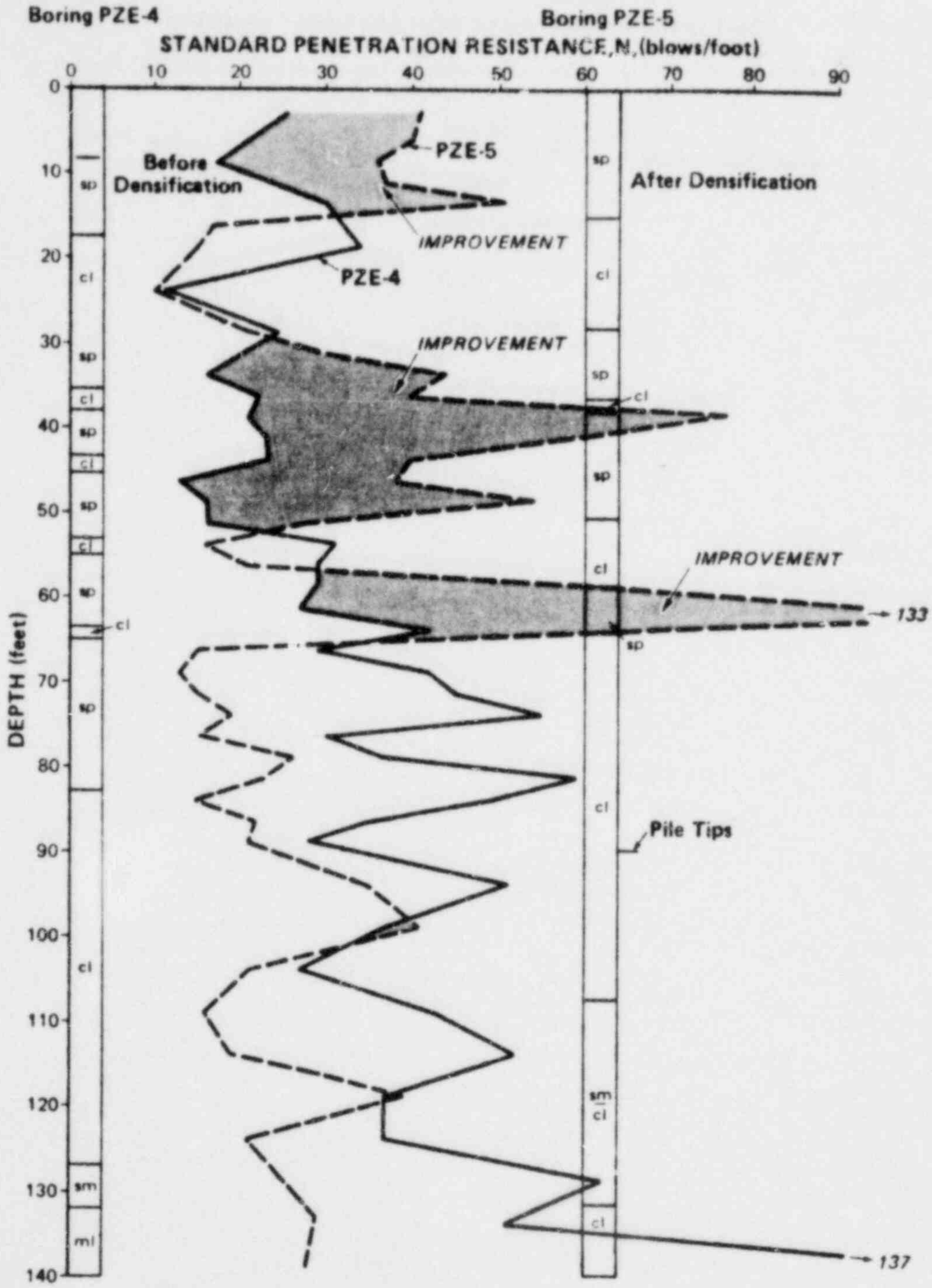
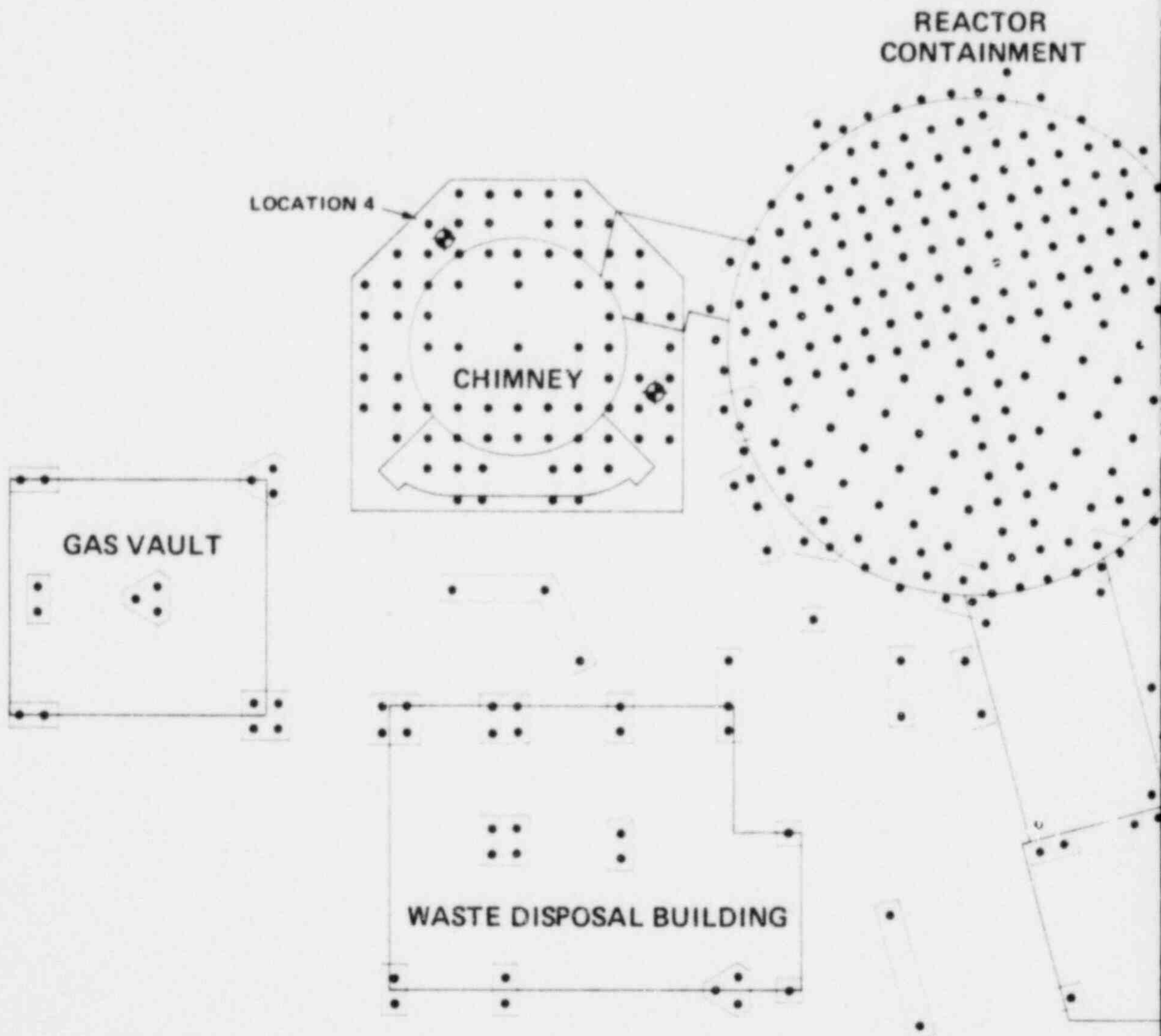


Figure 2b-6-1
Effect of Pile Driving on Standard Penetration Resistance
Bailly Generating Station

DAMES & MOORE



DRAWING REFERENCES:

Sargent & Lundy Drawings

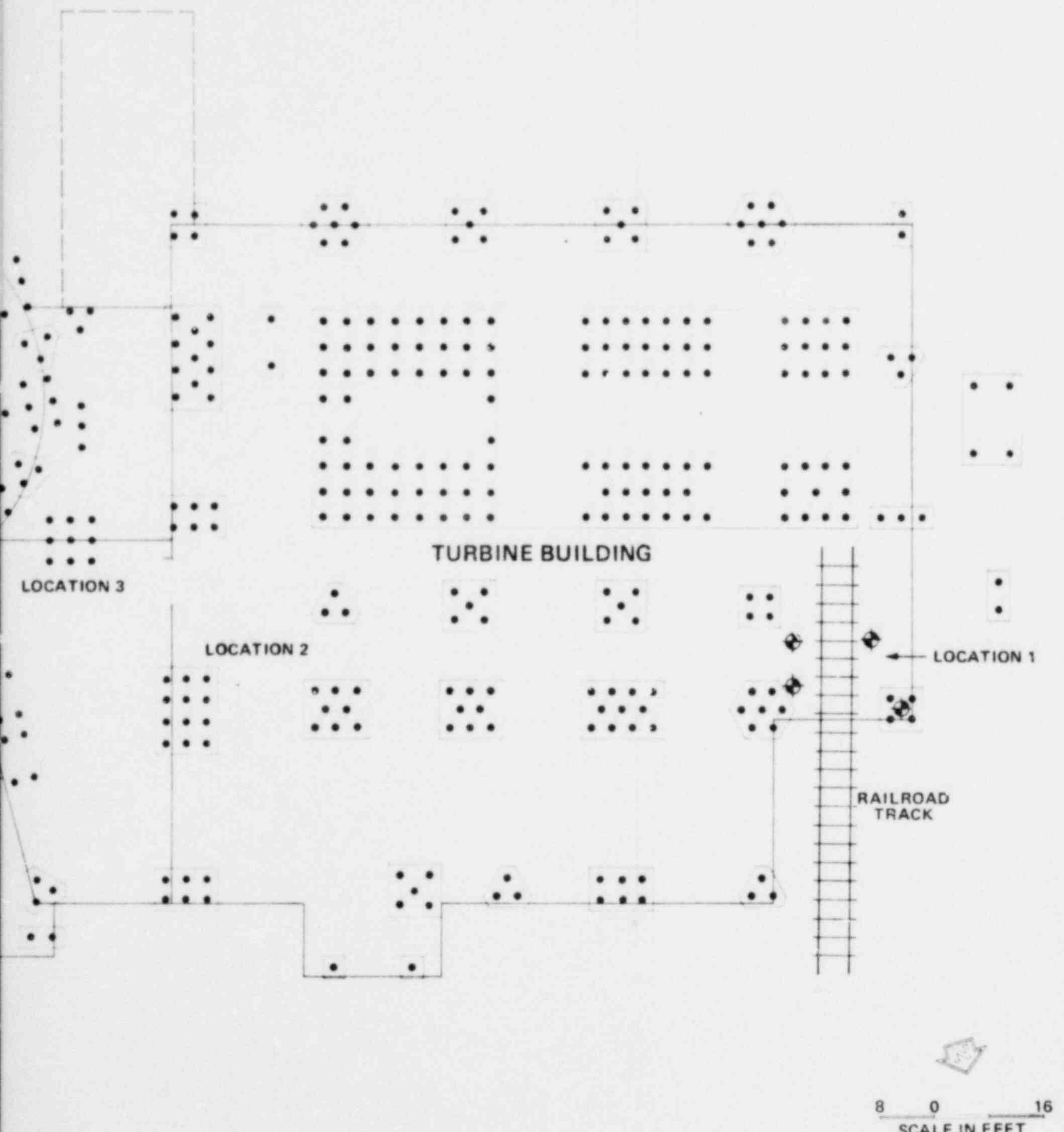
B-3, Rev. F: Piling Plan, LACBWR Generator Plant, Dairyland Power Cooperative, Genoa, Wi. (3/28/63)

B-4, Rev. H: Pile Caps, Plans and Details, LACBWR Generator Plant, Dairyland Power Cooperative, Genoa, Wi. (3/28/63)

41-503433, Rev. D: Reactor Containment Vessel, Piling and Foundation Plan and Section, LACBWR Project, USAEC, Genoa, Wi. (10/17/62)

41-503434, Rev. L: Auxiliary Structures, Foundation and Piling Plan, LACBWR Project-Reactor Plant, Genoa, Wi. (3/18/63)

STRUCTURE	NUMBER OF PILES	ELEV AVE
Reactor Containment	232	+58
Turbine Building	310	+5
Chimney	78	+58
Waste Disposal Building	28	+57
Gas Vault	16	+58



ATIONS
AGE TIP
0 0 ft
9 5 ft
2 0 ft
7.7 ft
0 0 ft

NOTES:

Plant Grade Elevation is +639 ft.

Bottom of Containment Vessel is Elevation +610 ft.

LEGEND:

Proposed Borings

Dairyland Power Cooperative
Genoa, Wisconsin

LACBWR PILING PLAN

DAMES & MOORE

FIGURE 2d-2-1

Question 3

Answer:

Research on Documentation of Heave/Settlement During Pile Driving

DPC personnel looked into several sources of documented information and also contacted the various agencies involved in the LACBWR plant construction to determine if any data regarding heave or settlement during excavation and pile driving had been recorded. Specifically, the following tasks were completed:

1) Reviewed the pile log dated April 10-16, 1975, for IB diesel building
2) Reviewed the pile-driving report for the containment vessel (February 1963, S&L report #SL-2003)
3) Reviewed the generator plant report of pile-driving operations, and the piling log and concrete log
4) Reviewed the construction engineers log of activities (DPC, Buck Dale) from August 1963 to May 1964
5) Reviewed the DPC monthly reports to AEC (Atomic Energy Commission) from August 1962 to May 1964
6) Talked with Mr. Robert Larson (DPC Surveyor during construction)
7) Tried to contact, without success, Maxon Construction Co., in Dayton, Ohio (the contractor for reactor construction)
8) Contacted Mr. David Larson of Sargent and Lundy.

No information related to settlement or heave during or after construction was disclosed by the above research.

Considering the time span between the date of construction, 1962, and the present time, it is highly unlikely that any information related to heave or settlement will be discovered. Therefore, it is safe to assume that no data related to settlement and heave exist in any of the available documents.

Question 4

Answer:

4-1 Density Increase under Reactor Vessel

The density increase of 3 pound/cubic feet under the reactor vessel was estimated by displacement volume of the piles. There was no densification assumed from the vibratory action during pile driving, although in loose sands such densification is likely to occur. By neglecting the possible density increase due to vibratory action, a certain degree of conservatism has been added to D&M's estimates of density increase.

The diameter of the reactor vessel under which the piles were driven at 3.5-foot spacings is approximately 60 feet. However, the effect of the soil displacement by the piles extends beyond the soil cylinder immediately below the reactor vessel. Affected soil cylinders with various diameters were evaluated to assess the influence of the density increase with distance from the piles.

The calculations to estimate the density increase by soil displacement by the 232 piles are summarized in table 4-1-1. The details of calculations are listed below.

D = assumed effective diameter of soil cylinder in which displacement occurs

V_t = volume of effective soil cylinder = $\frac{\pi D^2}{4}$ (for a representative thickness of 1 foot)

V_p = pile volume of 232 piles with average shaft diameter of 11 inches in depths being considered = $\frac{\pi(11/12)^2}{4} \times 232$ (for 1 foot thickness)

$V_t - V_p$ = volume of soil within cylinder after displacement

W_s = weight of dry soil in effective cylinder = $V_t \times \gamma_{di}$

γ_{di} = initial average dry density of soil

γ_d = average dry density of soil after displacement = $W_s/(V_t - V_p)$

Table 4-1-1

D	V_t	V_p	$V_t - V_p$	W_s	γ_d
60 ft	2827 ft^3	153 ft^3	2674 ft^3	294008 lb	110.0 lb/ft^3
65	3318	153	3165	345072	109.0
70	3848	153	3695	400192	108.3
75	4418	153	4265	459472	107.7
80	5027	153	4874	522808	107.3

The calculations shown in the table indicate that the increase over the initial average dry density of 104 pound/cubic foot varies from 6 to about 3 pound/cubic foot as the assumed diameter of the volume affected by the soil displacement varies from 60 to 80 feet. It is likely that the actual increase resulting from displacement alone is somewhere in this range; for conservatism, an average density increase of 3 pound/cubic foot was assumed.

4.2 Density Increase under Other Structures

Similar effects of density increase can be expected under the other structures supported by driven piles. The turbine building, for example, is supported by a number of pile groups. Because piles are not equally spaced under the entire building, there are isolated areas between pile groups that may experience little or no density increase due to soil displacement during pile driving. The effect in the immediate vicinity of each pile group can be estimated similarly to that under the reactor. A summary of these calculations is presented below for a representative group of 24 piles on 3.75-foot spacings. The summary of density calculations is given in table 4-2-1 with the following notations:

A = assumed affected area in which displacement occurs

V_t = volume of affected area for representative 1 foot thickness

V_p = pile volume of 24 piles with average shaft diameter of 11 inches in depths being considered = $\pi(11/12)^2$(for 1 foot thickness)

$V_t - V_p$ = volume of soil in affected area after displacement

W_s = weight of dry soil in affected area = $V_t \times \gamma_{di}$

γ_{di} = initial average dry density of soil

γ_d = average dry density of soil after displacement = $W_s/(V_t - V_p)$.

Table 4-2-1

D	V_t	V_p	$V_t - V_p$	W_s	γ_d
26.5 x 8.5 ft	225.3 ft^3	15.8 ft^3	209.4 ft^3	23426 lb	111.9 lb/ft^3
28.5 x 10.5	299.3	15.8	283.4	31122	109.8
30.5 x 12.5	381.3	15.8	365.4	39650	108.5
34.5 x 16.5	569.3	15.8	553.4	59202	107.0
38.5 x 20.5	789.3	15.8	773.4	82082	106.1

The last calculation in table 4-2-1 indicates an average density increase of 2 pound/cubic foot over an assumed affected area extending 6 feet beyond each side of the pile group. Near this point the influence begins to overlap with that from adjacent pile groups in some locations. The actual increase due to soil displacement is likely to be higher and concentrated closer to the piles.

Similar calculations can be performed for other structures supported on driven piles to estimate density increase only due to soil displacement by the piles.

References

4-1-1 Containment Vessel Pile Driving Operations for 50-MWe Boiling Water Reactor at Genoa, Wisconsin (Sargent and Lundy, February 25, 1963).

4-2-1 Drawing B-3, Revision 5: Piling Plan, LACBWR Generator Plant, Dairyland Power Cooperative, Genoa, Wisconsin (Sargent & Lundy).

Question 5

Answer:

Tabulation of N_1 Values

Attached are plots of N_1 values versus depth for each D&M boring, where N_1 is the measured blow count value corrected to 1 ton/square foot overburden (figures 5-1 through 5-13). Ground surface (depth = 0) is at approximate elevation +639 feet.

Also shown on each plot is a line indicating N_1 values for which the factor of safety equals 1.0 for a given depth. To determine the N_1 values for this line, average cyclic shear stresses from a one-dimensional wave propagation analysis for 0.12-g surface acceleration (from table 6 in the D&M report of September 28, 1979, reference I-4) were compared to effective overburden pressure to yield a cyclic shear stress ratio for each depth. The stress ratios were then related to the line indicating the upper bound for liquefaction potential for the given acceleration, as shown in figure 7 of reference I-4. The corresponding N_1 values were plotted as the line designated (F.S. = 1) in the plots attached here.

These N_1 values represent free-field conditions, relatively unaffected by the densification effect of pile driving.

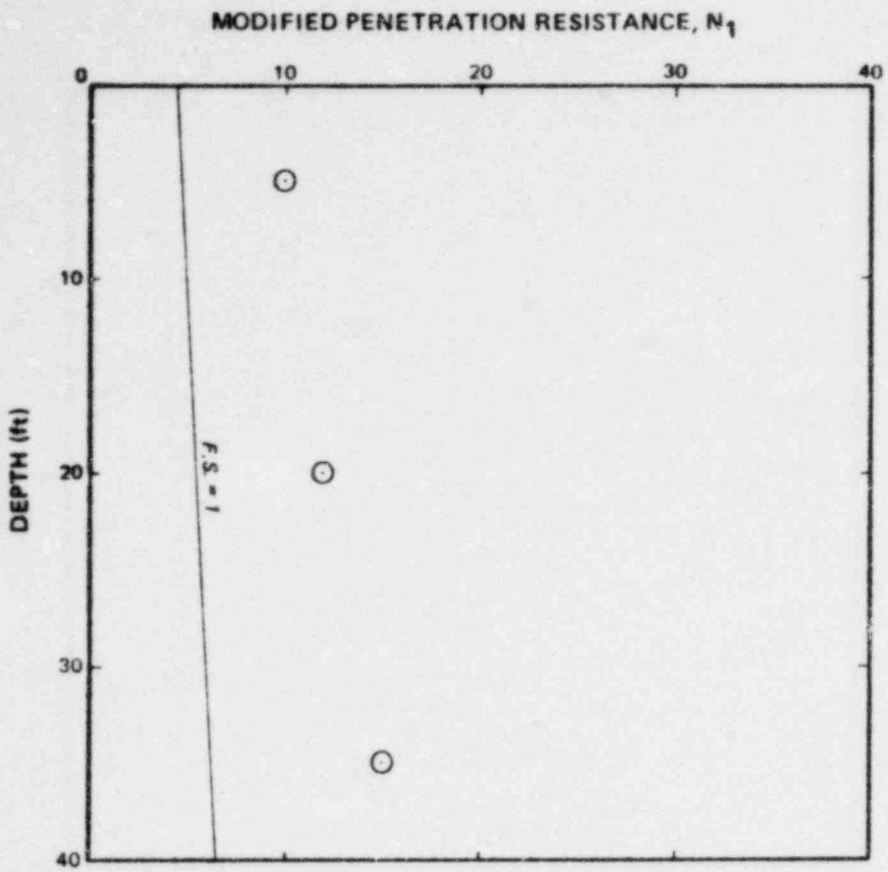


Figure 5-1 Boring DM-1 (1973)

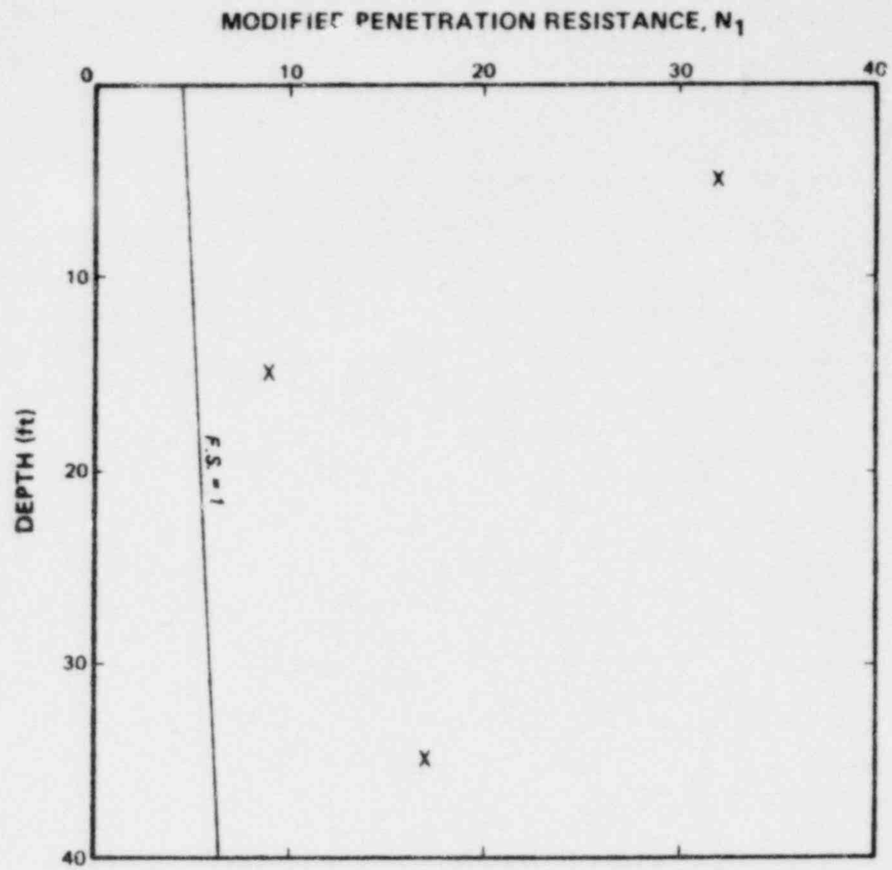


Figure 5-2 Boring DM-2 (1973)

VARIATION OF MODIFIED PENETRATION RESISTANCE WITH DEPTH

DAMES & MOORE

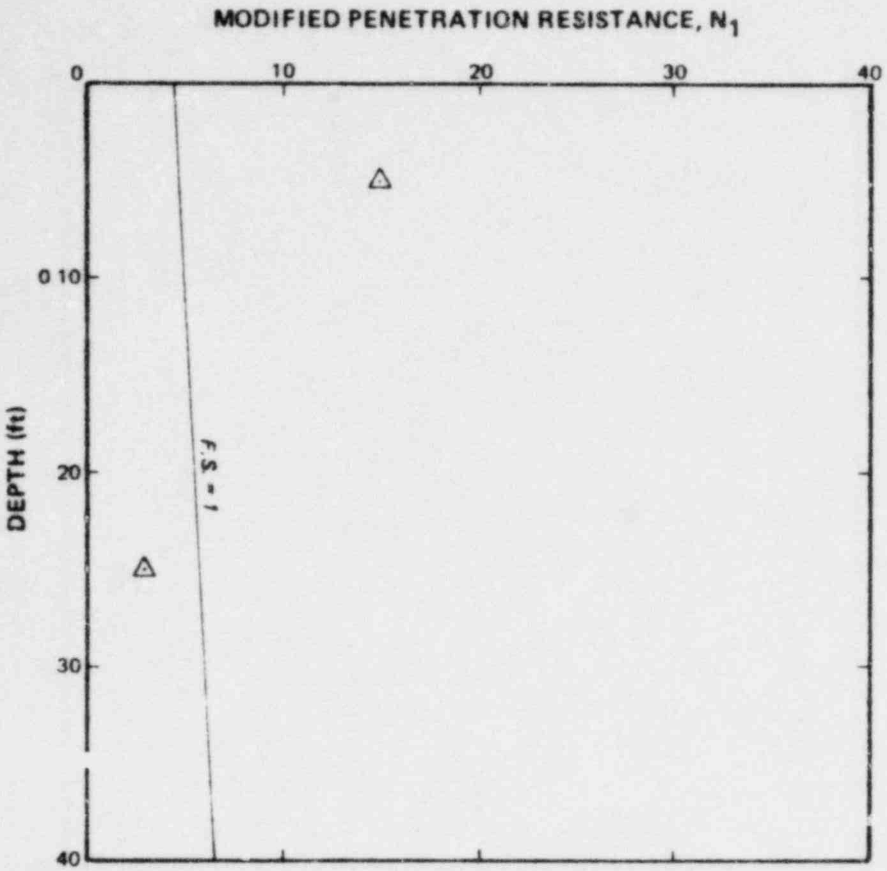


Figure 5-3 Boring DM-3 (1973)

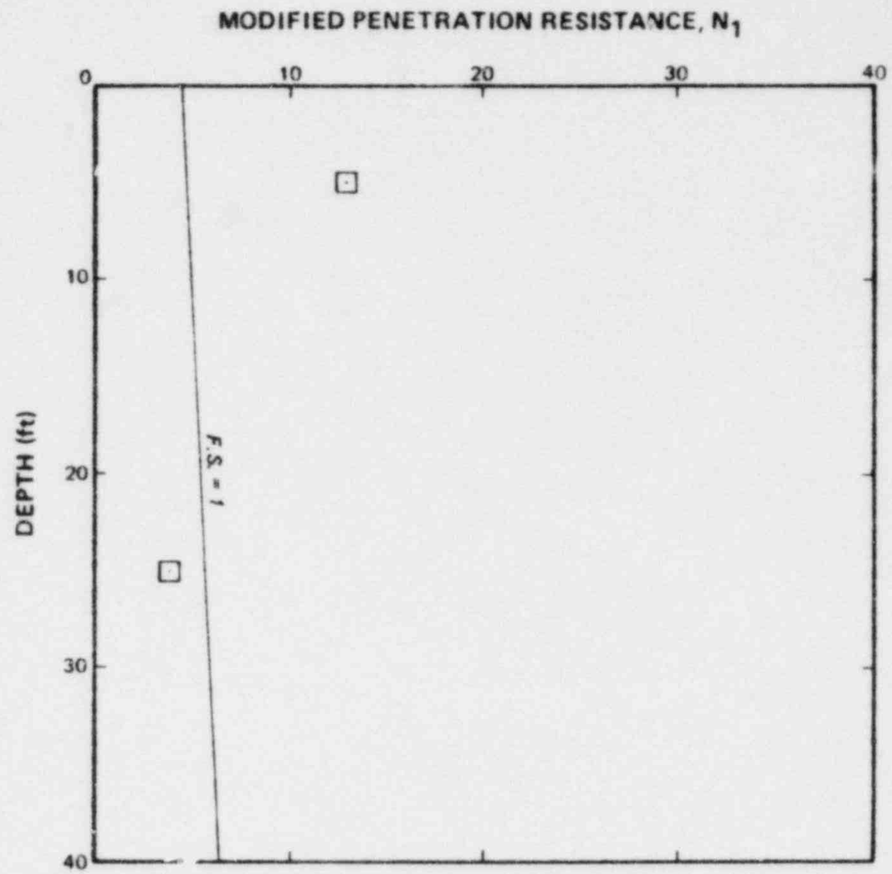


Figure 5-4 Boring DM-4 (1973)

VARIATION OF MODIFIED PENETRATION RESISTANCE WITH DEPTH

DAMES & MOORE

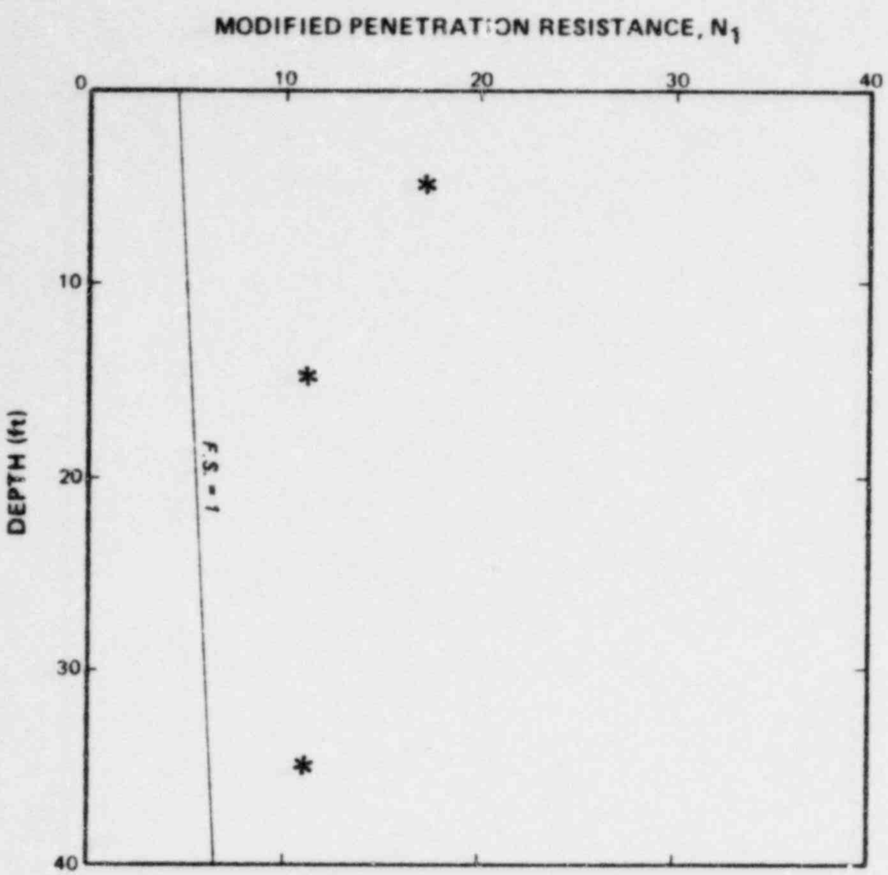


Figure 5-5 Boring DM-5 (1973)

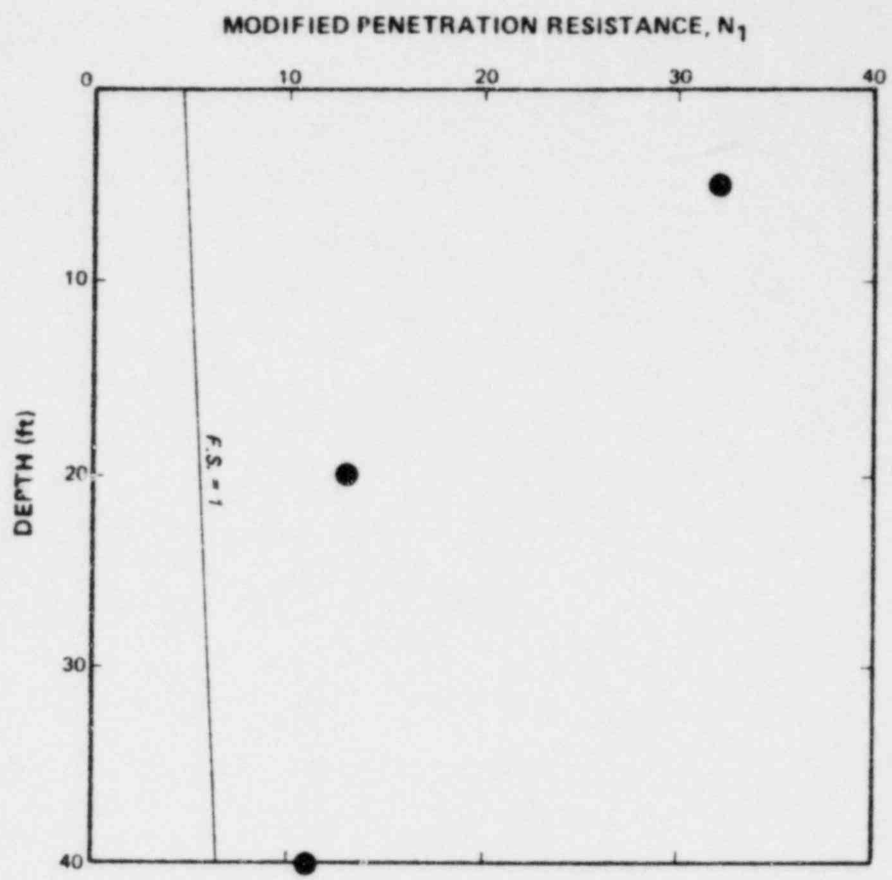


Figure 5-6 Boring DM-6 (1973)

VARIATION OF MODIFIED PENETRATION RESISTANCE WITH DEPTH

DAMES & MOORE

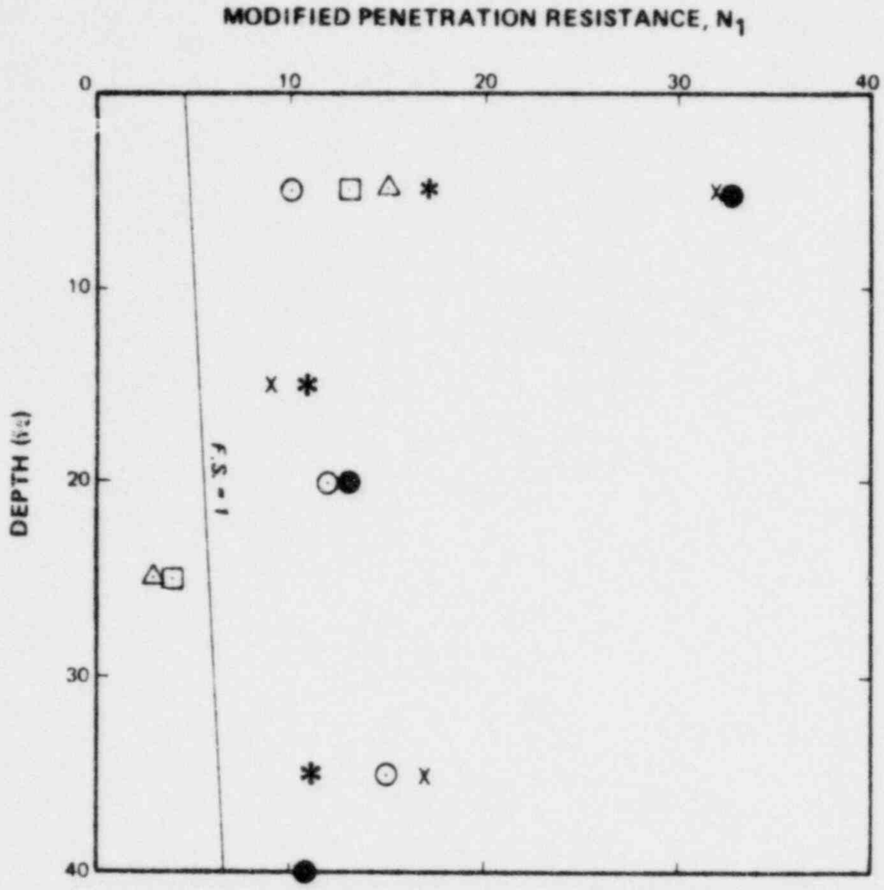


Figure 5-7 All 1973 Borings

KEY:

○ DM-1 ⊡ DM-4
x DM-2 * DM-5
△ DM-3 ● DM-6

VARIATION OF MODIFIED PENETRATION RESISTANCE WITH DEPTH

DAMES & MOORE

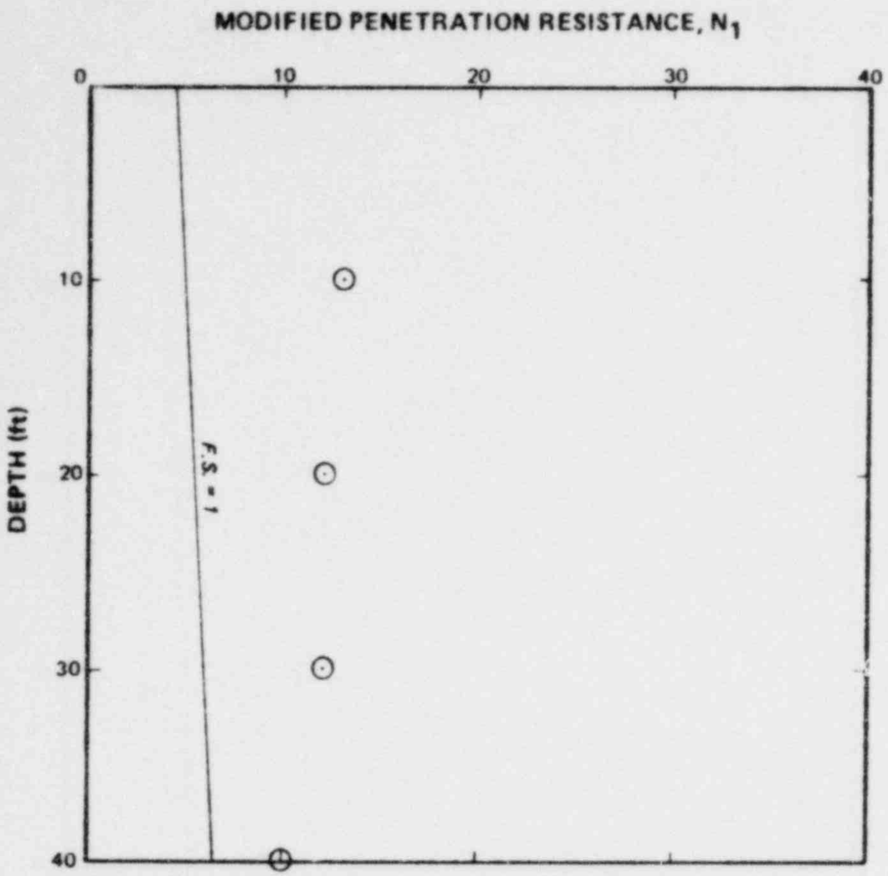


Figure 5-8 Boring DM-7 (1979)

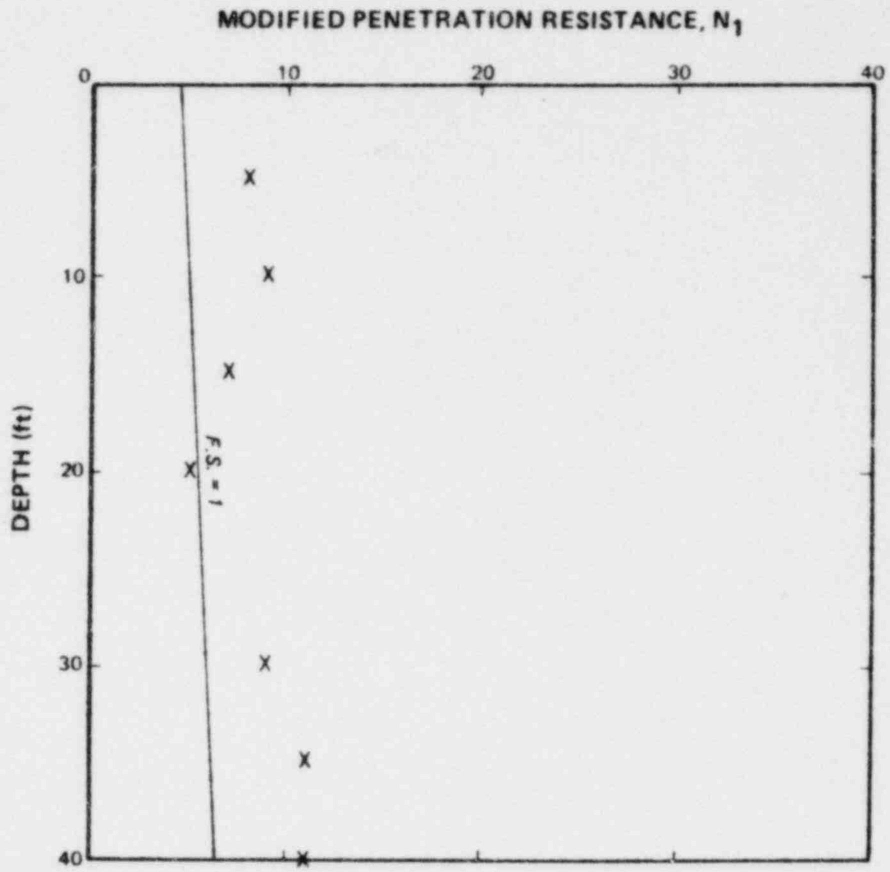


Figure 5-9 Boring DM-8 (1979)

VARIATION OF MODIFIED PENETRATION RESISTANCE WITH DEPTH

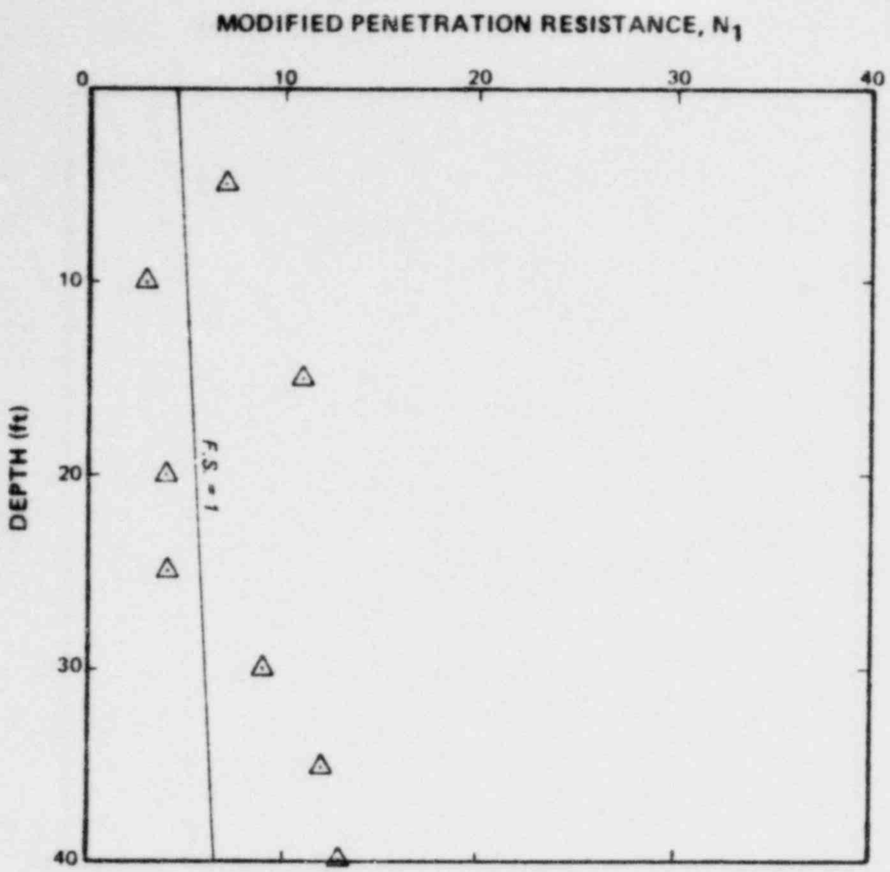


Figure 5-10 Boring DM-10 (1979)

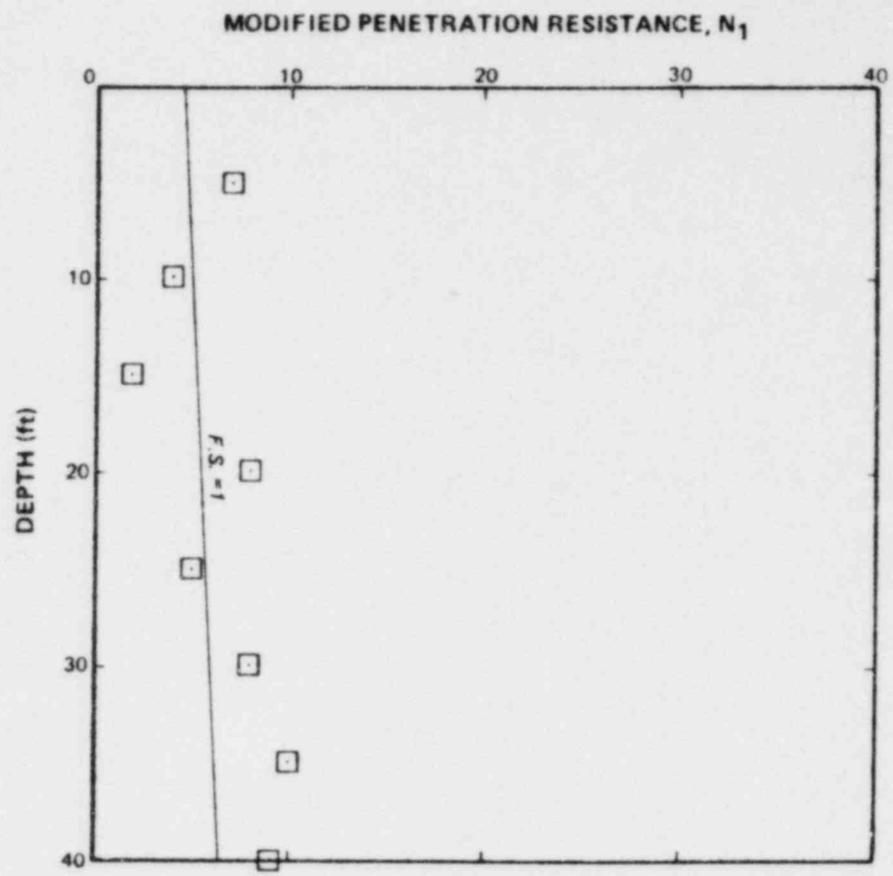


Figure 5-11 Boring DM-11 (1979)

VARIATION OF MODIFIED PENETRATION RESISTANCE WITH DEPTH

DAMES & MOORE

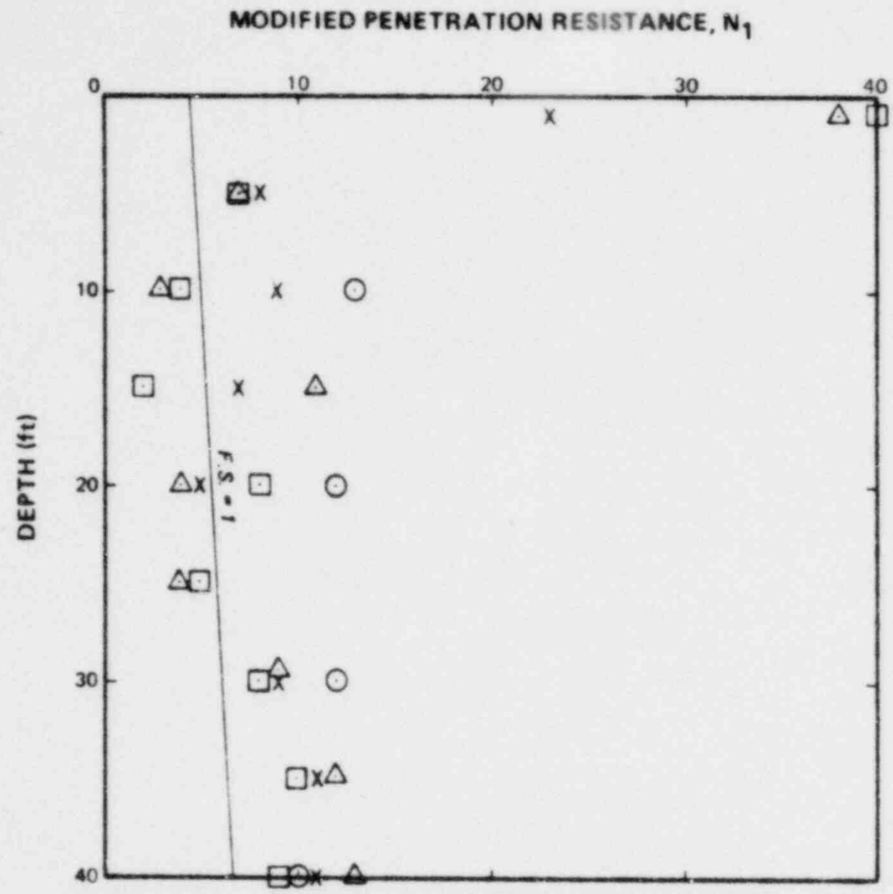


Figure 5-12 All 1979 Borings

KEY:

⊙	DM-7	△	DM-10
X	DM-8	⊡	DM-11

VARIATION OF MODIFIED PENETRATION RESISTANCE WITH DEPTH

DAMES & MOORE

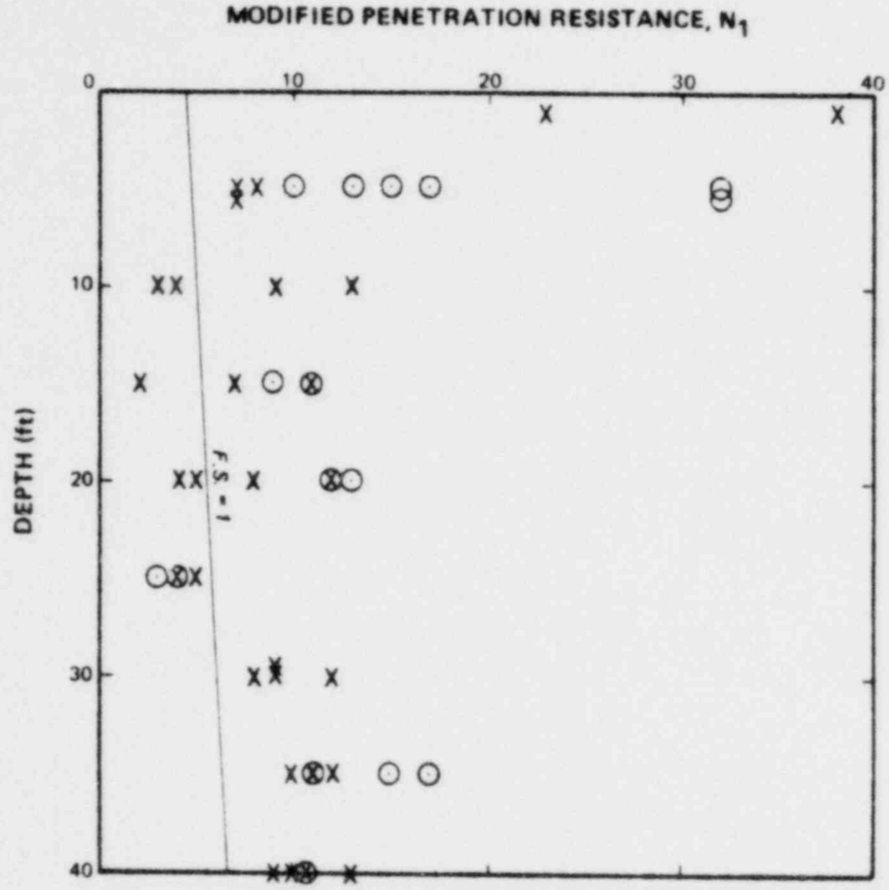


Figure 5-13 All DM Borings

KEY:
○ 1973
X 1979

VARIATION OF MODIFIED PENETRATION RESISTANCE WITH DEPTH

DAMES & MOORE

Question 6

Answer:

Effect of Oiled Rope on SPT Blow Counts

As described in the Response of March 1980 (reference I-5), the SPT N-values used in the D&M analysis were produced by a rope and pulley system for raising the drop weight, with frequent oiling of the rope to minimize friction between the rope and pulley. Dr. Seed observed that the commonly used techniques for collecting blow count data on which the correlations of N-values and field performance of sandy sites during past earthquakes are based did not involve this procedure of minimizing friction. To compensate for the reduced friction in the system used at the LACBWR site, Dr. Seed felt it appropriate to increase the measured blow counts by about 20 percent. However, no data are available to substantiate this judgement quantitatively (reference 6-1).

It should be noted that, even though the above discussion was made in the Response of March 1980, no advantage was taken of the suggested increase in the SPT N-values in the re-evaluation of liquefaction potential by the empirical approach. Although table I of reference I-5 shows the effect of the 20 percent increase on N_1 for each boring (column 1.2 N_1), the N values corrected for the effect of pile driving and for overburden pressure (N_1* in table I) are based on corrected measured blow counts. The N_1* values, which are plotted for the empirical approach in figures 1 through 3 in the Response, do not reflect the 20 percent increase recommended by Dr. Seed. The discussion was brought up solely to demonstrate yet another degree of conservatism introduced in the D&M analysis.

Reference

6-1 Personal communications with Dr. Seed.

Question 7

Answer:

Low Seismic Risk at LACBWR Plant Site

In 1973, an earthquake producing a peak ground acceleration of 0.12-g was designated as the SSE for the LACBWR plant site. NRC is using a seismicity study performed by TERA Corporation for Lawrence Livermore Laboratory as the basis for assigning an SSE for the LACBWR plant site. The TERA study concluded that the return period for an earthquake producing 0.11-g peak ground acceleration at LACBWR plant site is at least 1,000 years and could be larger by an order of magnitude.

D&M, after a preliminary review of the TERA study, concluded that the return period could indeed be an order of magnitude larger. In its Response of March 21, 1980, D&M concluded that a return period of 10,000 years was more likely for a 0.12-g earthquake at LACBWR plant site. Based upon the above conclusion, D&M stated that the seismic risk associated with the LACBWR plant site was very low in view of the short duration of the remaining plant life. D&M also believed (and still believes) that the liquefaction analysis performed using a very low probable event as an SSE would lead to conservative conclusions. As a response to question 7 of NRC, D&M has further examined the basis of the probabilistic study and the influence of various parameters involved. This review confirmed the earlier, preliminary finding that the best estimates of the seismic hazard at LACBWR plant site indicate that a peak horizontal acceleration of 0.11 g has an annual probability of 10^{-4} of being exceeded. (In other words, the return period of an 0.11-g earthquake is 10,000 years.) Details of the probabilistic analysis performed by Dr. Robin McGuire of D&M can be found in the accompanying appendix.

APPENDIX

Introduction

The purpose of this study is to make a probabilistic assessment of the seismic ground motion hazard at the La Crosse Boiling Water Reactor (LACBWR) site near Genoa, Vernon County, Wisconsin. The results of this study will be used to assess the likelihood of seismically - induced soil liquefaction at the facility.

A guiding principle in this study is that a "mean-centered" or best estimate analysis should be derived. Further, uncertainty in the best estimate relationship between ground motion levels and probabilities of exceedance will be investigated by examining uncertainty in the various assumptions critical to the analysis. To accomplish this, several hypotheses will be examined for seismogenic zones and attenuation functions.

Several sources of data are used as input to this analysis. The work of TERA Corp (1979) summarizes a wide range of opinion and expertise on seismicity in the central and eastern U.S., and the work of Nuttli and Herrmann (1978) provides an analysis of historical seismicity in the central U.S. The Nuttli earthquake catalog (Nuttli, 1979a) is the source of historical earthquake data used here.

The specific site examined in this study is the LACBWR facility, Vernon County, Wisconsin. The assumptions and hypotheses examined are appropriate for this site, but may not be for other sites. As an example, certain alternate configurations of seismogenic zones in the central U.S. may be appropriate for the analysis of seismic hazard at other sites in the central U.S. These alternate configurations were not examined here because they would have no appreciable effect on the calculated seismic hazard at the LACBWR facility.

Seismic Hazard Model

The seismic hazard model used in this study has been described elsewhere in detail (Cornell, 1968, 1971; McGuire, 1976), and this description will not be repeated here. Briefly, this model uses several basic assumptions:

1. Successive earthquakes are independent in size and location; their annual probabilities of occurrence in seismogenic zones are accurately estimated by the activity rates observed historically in these zones;
2. The relative distribution of earthquake magnitudes in seismogenic zones can be represented by a truncated exponential distribution; and
3. The peak acceleration at the site of interest can be represented as a function of the earthquake magnitude, the distance between the site and the source of energy release, and the local soil conditions.

Given these assumptions, the probabilistic hazard analysis consists of mathematically integrating over all possible earthquake magnitudes and locations, calculating for each magnitude and location the distribution of peak horizontal acceleration at the site, to evaluate the probability that various levels of acceleration will be exceeded annually. A standard computer program (McGuire, 1976) was used for calculations.

Seismogenic Zones

The seismic hazard analysis requires the delineation of seismogenic zones, within which earthquakes are considered to be of similar tectonic origin so that future seismic events can be modeled by a single function describing earthquake occurrences in time, space, and size. The initial seismogenic zones examined here were those of Nuttli and Herrmann (1978). It became evident that the major contributors to seismic hazard at the LACBWR site are earthquakes which occur in northern Illinois, and those which occur in the so-called "Central Stable Region" of the central U.S. Other seismogenic zones contribute relatively little to the seismic hazard; they were included in the analyses for completeness, using the boundaries and parameters suggested by Nuttli and Herrmann (1978), but their exact delineation and seismicity parameters are immaterial for the present study.

Two alternate hypothesis were examined for seismogenic zones in the vicinity of the site:

1. Central Stable Region (CSR). Under this hypothesis, earthquakes which have occurred historically in northern Illinois were assumed to be a part of the Central Stable Region of the central U.S., and no specific seismogenic zone was delineated to model the occurrence of earthquakes in northern Illinois in the future. For the purposes of this study, the Central Stable Region was taken to be the area between 39° and 49° north latitude, and between 82° and 96° west longitude, excluding the areas reported as seismic sources by Nuttli and Herrmann (1978). The largest historical earthquake in this area had an estimated magnitude (m_b) of 5.3.

2. (a) Wisconsin Arch zone. Earthquakes in northern Illinois were attributed to a seismogenic zone bounded to the north by the southern extent of the Wisconsin Dome, to the east by the western extent of Silurian rocks, to the south by the northern extent of Pennsylvanian rocks, and to the west by the

Mississippi River arch (see Figure 1). The area of this zone includes the larger historical events which have been reported in northern Illinois and southern Wisconsin. The delineation of this zone (Figure 1) is somewhat different from that suggested by Nuttli and Herrman (1978), but this difference is immaterial for the present study.

(b) Central Stable Region (CSR). Under the second hypothesis, the CSR was taken to be the area defined in the first hypothesis but excluding the Wisconsin Arch zone (Figure 1). The largest historical earthquakes in the CSR under this hypothesis had an estimated magnitude (m_b) of 4.5 (several larger historical earthquakes are thought to be over-rated [Nuttli, personal communication, 1980]).

These two hypotheses, which represent a range of possible seismogenic zones in the vicinity of the site, were examined in detail. Results for each hypothosis are reported below. While other seismogenic zones might be hypothesized which would indicate larger (or smaller) seismic hazard at the site, it is felt that no such zones can be justified on a geological basis, given the present understanding of tectonic processes in the central U.S.

Seismicity Parameters

For the probabilistic calculation of seismic hazard, several parameters describing seismicity are required for each seismogenic zone. These parameters, and the methods used to estimate mean values, are discussed below.

Seismic Activity Rate. The rate of earthquake occurrence was estimated for each seismogenic zone using the historical seismicity in that zone as reported by Nuttli and Herrmann (1978). Several modifications were made to the activity rates reported by Nuttli and Herrmann (1978), as follows:

1. Data were corrected to account for the fact that Nuttli and Herrmann (1978) plotted observed cumulative rates of activity at the center of 0.5 unit magnitude intervals, rather than at the lower end of the interval. The latter procedure is more appropriate for cumulative plots of seismic activity.
2. Activity rates were calculated for occurrences of earthquakes with $m_b \geq 4.6$. This decision was based on the presumption that events with M_L less than 5 do not cause soil liquefaction due to the short duration (small number of cycles) of strong shaking involved. Using the Nuttli (1979b) relation:

$$M_L = 1.023\ m_b + 0.3$$

a lower-bound M_L of 5 corresponds to a lower-bound m_b of 4.6.

The method used by Nuttli and Herrmann (1978) to account for incomplete reporting of small events was reviewed and found to be adequate.

Richter b-value. The Richter b-value describes the slope of the log-number versus magnitude relation:

$$\log_{10} n(m_b) = a - bm_b \qquad (1)$$

where $n(m_b)$ is the annual number of earthquakes of body-wave magnitude m_b, and a and b are parameters fit to seismicity data. Parameter a is related to the seismic activity rate discussed in the previous paragraph. The average Richter b-value was taken to be 0.92 for all seismogenic zones, the value reported by Nuttli and Herrmann (1978). The experts polled in the TERA Corp (1979) study generally felt a single b-value for all zones in the central and eastern U.S. is appropriate. The value of 0.92 is typical of numbers offered by the experts (b-values for Modified Mercalli intensity I_o were converted to b-values for m_b using the Nuttli and Herrmann (1978) relation $I_o = 2m_b - 3.5$).

Maximum Magnitude. The maximum body-wave magnitude $m_{b,max}$ of each hypothesized seismogenic zone was assigned using the expert opinions reported in the TERA Corp (1979) study, and using subjective judgment. For the first hypothesis on seismogenic zones, the maximum magnitude $m_{b,max}$ was taken to be 5.8 for the CSR. Using the Nuttli and Herrmann (1978) relation $I_o = 2m_b - 3.5$, this corresponds to an epicentral Modified Mercalli intensity I_o of VIII, which is typical of the values suggested by experts in the TERA Corp (1979) study. For the second zone hypothesis the same value (5.8) of $m_{b,max}$ was used for the Wisconsin Arch zone, and a value of 5.0 was used for the CSR. The latter value is appropriate if regions of large historical earthquakes in the central U.S. are accounted for by specific seismogenic zones (Nuttli, personal communication, 1980).

Table 1 presents the two hypotheses on seismogenic zones and their associated seismicity parameters. No uncertainty in activity rates, b-values, or maximum magnitudes was included in this analysis because the use of best-estimate point values for these parameters results in best estimate seismic hazard curves (McGuire and Shedlock, 1980).

Estimation of Seismic Ground Motion

Estimates of peak single-component horizontal ground acceleration for an earthquake of given magnitude m_b and epicentral distance Δ were made following the theory of Nuttli (1979b) for higher mode surface waves. This theory estimates a sustained level of acceleration corresponding to the third highest peak in the acceleration time history, for earthquakes of several magnitudes. An equation was fit to this theory to allow estimation of sustained acceleration a_s for a continuous range of magnitudes and distances:

$$a_s = 0.584 \exp(-0.427 \exp(-.444m_b) + 1.098m_b) \qquad \Delta < 10km$$

$$(2)$$

$$a_s = 3.98 \ \Delta^{-5/6} \exp(-.0427\Delta \ \exp(-.444m_b) + 1.098m_b) \quad \Delta \geq 10km$$

These equations are appropriate for estimating sustained acceleration at sites underlain by soils, and thus are appropriate for the LACBWR site. To estimate peak acceleration, the sustained acceleration was multiplied by the factor 1.4 (Nuttli, 1979b).

There is a second modification to Nuttli's theory required to estimate peak acceleration. Nuttli's work was based on, and calibrated to, the larger of the two horizontal components, whereas we wish to estimate the peak horizontal acceleration in a randomly-oriented direction. The appropriate factor (mean ratio of the peak of a randomly chosen horizontal component to the larger of the two peak horizontal component accelerations), is 0.9, based on an analysis of the data used by Nuttli.

Combining these two effects into a single factor of 1.26 (1.4 times 0.9), we estimate the peak sustained-based acceleration a_{ps} as:

$$a_{ps} = 1.26a_s \qquad (3)$$

This acceleration is plotted as a function of distance for several values of m_b in figure 2.

Several alternate equations were examined for estimating peak horizontal acceleration at soil sites in the central U.S. The relation reported by Nuttli and Herrmann (1978):

$$a_{p,nh} = 6.92\, e^{1.2m_b} \Delta^{-1.02} \qquad \Delta \geq 15\text{km} \qquad (4)$$

$$= 0.437\, e^{1.2m_b} \qquad \Delta < 15\text{km}$$

is appropriate for estimating peak horizontal vector acceleration. Multiplication by the factor 0.7, based on the Nuttli (1979b) data set (and confirmed independently by Herrmann) gives an estimate of peak horizontal component acceleration. The modified relationship is plotted in figure 2 and gives values almost identical to the modified Nuttli theory. Use of equation 4 in the hazard analysis gives results which are virtually identical to those obtained using Nuttli's modified theory; hence equation 4 is not examined further in this study.

Two alternate methods of estimating peak horizontal acceleration at soil sites in the central U.S. were examined: those of TERA Corp (1980) based on intensity attenuation observed during the Ossippee earthquake, and based on the Gupta and Nuttli (1976) isoseismal attenuation. Accelerations estimated by these two methods, respectively, are as follows:

$$a_{TO} = \exp(-0.78 + 1.12m_b - 0.007\Delta - 0.189 \ln\Delta) \qquad (5)$$

$$a_{TGN} = \exp(+0.74 + 1.12m_b - 0.0007\Delta - 0.732 \ln\Delta) \qquad (6)$$

The equations published by TERA Corp (1980) based on Modified Mercalli intensity I_o were converted to body-wave magnitude using the Nuttli and Herrmann (1978) relation $I_o = 2m_b - 3.5$. Estimates based on these equations are shown in Figure 2. The effects of these alternate equations on calculated seismic hazard are examined below.

For calculation of seismic hazard, a lognormal distribution of acceleration about the mean value was assumed, with a value of 0.6, corresponding to a factor of 1.8 uncertainty in the estimate. This distribution is widely used to represent uncertainty in ground motion estimates; the uncertainty modeled is typical of the scatter exhibited by strong motion data sets, as shown in Table 2, when the data are restricted to a specific area such as the western U.S. Some of the studies listed in Table 2 (Shannon and Wilson, Inc., and Agbabian Assoc., 1979, and Trifunac, 1976) are heavily biased by data from the San Fernando earthquake; others (McGuire, 1974, 1978) are not. When data from world-wide locations are used in the analysis, larger values of uncertainty are obtained because of different mean attenuations. In this study we prefer to use an uncertainty typical of a specific geographic area, and model uncertainty in the mean attenuation directly by examining several attenuation functions.

Results of Analysis

Table 3 presents results in terms of peak accelerations with 10^{-3}, 10^{-4}, and 10^{-5} annual probabilities of exceedance, for the various hypotheses on seismogenic zones and attenuation functions. The Central Stable Region (CSR) indicates a slightly larger hazard than the Wisconsin Arch - CSR zone hypothesis, because the former implies that earthquakes with m_b up to 5.8 can occur at the site. The results are not sensitive to which attenuation function is used because there are compensating effects of one equation indicating larger accelerations at some distances but smaller accelerations at other distances, than its counterparts. For the two TERA attenuations, other seismogenic zones (modeled here after Nuttli and Herrmann, 1978, as discussed above) in the midwest U.S. contribute slightly to the hazard, although the major contributors are the CSR and the Wisconsin Arch zone.

The effect on peak acceleration of truncating the lognormal distribution was examined by truncating the distribution at two standard deviations from the mean (and renormalizing the truncated distribution so it remained a proper probability density function with unit area). Table 4 shows results for this truncation: the differences from Table 3 are not large. Although truncating the acceleration distribution is sometimes mentioned as a reasonable procedure for mean-centered seismic hazard analyses (e.g. TERA Corp, 1980), the number of standard deviations at which truncation should be made is a matter of opinion. Since the results here are apparently not sensitive to this truncation, at least for two standard deviations, this effect is not examined further.

Table 3 presents Bayesian estimates of acceleration, obtained by weighting the various hypotheses. Equal weights were used for the attenuation equations, since these indicate similar seismic hazards. The second seismogenic zone hypothesis, the Wisconsin Arch zone and CSR, was assigned a subjective weight of 2/3, since at least one seismologist familiar with the midwest U.S. feels strongly that a zone in northern Illinois exists (Nuttli, personal communication, 1980), although its precise boundaries are not well defined. Most of the experts in the TERA Corp (1979) study did not indicate such a zone, perhaps because of a lack of familiarity with seismicity in the region. The first seismogenic zone hypothesis, the CSR alone, was assigned a subjective weight of 1/3.

Summary

We present here an analysis of seismic hazard at the LACBWR facility, Vernon County, Wisconsin. The results are insensitive to which of a set of peak acceleration attenuation equations is used in the analysis, but is somewhat sensitive to the seismogenic zones used to represent seismicity in the central U.S. Best estimates of the seismic hazard indicate that a peak horizontal acceleration of 11%g has an annual probability of 10^{-4} of being exceeded at the site. Accelerations of 4.5%g and 22%g have associated probabilities of 10^{-3} and 10^{-5}, respectively.

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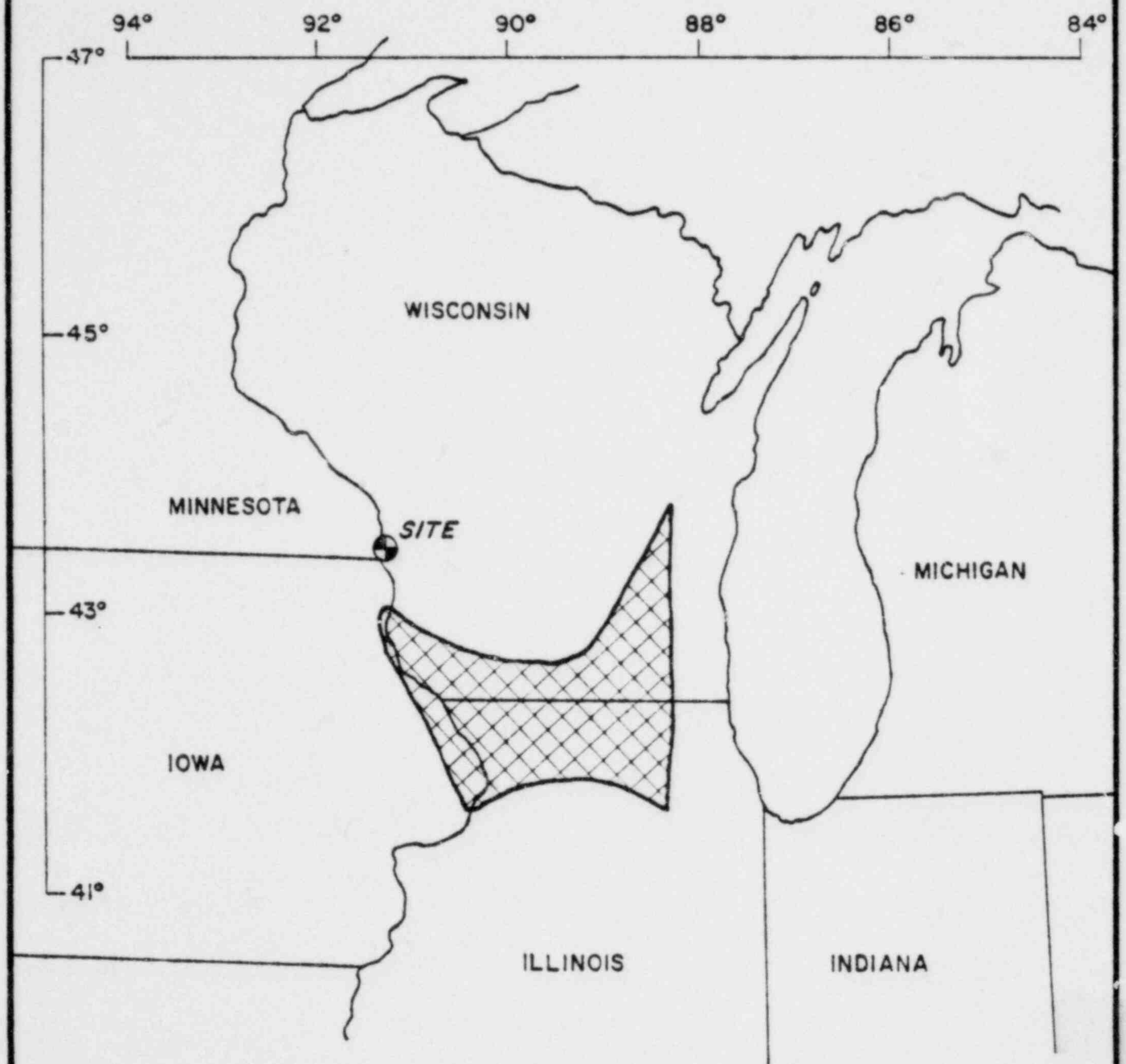

FIGURE 1
WISCONSIN ARCH SEISMOGENIC ZONE

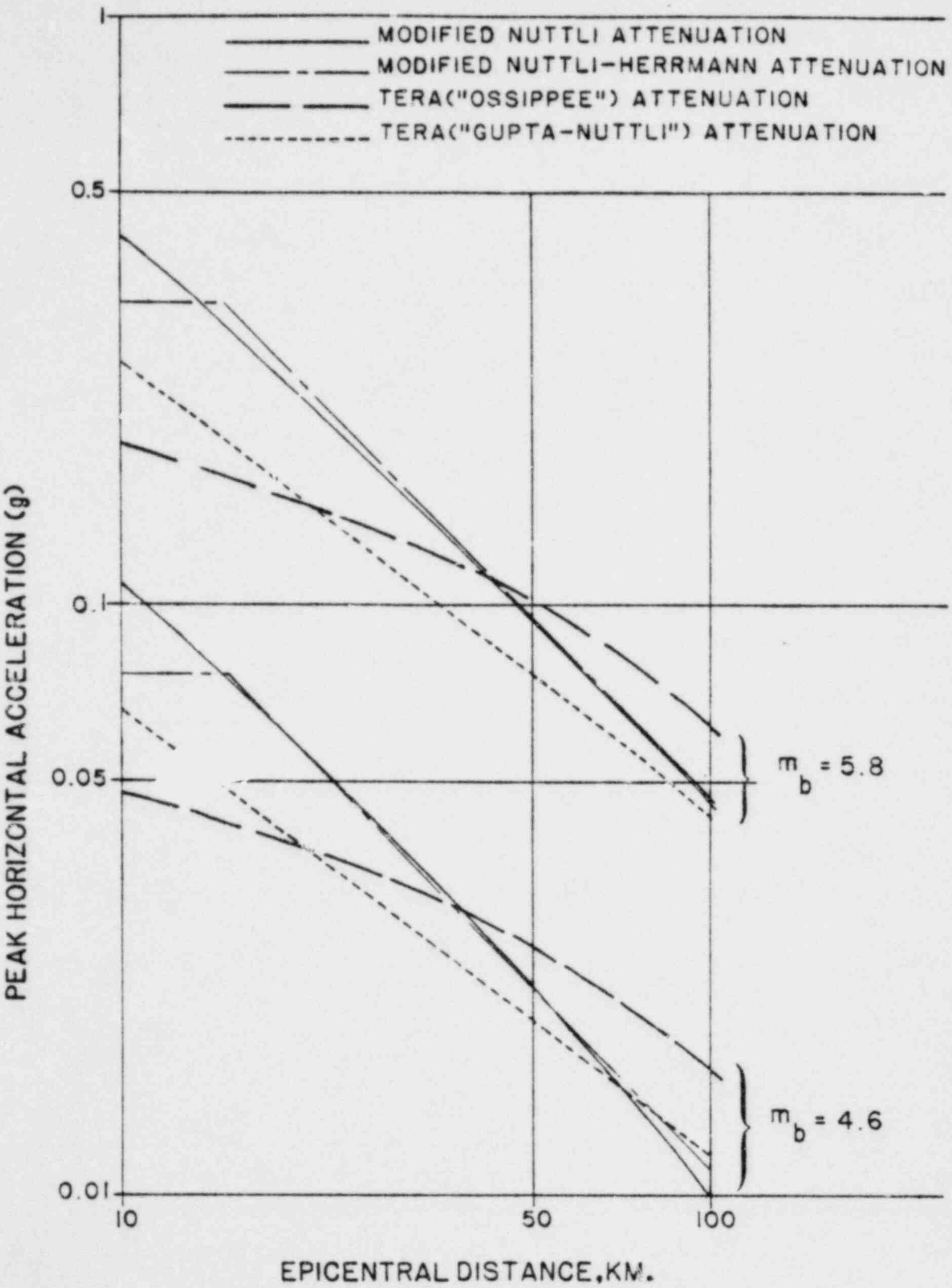

FIGURE 2

ESTIMATED ACCELERATION FOR FOUR ATTENUATION FUNCTIONS

TABLE 1

SEISMOGENIC ZONES AND ASSOCIATED SEISMICITY PARAMETERS

Hypothesis	Seismogenic Zone(s)	Activity Rate (Events per year with $m_b \geq 4.6$)	Richter b-value	$m_{b,max}$
1	Central Stable Region	7.0×10^{-4}*	0.92	5.8
2	Wisconsin Arch	0.0184	0.92	5.8
	Central Stable Region	5.3×10^{-4}*	0.92	5.0

* Annual rate per 10,000 km^2.

TABLE 2

UNCERTAINTIES REPORTED FOR ATTENUATION EQUATIONS

Reference	Data Base	$\sigma_{\ell n\ a}$
Donovan (1973)	World-wide	0.84
Donovan (1974)	San Fernando	0.481
Donovan (1974)	World-wide	0.707
Esteva & Villaverde (1974)	Western U.S.	0.64
McGuire (1974)	Western U.S.	0.51
McGuire (1978a)	Western U.S.	0.62
Shannon and Wilson, Inc., and Agbabian Assoc. (1979)	Western U.S.	0.573
Trifunac (1976)	Western U.S.	0.60*

* Calculated using procedure discussed in McGuire (1978b)

TABLE 3

SEISMIC HAZARD RESULTS
(with no truncation in acceleration distribution)

Seismogenic Zone Hypothesis	Attenuation	Peak Acceleration (g) for annual probability of		
		10^{-3}	10^{-4}	10^{-5}
Central Stable Region	Modified Nuttli	0.039	0.13	0.33
	TERA("Ossippee")	0.051	0.13	0.25
	TERA("Gupta-Nuttli")	0.050	0.11	0.24
Wisconsin Arch and Central Stable Region	Modified Nuttli	0.034	0.093	0.22
	TERA ("Ossippee")	0.050	0.11	0.19
	TERA ("Gupta-Nuttli")	0.048	0.096	0.17
Bayesian estimates		0.045	0.11	0.22

TABLE 4

SEISMIC HAZARD RESULTS
(with acceleration distribution truncated at two standard deviations)

Seismogenic Zone Hypothesis	Attenuation	Peak Acceleration (g) for annual probability of		
		10^{-3}	10^{-4}	10^{-5}
Central Stable Region	Modified Nuttli	0.035	0.12	0.30
	TERA ("Ossippee")	0.048	0.11	0.21
	TERA ("Gupta-Nuttli")	0.044	0.096	0.20
Wisconsin Arch and Central Stable Region	Modified Nuttli	0.032	0.083	0.19
	TERA ("Ossippee")	0.045	0.090	0.14
	TERA ("Gupta-Nuttli")	0.042	0.077	0.13