LIQUEFACTION POTENTIAL
AT LA CROSSE BOILING WATER REACTOR (LACBWR) SITE
NEAR GENOA, VERNON COUNTY, WISCONSIN

Prepared by

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Prepared for

Dairyland Power Cooperative
La Crosse, Wisconsin 54601

August 10, 1979

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

Attention: Mr. R. E. Shimshak
Plant Superintendent

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Gentlemen:

We submit herewith three copies of our draft report, "Liquefaction Potential at La Crosse Boiling Water Reactor (LACBWR) Site, Near Genoa, Vernon County, Wisconsin," for your review. This report includes:

a) Brief summaries of all previous liquefaction analyses performed at LACBWR Site and related background studies;

b) Details of field, laboratory and analytical investigations that were performed to verify the earlier findings regarding liquefaction potential at LACBWR Site; and

c) Our conclusions based on rigorous analyses and sophisticated testing performed on undisturbed samples obtained by utilizing state-of-the-art techniques.

We have concluded in our study that a threshold liquefaction resistance level for the LACBWR site corresponds to an SSE producing an acceleration of 0.20g at the ground surface.

The scope of services for this report was prepared by us after discussions with Mr. Richard Shimshak of Dairyland Power Cooperative. We will look forward to finalizing this report soon after receiving your review comments. In the meantime, the report is being technically reviewed to fulfill the quality assurance requirements.

Very truly yours,

DAMES & MOORE

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1.0 INTRODUCTION

1.1 General

In 1973, Dames & Moore (D&M) performed a Geotechnical Investigation of Geology, Seismology, and Liquefaction Potential at the LaCrosse Boiling Water Reactor (LACBWR) site (Ref. 1). This study was conducted for Gulf United Nuclear Fuels Corporation. D&M's report was submitted to the U.S. Nuclear Regulatory Commission (NRC) in 1974, as part of the application for an operating license for the LACBWR plant (Ref. 2). In the study, D&M concluded that the LACBWR plant had adequate factors of safety against potential for liquefaction under the design Safe Shutdown Earthquake (SSE).

NRC initiated a review process under its Systematic Evaluation Program (SEP) in 1978. As a part of SEP, the U.S. Army Engineer Waterways Experiment Station (WES) was requested by NRC to review the 1973 D&M soils investigation. After reviewing the data and analyses presented by D&M, WES performed its own analyses based on interpretations of the same data. The WES report submitted to NRC, and made public in 1978 (Ref. 3), concluded that the factors of safety against liquefaction potential were considerably lower than those calculated by D&M.

Upon request of the Dairyland Power Cooperative (DPC), D&M reviewed the WES report and reevaluated its 1973 report in view of the WES analyses. Based on this effort, D&M presented to NRC a position which was essentially consistent with its 1973 study. It was decided during the meeting with NRC on February 9, 1979, that a written report should be prepared summarizing the meeting, the reviews made, and the various analyses on liquefaction potential for the LACBWR site. Accordingly, a report (Ref. 4) was submitted to NRC in which D&M reiterated its earlier stand that the LACBWR site had adequate factors of safety against potential for liquefaction under the design SSE. However, certain questions raised by NRC regarding the lack of test data on undisturbed samples and the lack of continuous standard penetration test results could not be satisfactorily answered with the existing data. Therefore, DPC agreed to perform modest field and laboratory investigations and limited analyses to verify the earlier findings on liquefaction potential.

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In its March 1979 report (Ref. 4), D&M recommended a modest program consisting of a minimum of four test borings, undisturbed sampling, and cyclic triaxial testing and analyses. After review of the D&M report, NRC approved the proposed geotechnical program and suggested minor modifications.

1.2 Purpose and Scope

The purpose of this report is to summarize all of the liquefaction analyses performed at the LACBWR site--(a) D&M (1973), (Ref. 1); (b) WES (1978), (Ref. 3); (c) D&M (1979), (Ref. 4); and (d) NRC/WES (April 1979), (Ref. 5). Additionally, new analytical investigations have been conducted to verify prior findings on liquefaction potential. The report is organized as detailed below:

a. Brief summary of the Dames & Moore soils investigation of 1973 (Section 2.0).

b. Brief summary of the WES analysis of 1978 (Section 3.0).

c. Brief review of the WES report and discussions on the approach taken by NRC (represented by WES), (Section 4.0).

d. Summary of the reevaluation of the report of 1973 (Section 5.0).

e. Conclusions and recommendations for further work presented by D&M in March 1979 (Section 6.0).

f. Review comments by NRC/WES on D&M conclusions and recommendations (Section 7.0).

g. Details of the current field, laboratory, and analytical investigations performed by D&M to verify earlier findings on liquefaction analyses at the LACBWR site (Sections 8.0, 9.0, and 10.0).

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2.0 SUMMARY OF DAMES & MOORE GEOTECHNICAL INVESTIGATION OF 1973

Two studies were performed by Dames & Moore in 1973--a study of geology and engineering seismology, and an investigation of static and dynamic soil properties and evaluation of liquefaction potential. A report containing the results of these studies was prepared in 1973 (Ref. 1) and was presented to NRC as a part of the application for an operating license for the LACBWR plant. The conclusions of this report are discussed in Sections 2.1, 2.2, and 2.3.

2.1 Geology

LACBWR is situated within the Central Stable Region of the North American continent. This region includes the dense igneous and metamorphic rocks of the Canadian Shield and adjacent early Paleozoic sedimentary strata. The geologic structure of the Central Stable Region is relatively simple. Other than uplift and subsidence, very little structural activity has occurred in this quiescent area since Proterozoic time.

The region is characterized by a system of broad, circular-to-elliptical erosional uplifts--the Wisconsin and Ozark Domes, and three sedimentary basins--the Forest City, Michigan, and Illinois Basins. The site is located on the western flank of the Wisconsin Arch, a southern extension of the Wisconsin Dome.

Minor structures, consisting primarily of synclines and anticlines of low relief, show no preferred orientation. They are superimposed on the broader features in the region. Faults in the region are believed to have been dormant since late Paleozoic time, i.e., for at least 200 million years. The Paleozoic strata and overlying unconsolidated sediments are essentially undeformed within about 50 miles of the site.

LACBWR is located within the Wisconsin Driftless section of the Central Lowland physiographic province. This section is characterized by flat-lying, maturely dissected sedimentary rocks of early Paleozoic age. Moderate-to-strong relief has been produced on the unglaciated landscape which has been modified only slightly by a mantle of loess and glacial outwash in the larger stream valleys of the area.

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The LACBWR facilities are situated on about 20 feet of hydraulic fill overlying 100 to 130 feet of glacial outwash and fluvial deposits on the east flood plain of the Mississippi River Valley. The surface configuration of the underlying bedrock is unknown because of the relative paucity of borehole data. The bedrock below the site consists of nearly flat-lying sandstone and shales of the Dresbach Group (Upper Cambrian). Dense Pre-cambrian crystalline rock underlying these sedimentary rocks is estimated to be at a depth of 650 feet.

2.2 Seismology

Based on the seismic history and the tectonics of the region, D&M concluded that the site will not experience any significant earthquake-induced ground motion during the remaining economic life of the nuclear facility. Historically, there is no basis for expecting ground motion of more than a few percent of gravity at the LACBWR plant site. However, three possible sources of earthquake motion at the site were considered.:

a. The nearest zone of repeated earthquake activity, which is in northern Illinois-southern Wisconsin.

b. The effect of a series of events such as those which occurred in 1811-1812 near New Madrid, Missouri.

c. The effect of several shocks in the region which have not been related to any currently identifiable geologic structure or tectonic feature.

After a careful evaluation of these possible sources of earthquake motion and their possible effect on the LACBWR site, it was concluded that the SSE should be considered as the occurrence of an MM Intensity VI shock with its epicenter close to the site. It was estimated that the maximum horizontal ground acceleration induced by such an event would be 12 percent of gravity at the ground surface.

2.3 Liquefaction Potential

The liquefaction potential of the granular soils underlying the existing plant was analyzed by comparing the anticipated shear stresses due to the SSE with the shear stresses required to produce liquefaction at various depths. The analysis was confined to the upper strata (from the ground surface to a depth of 100 feet) of the zone of potential liquefaction. To provide pertinent subsurface data for this analysis, a field exploration and laboratory program of index properties tests and dynamic tests was conducted.

The factors of safety against liquefaction were calculated for various depths. The calculations were based on 10 significant stress cycles, following the engineering practice of 1973. The results of the analysis indicated that the calculated minimum factor of safety against liquefaction under the SSE was 1.47.

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3.0 SUMMARY OF WES REPORT OF 1978

3.1 Background

The NRC requested that WES review the foundation conditions at the LACBWR site and prepare a report (Ref. 3) specifically examining the earthquake safety of the pile foundation which supports the containment vessel.

3.2 Scope and Purpose of Report

The scope of WES's report included the following:

a. Review of Chapter 3, Soil Engineering Properties, in the DPC's Application for Operating License for the LaCrosse Boiling Water Reactor (Ref. 2), including portions of Appendix A, entitled "Field Exploration and Laboratory Tests," and associated design drawings.

b. Performance of a liquefaction analysis using the Seed-Idriss Simplified Procedure (Ref. 6), assuming peak ground surface accelerations of 0.12 g and 0.20 g.

c. Performance of a liquefaction analysis using Seed's empirical method (Ref. 7), assuming both the 0.12-g and 0.20-g earthquakes, and comparison with a "rule of thumb" based on the Japanese experience at Niigata in 1964 (Ref. 8).

3.3 Conclusions by WES

The liquefaction potential was evaluated for two earthquakes--an SSE with a peak ground acceleration of 0.12 g and an SEE with a peak ground acceleration of 0.20 g. Two methods were employed in the analysis-- the Seed-Idriss Simplified Procedure and an empirical procedure. Also, a Japanese "rule of thumb" based on blowcounts from standard penetration tests was used to predict liquefaction. The following were the conclusions:

a. Liquefaction was predicted between depths of 32 and 48 feet by Seed-Idriss calculations for 0.12-g ground acceleration.

b. Liquefaction was predicted between depths of 24 and 35 feet by the empirical procedure for 0.12-g ground acceleration.

c. Liquefaction was predicted below a depth of 25 feet by Seed-Idriss calculations for 0.20-g ground acceleration.

d. Liquefaction was predicted between depths of 25 and 60 feet and 85 and 105 feet by the empirical procedure for 0.20-g ground acceleration.

e. Japanese experience, based on the Niigata earhquake of 1964, also indicated liquefaction potential below a depth of 15 feet (for both cases of 0.12 g and 0.20 g).

f. If lateral support was lost at the depths indicated above, piles would be in danger of failure due to buckling.

3.4 Summary

Based on judgements concerning the density and strength data and on analyses presented in the WES report, the soils below the reactor at the LACBWR site were predicted to strain "badly" under an SSE which produces 0.12-g acceleration at the ground surface. The soils beneath the reactor vessel at the site were predicted to experience excessive strains and liquefaction under an SSE with a peak acceleration at the ground surface of 0.20-g. According to the WES report, because of limitations and the limited data available, it was concluded that the reactor vessel foundation was unsafe under the 0.20-g SSE, but no conclusion was reached on whether the reactor vessel foundation was safe under the 0.12-g SSE.

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4.0 BRIEF REVIEW AND DISCUSSION OF WES REPORT

The WES report, Liquefaction Analysis for LaCrosse Nuclear Power Station (Ref. 3), now a public document, was discussed for the first time at a meeting with NRC on January 9, 1979. DPC took exception to the contents of the WES report and requested that NRC arrange another meeting for discussion of the report after D&M had an opportunity to review it. As a result, a second meeting was held at NRC on February 9, 1979, during which D&M presented its review of the report to NRC. Dr. W. F. Marcuson, the principal auther of the WES report, was present at the meeting. The details of the February 9, 1979, meeting are presented in Ref. 4. The following review comments were expressed by D&M and DPC:

a. In general, WES adopted a very conservative approach in interpreting the available data.

b. WES postulated an earthquake of MM Intensity IX as a design SSE for the LACBWR site; this was considered unrealistic by D&M and DPC.

c. The report consistently reflected conservatism in selection of soil parameters, selection of cyclic shear stress ratio, and selection of stress reduction factor, which resulted in a cumulative underestimation of safety factors.

d. WES performed empirical analysis based on standard penetration results and compared the LACBWR site with sites which have experienced much higher seismic activity.

e. A Japanese "rule of thumb" developed after the experiences of the 1964 Niigata earthquake and based on standard penetration test results was applied to the LACBWR site; such a direct application was considered inappropriate by D&M.

In summary, D&M felt that the conservative approach taken by WES in each individual step of the analysis resulted in low factors of safety against liquefaction.

5.0 REEVALUATION OF DAMES & MOORE REPORT OF 1973

After the January 9, 1979, meeting with NRC, D&M reevaluated its 1973 report (Ref. 1) in light of the comments and concerns raised in the 1978 WES report (Ref. 3). In general, the D&M approach was found to be consistent with the state-of-the-art in 1973. The obvious limitation of the 1973 study was the lack of liquefaction test data on "undisturbed" samples. This limitation was indeed realized in the D&M analysis of 1973 and, therefore, a conservative approach was followed.

Two possible modifications to the D&M analysis of 1973 were considered:

a. Redrawing of the strength curves based on densities, rather than relative densities, in a manner similar to the procedure used in the WES report of 1978.

b. Selecting the design shear stress ratio corresponding to five equivalent cycles to represent more realistically the postulated design SSE.

Based on these modifications, factors of safety against liquefaction were recomputed and found to be essentially similar to those cited by D&M in 1973.

6.0 DAMES & MOORE RECOMMENDATIONS OF MARCH 1979

The 1973 data and the analyses indicated that the factors of safety against liquefaction under the design SSE were adequate at the LACBWR site. However, two basic issues needed to be addressed to further strengthen confidence in the results obtained--better definition of in situ densities at the LACBWR site, and development of continuous standard penetration test data at the site. Also, it was necessary to have better estimates of cyclic shear stresses that would result from the design SSE, and better estimates of shear strength data from test results on relatively undisturbed samples.

To address the above concerns, D&M recommended a testing and analysis program for the LACBWR site, consisting of a test boring program with a minimum of four borings, a limited laboratory testing program, and limited analyses. Details on the procedures and the actual program are discussed in Ref. 4.

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7.0 NRC/WES COMMENTS ON DAMES & MOORE RECOMMENDATIONS

The 1979 D&M recommendations (Ref. 4) were reviewed by the authors of the WES report and NRC staff and approved by NRC subject to WES comments. The following are the main points of the review as outlined in Ref. 5:

a. The program outlined by D&M is acceptable for determining the potential for liquefaction in the immediate vicinity of the containment building of the LACBWR plant.

b. Additional borings may be required near the turbine building and the cribhouse.

c. State-of-the-art techniques should be employed to obtain undisturbed samples in cohesionless soils.

d. Commercial transportation of samples should be avoided.

e. A sufficient number of cyclic triaxial tests should be performed to cover all the depths and confining pressures of interest and to obtain a good definition of strength at different confining pressures and at different significant stress cycles.

f. Analysès should be performed to cover a range of assumed peak ground acceleration levels between 0.12 g and 0.20 g, so that a threshold liquefaction resistance level can be estimated for the LACBWR plant site.

The remaining sections of this report describe the actual test boring program, the laboratory testing program, and the analyses performed to estimate a threshold liquefaction resistance level for the LACBWR plant site.

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8.0 TEST BORING PROGRAM

8.1 General

Five additional borings were drilled at the LACBWR site to obtain more complete data for verification of the earlier liquefaction analyses. Two borings were drilled on each side of the reactor building near 1973 borings DM-1 and DM-3, and a fifth hole was drilled near the cribhouse near boring DM-5. Approximate locations of the new borings (DM-7 through DM-11) are shown along with the earlier borings on Figure 1. Detailed descriptions of the soils that were encountered are presented on boring logs in the Appendix.

Three of the borings (DM-8, DM-10, and DM-11) provided standard penetration test (SPT) blow counts at 5-foot intervals throughout the depth of the holes; continuous blow counts had been precluded in previous borings by the use of several types of samplers in each hole. Borings DM-7* and DM-9 yielded relatively undisturbed samples suitable for density determinations and laboratory cyclic triaxial strength testing. The split-spoon samples from the SPT holes were used for field classification and laboratory confirmation of index properties.

8.2 Drilling and Sampling Procedures

Drilling operations were performed by Raymond International of Chicago, using a Mobile B-61 truck-mounted rotary wash drill rig. The rig was leveled before beginning each hole to ensure vertical drilling. Drilling to the specified sampling depths was done with a 4 1/8-inch tri-cone roller bit attached to A-size drill rods, with side discharge of drilling fluid to minimize disturbance to soil below the bit. Casing was advanced at intervals to keep the hole open as drilling progressed, and a thick drilling mud was mixed and maintained above the groundwater level in the hole at all times. When completed, each hole was grouted at no pressure with a thick cement slurry to prevent caving.

8.2.1 Standard Penetration Tests. The standard penetration tests (SPT) were performed at 5-foot intervals in borings DM-8, DM-10, and DM-11 to

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*In DM-7 samples were taken alternately every 5 feet by the Osterberg piston sampler and the SPT split-spoon.

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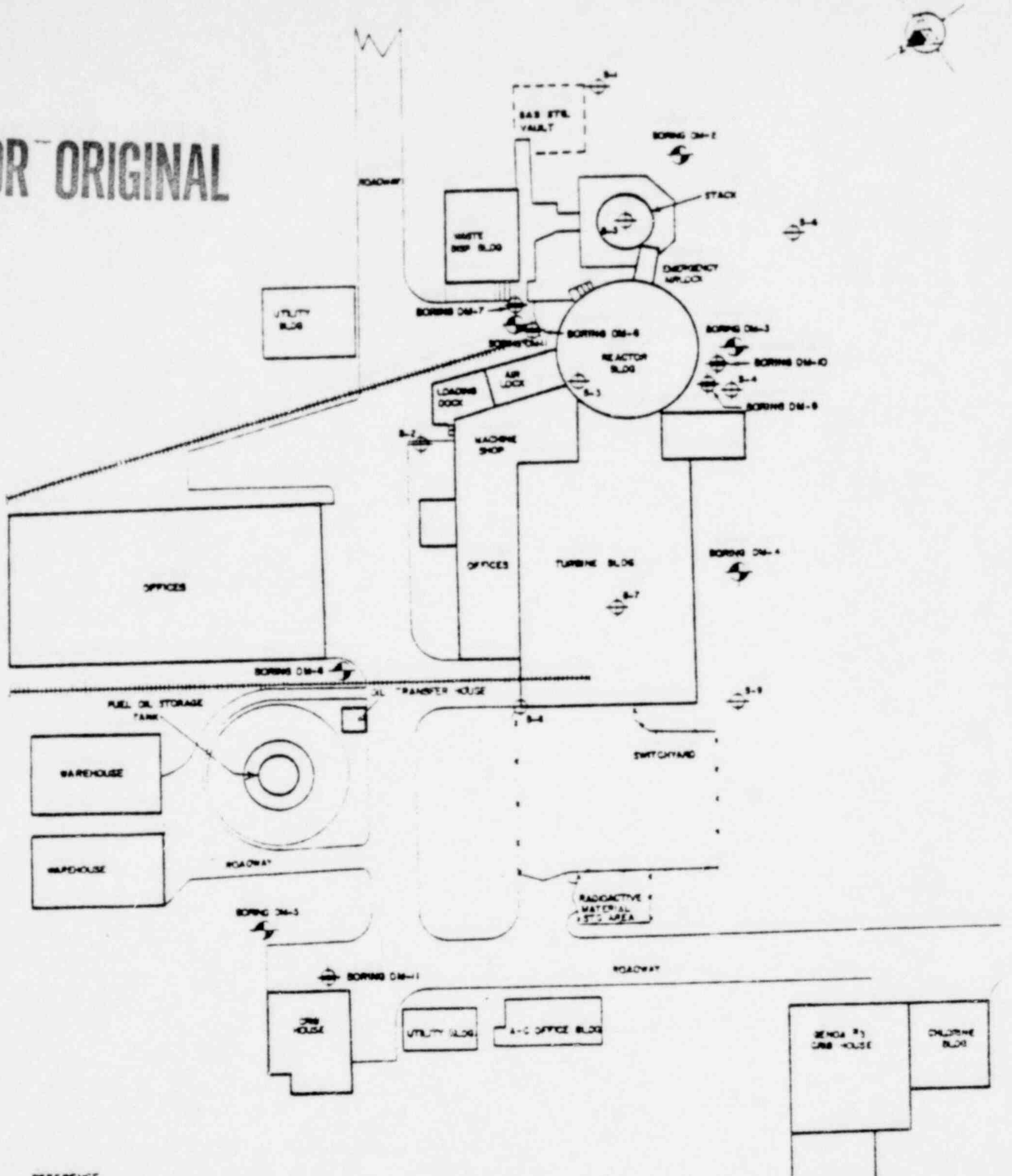

REFERENCE

MISSISSIPPI RIVER

PLOT PLAN

KEY:

TEST BORING FOR D&M INVESTIGATION IN 1973

TEST BORING BY RAYMOND INT'L IN JULY 1962

TEST BORING FOR D&M INVESTIGATION IN 1979

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provide blow-count values through all depths to be considered in the analysis, and to provide samples for field classification and laboratory verification of index properties. Sampling was done in accordance with ASTM D-1586-67 (Standard Penetration Test) specifications, using a calibrated 140-pound pin-held hammer dropping 30 inches. The pull rope used was old and flexible, wrapped two turns around the cathead, and was oiled frequently to minimize friction and approach as free a fall of the hammer as possible. The 2-inch split-spoon was driven 18 inches into the soil, and blow counts were recorded in 6-inch increments. The split-spoon was then slowly withdrawn and the disturbed sample was preserved for classification and testing after field identification. Figure 2 shows N-values plotted with depth for all borings; these values represent blow counts for the last 12 inches of each sample.

8.2.2 Undisturbed Sampling. Relatively undisturbed samples were obtained at 5-foot intervals in boring DM-9 and at 10-foot intervals in DM-7. Samples were taken in thinwall tubes by means of an Osterberg piston sampler. The tubes were coated with polyurethane to minimize frictional disturbance. Before each sampling operation, the piston sampler was cleaned and oiled and extended by hydraulic pressure applied by the rig to ascertain that grit would not hinder its even extension into the soil once it was lowered to sampling depth. When clean, the sampler was lowered to rest on the bottom of the hole, and the tube was extended 30 inches into the soil by even hydraulic pressure. The rig was chained down during the sampling to prevent uplift of the rig and uneven pressure application on the sampler.

The sampler was then slowly withdrawn from the hole, maintaining the mud level near the top of the hole. When the sampler cleared the top of the casing, a small amount of soil was removed from the bottom of the tube (and dimensions recorded) to permit insertion of a solid cap in the end of the tube. The purpose of the end cap was to prevent loss of sample material and moisture during removal from the sampler. The sample tube was then severed at the top of the sample with a pipe cutter to release any vacuum within the tube and minimize disturbance while disengaging the tube from the sampler. The tube was capped on top, while maintaining its vertical orientation, and carried by a D&M field engineer to the onsite laboratory for measurement and storage.

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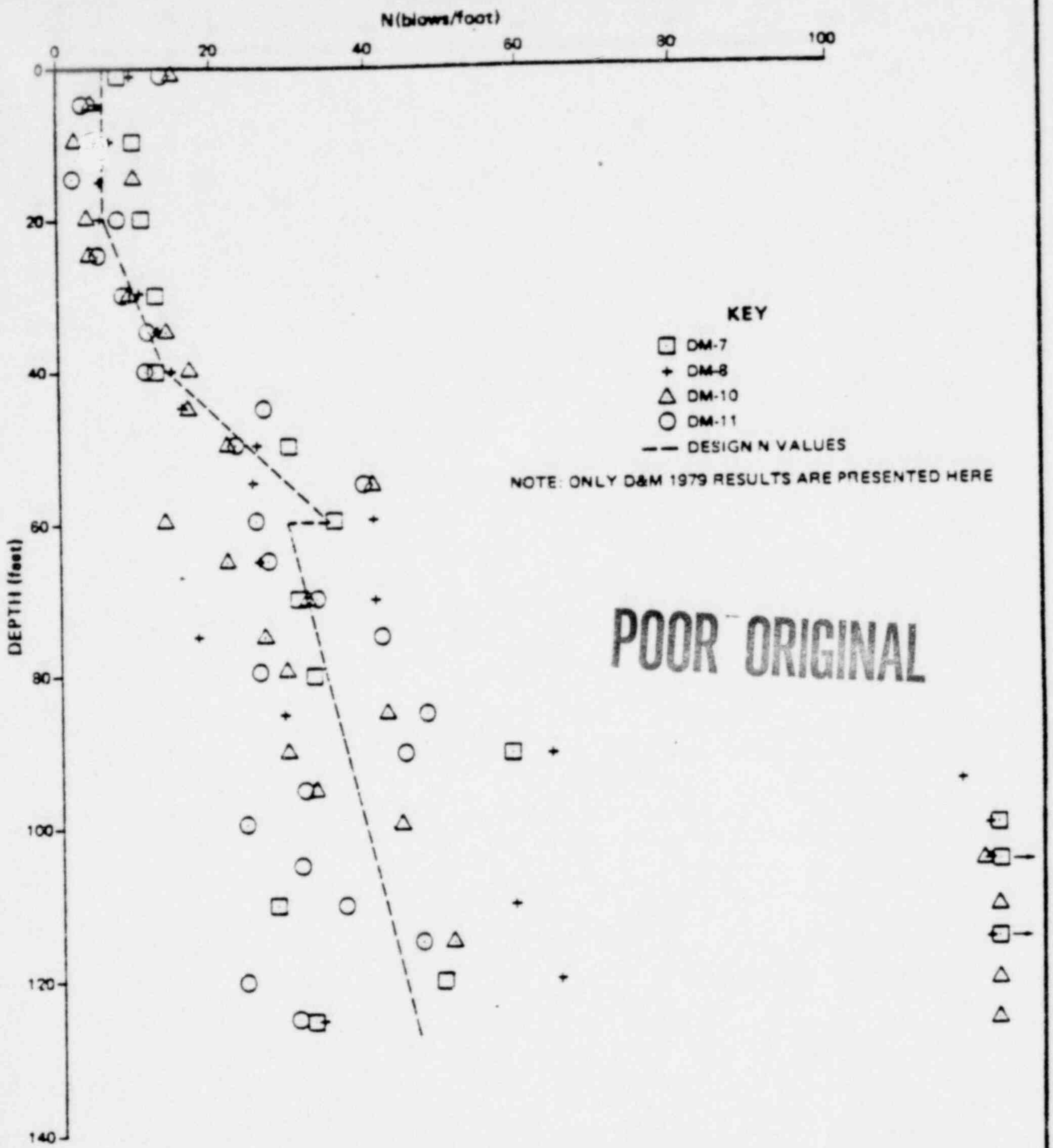

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VARIATION OF SPT N- VALUES WITH DEPTH

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FIGURE 2

8.3 Undisturbed Sample Handling

Field density measurements were made in an onsite temperature-controlled laboratory accessible only to the site security chief and D&M personnel. Upon arrival in the laboratory, a sample tube was immediately measured and weighed, using appropriate tare weights, to determine a field density. A small amount of soil was then removed from the top and bottom of the sample to determine moisture content. A drainage cap, consisting of two perforated metal disks separated by a rubber grommet, was installed in the bottom of each sample tube to prevent displacement of the soil as drainage occurred. The rubber grommet could be tightened or loosened by means of a wing nut. The sample was covered with a non-airtight cap and allowed to drain at least 24 hours in a vertical tube rack. It was anticipated that, after drainage of the free water, freezing of the remaining capillary moisture would create minimal, if any, disturbance of the structure of the sand samples. This freezing technique currently is considered the best means of preserving the structure of clean, loose sands below the water table for transport and testing (Ref. 9).

After draining, the samples were placed in vertical racks in 55-gallon drums and packed with dry ice surrounded by shredded insulation. The samples were allowed several hours to freeze, checked by a short length of tube filled with water (which froze completely within a half hour). The drums were then transported to a commercial cold-storage plant in LaCrosse, where they were stored at -20°F for the duration of the field operations. Upon completion of the drilling program, the samples were repacked in dry ice and insulation and driven to Chicago for laboratory testing, where they were unloaded and stored in a freezer maintained at about -10°F. The sample transport was performed by a D&M field engineer to ensure careful handling.

9.0 LABORATORY TESTING PROGRAM

9.1 General

The purpose of the testing program was to provide additional strength data from undisturbed samples for the liquefaction analysis, and to make a limited number of confirmations of index properties. In addition to 15 stress-controlled cyclic triaxial tests, testing included specific gravity determinations, particle size analyses, minimum and maximum density determinations, and measurements of dry density of the undisturbed samples.

9.2 Specific Gravity

Determinations of specific gravity of sands at the site were made in the D&M laboratory in accordance wth ASTM D-854-58 (Specific Gravity of Soils). Tests of four samples from depths of 31 to 47 feet yielded specific gravity figures ranging from 2.60 to 2.65. These results correlate with expected values for such soils and with 1973 results.

9.3 Particle Size Analyses

Particle size analyses were performed on 25 samples from boring DM-10 by a D&M laboratory according to ASTM D-422-63 (Particle Size Analysis of Soils). Results are shown as ranges for the sandy and gravelly soils respectively, on Figures 3 and 4. The coefficient of uniformity, C_u (Table 1), or ratio of D_{60}* to D_{10}, provides a useful comparison of grain-size distribution at various depths and can be used in relative density calculations.

9.4 Minimum and Maximum Densities

Minimum and maximum densities for a composite of samples between 31 and 47 feet were determined in the laboratory by using equipment and methods similar to those specified by ASTM D-2049-69 (Relative Density of Cohesionless Soils). These tests produced an average minimum dry density of 97.2 pounds per cubic foot (pcf) and an average maximum of 114.3 pcf. Relative density, D_r, could then be calculated by comparing these densities with measured in situ densities.

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*D_{60} refers to the grain size which is coarser than 60 percent of the sample by weight. D_{10} is defined similarly as coarser than 10 percent of the sample.

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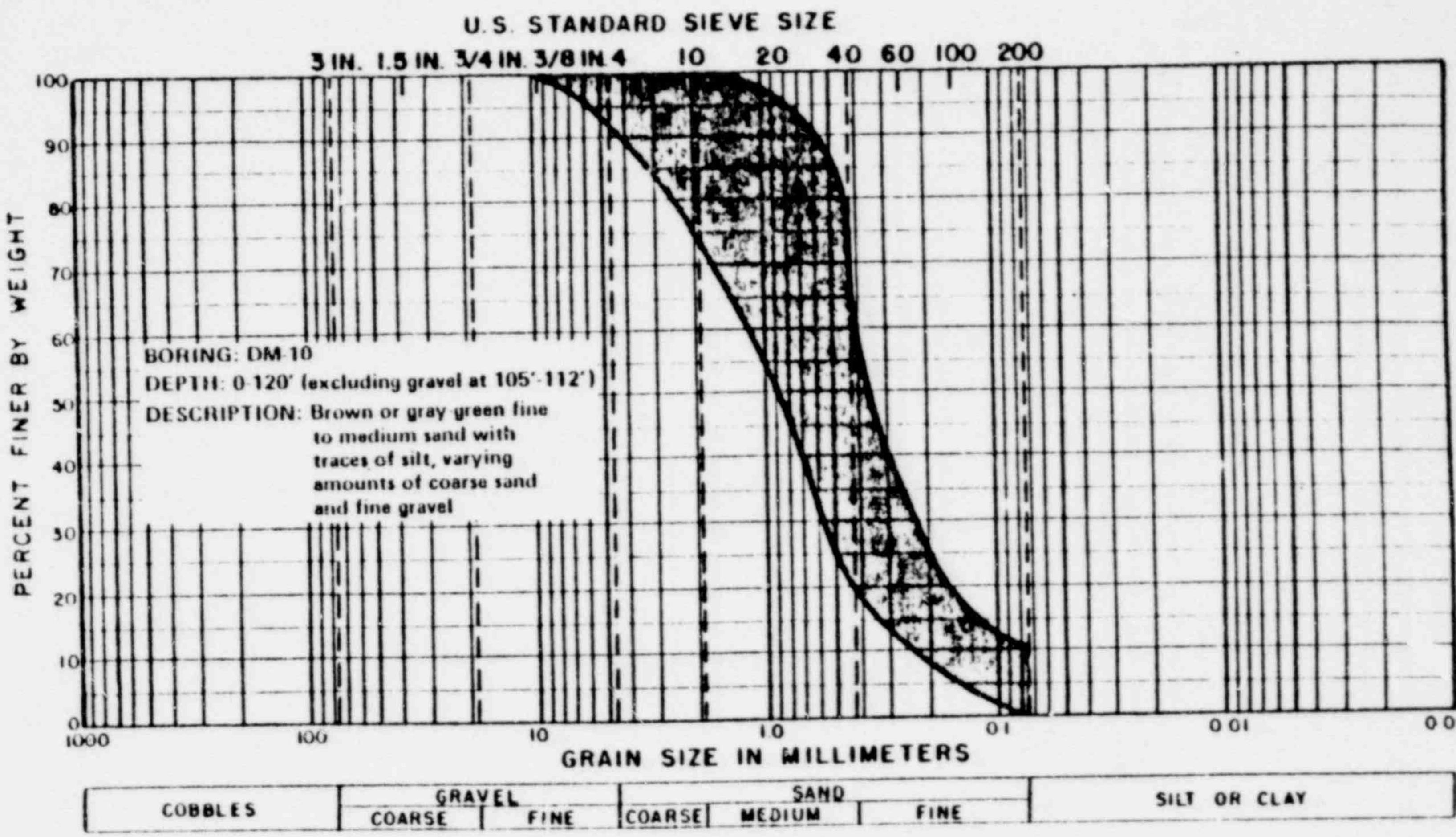

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PARTICLE SIZE ANALYSIS
RANGE FOR SANDS ENCOUNTERED AT SITE

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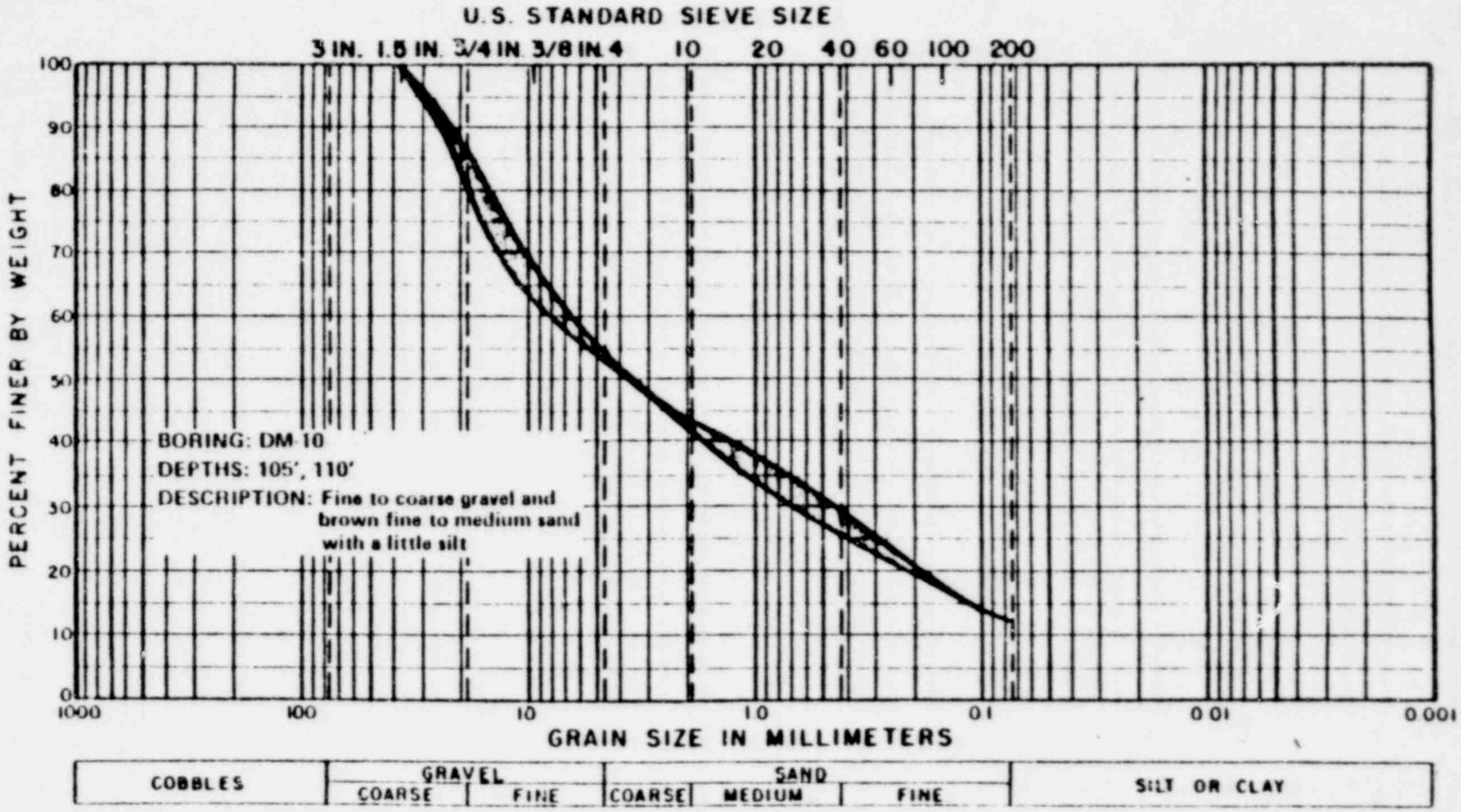
FIGURE 7



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PARTICLE SIZE ANALYSIS
RANGE FOR GRAVELS ENCOUNTERED AT SITE

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TABLE 1

PARTICLE SIZE CHARACTERISTICS

Depth (ft)	D_{10}* (mm)	D_{50} (mm)	D_{60} (mm)	C_u $=D_{60}/D_{10}$
10	0.12	0.40	0.42	3.5
20	0.20	0.42	0.43	2.2
30	0.15	0.44	0.51	3.4
40	0.15	0.41	0.49	3.3
50	0.16	0.39	0.42	2.6
60	0.16	0.54	0.67	4.2
70	0.18	0.39	0.43	2.4
80	0.18	0.41	0.43	2.4
90	0.20	0.66	0.72	3.6
100	0.19	0.62	0.71	3.7

*D_{60}.

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Relative densities were also calculated by means of an expression developed by Meyerhof in 1957 (Ref. 10) which relates relative density to blow counts and overburden stress at a particular depth. These values were used in the Japanese analysis (Ref. 8) and in the shear modulus calculations for the one-dimensional analysis.

Another check of relative densities was made by means of the Marcuson/Bieganousky (Ref. 11) expression involving uniformity coefficients, blow counts, and overburden stresses. These values compared satisfactorily with those used in the analysis.

9.5 Dry Density of Undisturbed Samples

As described previously, field densities were calculated by measuring and weighing the undisturbed samples immediately on coming out of the boring. We believe that these values are fairly accurate because the measurements were made for the entire sample and as soon as possible after sampling. Dry density values were derived after field moisture contents were taken from each end of a sample.

Densities were also measured on the frozen samples in the laboratory. Sample tubes were cut into smaller sections and accurately measured, and the weight of soil solids was determined by drying the sample. Dry densities were calculated by dividing this weight of solids by the frozen volume.

9.6 Cyclic Triaxial Tests

Fifteen stress-controlled cyclic triaxial tests were performed on undisturbed samples in the laboratory of the University of Illinois at Chicago Circle. Samples for testing were chosen in the depth ranges of 10 to 20 feet (hydraulic fill), 30 to 40 feet, 40 to 50 feet, and 80 to 90 feet.

9.6.1 Sample Preparation. In preparation for testing, sample tubes were removed from the freezer and cut into sections with a tube cutter to produce test specimens. An inch of possibly disturbed material was wasted from the bottom of each tube. A vertical band-saw was then used to split one side of the tube, which allowed the frozen sample to be extruded vertically into a split brass cyclinder for trimming and transporting. The specimen was placed in the triaxial cell in a membrane with filter paper at top and bottom, and a small vacuum minus 5 inches of mercury was applied

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while thawing the specimen. The sample was then consolidated under pressure corresponding to slightly above the in situ effective confining pressure. Specimen dimensions were recorded before and after thawing and after consolidation.

9.6.2 Testing. Cyclic triaxial testing of the specimens was performed according to procedures outlined by Silver (Ref. 12). Samples were placed in a triaxial cell capable of being loaded with a periodic cyclic stress of constant amplitude. Cyclic loading was begun and continued until double amplitude strains exceeded 10 percent, axial compressive or extensive strains exceeded 20 percent, or the predetermined number of load cycles was achieved.

These test results were evaluated with respect to the magnitude of cyclic axial stress and the number of cycles required to produce double amplitude, compressive, or extensive strains of 5 percent and 10 percent. Also recorded was the first cycle at which the induced excess pore pressure became equal to the cell pressure, which is referred to as initial liquefaction. Ranges of stress ratios at failure were selected to obtain relationships between stress ratios and number of cycles required to cause liquefaction.

Using the dimensions of the frozen specimen and the weight of the solid particles, which was determined by drying and weighing the sand particles after completion of the triaxial test, three density calculations were made for the tested specimens. This density was called the frozen density. After the sample was allowed to thaw for 2 or 3 hours in the laboratory under vacuum confinement, new diameter values were measured with the Pi tape at three locations on the specimen. The change in height in the vertical dial gage was noted. A new volume calculation for the specimen was made with the new height and diameter. By dividing the new volume into the weight of the solid particles of the specimen, a second density was determined. This density was called the thawed density. The cell was then assembled around the specimen and the specimen was saturated and consolidated. Vertical dial readings and volume change readings were made and used to calculate the consolidated volume of the specimen. By dividing the dry weight of the solids by the consolidated volume, a third density, the consolidated density, was determined. A summary of densities and the triaxial test results is given in Table 2, and the variation of dry density with depth is shown in Figure 5.

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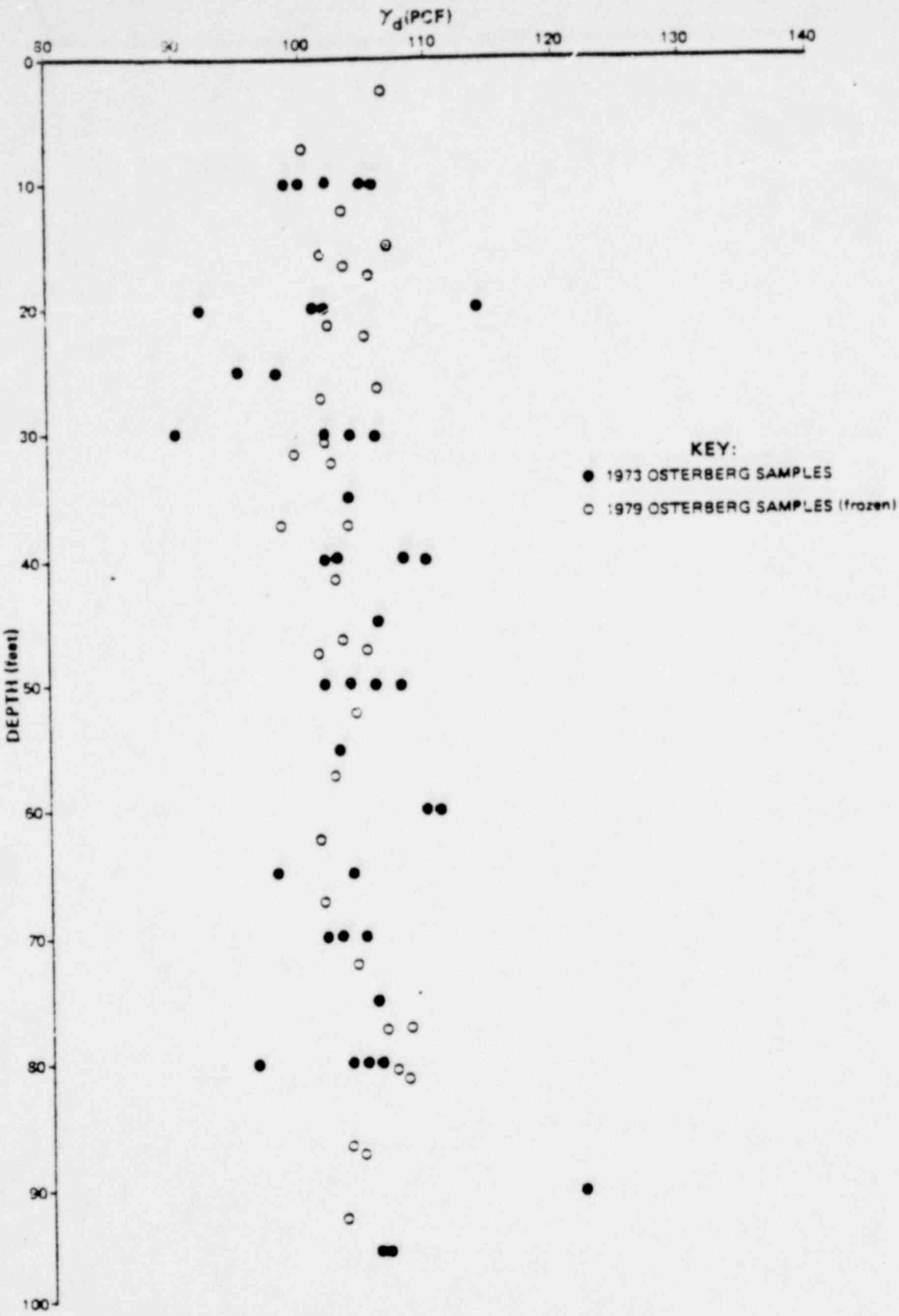
TABLE 2

SUMMARY OF CYCLIC TRIAXIAL TEST RESULTS*

Test Number	Sample/ Specimen Number	Depth (ft)	Dry Density (pcf) Frozen Condition	Thawed Condition	Thawed and Consolidated Condition	Effective Confining Pressure $\bar{\sigma}_c$ (psf)	Stress Ratio $\tau/\bar{\sigma}_c$	Saturation Details 'B' Parameter	Number of Cycles to Liquefaction N_i (Initial)	N_5 (5% DA Strain)	N_{10} (10% DA Strain)	Remarks
	5/2	21.5	102.2	104.6	105.4	2,000	0.22	0.97	5	5	5	
1	4/3	16.0	101.8	103.5	104.1	2,000	0.18	0.98	11	10	11	
	4/2	16.5	103.9	105.4	106.4	2,000	0.32	0.97	3	2	3	
	7/2	31.5	99.7	101.0	101.4	2,500	0.20	0.86	10	10	14	Low "B" value
2	7/3	31.0	102.0	103.7	104.2	2,500	0.28	0.99	10	12	16	
	8/1	37.0	104.1	105.4	105.7	2,500	0.39	0.96	6	11	25	
	8/3	36.5	102.9	104.3	104.8	2,500	0.12	0.99	> 1,000	> 1,000	> 1,000	Did not liquefy
	10/1	47.0	101.6	103.1	103.7	4,000	0.32	0.95	4	3	5	
	10/2	46.5	103.0	104.7	105.4	4,000	0.23	0.97	5	4	7	
3	9/2	41.5	103.0	104.8	106.1	4,000	0.43	0.96	5	3	6	
	10/3	46.0	103.5	104.8	105.5	4,000	0.18	0.99	8	8	11	
	11/2	52.0	103.0	104.5	105.1	4,000	0.13	0.98	84	86	92	
	18/1	87.0	104.7	105.8	107.8	8,000	0.35	1.0	7	8	16	
4	18/2	86.5	103.6	104.6	105.6	8,000	0.26	0.96	14	16	18	
	19/1	92.0	103.2	104.8	106.0	8,000	0.45	0.99	3	2	3	

*All tests were performed on soil type SP from boring no. DM-9.



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VARIATION OF DRY DENSITY WITH DEPTH

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10.0 LIQUEFACTION ANALYSES

10.1 General

The liquefaction analyses performed for the LACBWR site in the past were based on test borings and laboratory data from the D&M investigations of 1973 (Ref. 1). As mentioned in earlier sections of this report, the findings of the 1973 studies were evaluated by NRC in 1978 under its Safety Evaluation Program, and several questions were raised regarding the factors of safety under the design SSE. D&M also reevaluated its earlier findings as a result of the NRC review using current state-of-the-art methods. Although D&M concluded that the factors of safety against liquefaction under the design SSE remained unchanged, there was agreement among NRC, WES, D&M, and DPC in the following:

a. "Undisturbed" sampling in the cohesionless soils was necessary.

b. In situ dry densities must be estimated with greater accuracy using "undisturbed" samples.

c. There was a need for development of continuous standard penetration "N" values under carefully controlled conditions.

d. Cyclic shear strength parameters of the liquefiable soils had to be obtained by performing cyclic triaxial tests on "undisturbed" samples.

e. Estimates of the cyclic shear stresses resulting from the design SSE must be made by performing a one-dimensional wave propagation analysis.

f. The seismicity of the LACBWR plant site and the potential for liquefaction under an acceleration level which realistically represents the seismicity of the site must be analyzed.

With consideration of these requirements, a limited but carefully controlled field and laboratory investigations program was undertaken. Using the data developed in these investigations, detailed liquefaction analyses were performed.

10.2 Liquefaction Potential

There are two basic approaches for evaluating the liquefaction potential of a deposit of saturated sand when it is subjected to earthquake loading. The first approach uses the information available on the performance of various sand deposits during past earthquakes. This approach is essentially empirical, and the response of soil to earthquake loading is

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not evaluated by any direct means. Simplified methods of analysis, with known limitations, have been proposed by various investigators. Also, a large number of factors that significantly affect the liquefaction characteristics of a given sand have been recognized and may be studied in detail to confirm the conclusions of such an analysis.

In the second approach, stress conditions in the field are evaluated by using an analytical technique, such as the one-dimensional wave propagation analysis. Laboratory investigations are conducted to determine the cyclic shear stresses required to cause liquefaction at various depths. At a given depth, a factor of safety against liquefaction can be evaluated by dividing the cyclic shear stress required to cause liquefaction by the cyclic shear stress induced during the design earthquake.

Methods based on these two approaches were used to assess the liquefaction potential of the granular soils at the LACBWR site.

10.3 Evaluation of Liquefaction Potential, Approach 1

10.3.1 Simplified Procedure (Procedure 1). In the first approach, the procedure recommended by Seed (Ref. 7) was used to estimate the cyclic shear stress required to cause liquefaction. The cyclic shear stress induced during shaking was computed by the Seed and Idriss Simplified Procedure (Ref. 6). The following steps are used in Procedure 1:

a. Convert the "N"* values from the Standard Penetration Tests to N_1 values (N_1 is the penetration resistance, corrected to an effective overburden pressure of 1 ton/ft^2) using the relationship:

$$N_1 = C_N (N)$$

where:

$$C_N = 1 - 1.25 \log \bar{\sigma}_c/\bar{\sigma}_1$$

$\bar{\sigma}_c$ = effective overburden pressure (tons/ft^2)

$\bar{\sigma}_1$ = a constant equal to 1 ton/ft^2.

*N = number of blows required to advance a standard split-spoon 12 inches into the ground, when driven by a hammer weighing 140 pounds dropping a distance of 30 inches.

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b. Based on a collection of data from actual field performance and a few additional site studies, the lower bounds for the cyclic shear stress ratios that cause liquefaction in the field and which correspond to different N_1 values and magnitudes of earthquakes have been established (Ref. 7). Using this relationship, the N_1 values can then be converted to the cyclic shear stress ratio, $\tau/\bar{\sigma}_c$, required to cause liquefaction for the design earthquake.

c. Compute the cyclic shear stress ratio at any depth in the ground that is induced by the design earthquake (Ref. 6) using the relationship:

$$\tau_{av}/\bar{\sigma}_c = 0.65\ (a_{max}/g)\ (\sigma_o/\bar{\sigma}_c)\ r_d$$

where:

$\bar{\sigma}_c$ = effective overburden pressure on sand layer

a_{max} = maximum acceleration at the ground surface (ft/sec^2)

σ_o = total initial overburden pressure on sand layer under consideration (tons/ft^2)

r_d = a stress reduction factor varying from a value of 1.0 at the ground surface to a value of 0.9 at a depth of about 30 feet.

τ_{av} = average cyclic shear stress in the sand layer under consideration (tons/ft^2)

g = acceleration due to gravity (ft/sec^2).

d. The lower bound cyclic shear strength values that are obtained from Step "b" can then be compared with the average cyclic shear stresses obtained in Step "c," and the liquefaction potential at various depths can be evaluated.

SPT(N) values from recent D&M investigations were plotted as a function of depth (Figure 2). Average design N values were chosen for different depths and were converted to corrected blow counts (N_1). Relative boundaries between "liquefaction" and "no liquefaction" conditions for

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various magnitudes of earthquakes corresponding to ground surface acceleration levels between 0.10 g and 0.20 g were drawn using the data and principles presented by Seed (Ref. 7). Figures 6 through 11 show such boundaries on which are plotted LACBWR plant site data of N_1 values corresponding to certain cyclic shear stress ratios induced by the respective earthquakes of different magnitudes. The SPT(N) values, the corresponding N_1 values used to compute the strength, the cyclic shear strengths, and the cyclic shear stresses were computed using a simplified procedure, and the resulting factors of safety at different depths for various acceleration levels are presented in Table 3. The stresses and strengths are presented as functions of depth for the six acceleraion levels in Figures 12 through 17.

Data in Figures 12 through 17 and Table 3 shown that no liquefaction is suggested for acceleration levels less than or equal to 0.12 g at any depth. As the acceleration level increases from 0.14 g to 0.20 g, the depths susceptible to liquefaction increase from 20 feet to 30 feet to 10 feet to 40 feet.

10.3.2 <u>Japanese Procedure (Procedure 2)</u>. Another procedure being used in Japan (Ref. 8) also falls under the general category of Approach 1. The procedure for computing the cyclic shear stresses is the same as described in Step "c" above using Seed and Idriss (Ref. 6) simplifications. The estimation of the cyclic shear strength is as follows:

a. Estimate the relative density of the liquefiable soil using the relation developed by Meyerhof (Ref. 10) based on laboratory tests performed by Gibbs and Holtz (Ref. 12):

$$D_r^* = 21\sqrt{N/(\bar{\sigma}_v + 0.7)}$$

where:

D_r^* = estimated relative density

N = blow count from SPT

$\bar{\sigma}_v$ = effective overburden pressure at the depth of interest (kg/cm^2).

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TABLE 3

SUMMARY OF LIQUEFACTION ANALYSIS
APPROACH 1, PROCEDURE 1

Depth (ft)	Design N/N_1	Average Cyclic Shear Stresses, Cyclic Shear Strengths, and Factors of Safety for Various Accelerations*																	
		a_{max} = 0.10 g			a_{max} = 0.12 g			a_{max} = 0.14 g			a_{max} = 0.16 g			a_{max} = 0.18 g			a_{max} = 0.20 g		
		τ_{av}	τ	FS	τ_{av}	τ	FS	τ_{av}	τ	FS	τ_{av}	τ	FS	τ_{av}	τ	FS	τ_{av}	τ	FS
10	6/8	73	138	1.69	88	138	1.57	102	127	1.23	118	127	1.08	132	127	0.96	147	127	0.86
20	6/6	145	167	1.15	174	174	1.00	203	148	0.73	232	148	0.64	261	148	0.57	290	148	0.51
30	10/9	213	317	1.49	255	317	1.24	298	292	0.98	341	292	0.86	383	292	0.76	426	292	0.69
40	14/11	270	525	1.94	324	494	1.52	378	494	1.31	432	463	1.07	486	463	0.95	540	463	0.86
50	25/17	301	1,005	3.34	361	968	2.68	421	894	2.12	482	894	1.85	542	856	1.58	602	856	1.42
60	36/20	328	1,371	4.18	394	1,327	3.38	459	1,283	2.80	525	1,239	2.36	590	1,239	2.10	656	1,239	1.89
70	32/16	343	1,256	3.66	411	1,206	2.93	480	1,155	2.41	548	1,105	2.02	617	1,105	1.79	685	1,105	1.61
80	35/15	363	1,317	3.63	435	1,317	3.03	508	1,260	2.48	581	1,203	2.07	653	1,203	1.84	726	1,203	1.66
90	37/14	389	1,414	3.63	467	1,349	2.89	545	1,285	2.36	622	1,285	2.07	700	1,285	1.84	778	1,285	1.65
100	40/12	409	1,349	3.30	491	1,278	2.60	573	1,207	2.11	655	1,207	1.84	737	1,207	1.64	818	1,207	1.48

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*τ_{av} = average cyclic shear stress from Seed & Idriss Simplified Procedure (Ref. 6).

τ = cyclic shear strength based on corrected blow counts and recorded cyclic behavior during earthquakes.

Factor of safety (FS) = (cyclic shear strength) ÷ (average cyclic shear stress).

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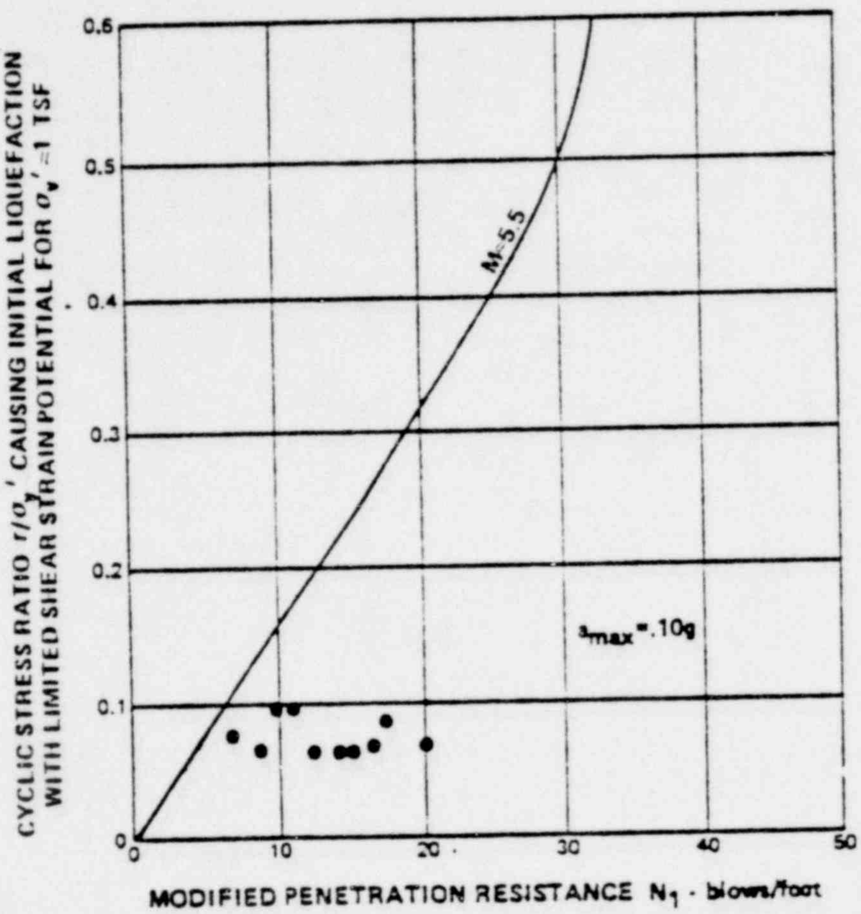

CORRELATION BETWEEN FIELD LIQUEFACTION BEHAVIOR
OF SANDS FOR LEVEL GROUND CONDITIONS AND PENETRATION RESISTANCE

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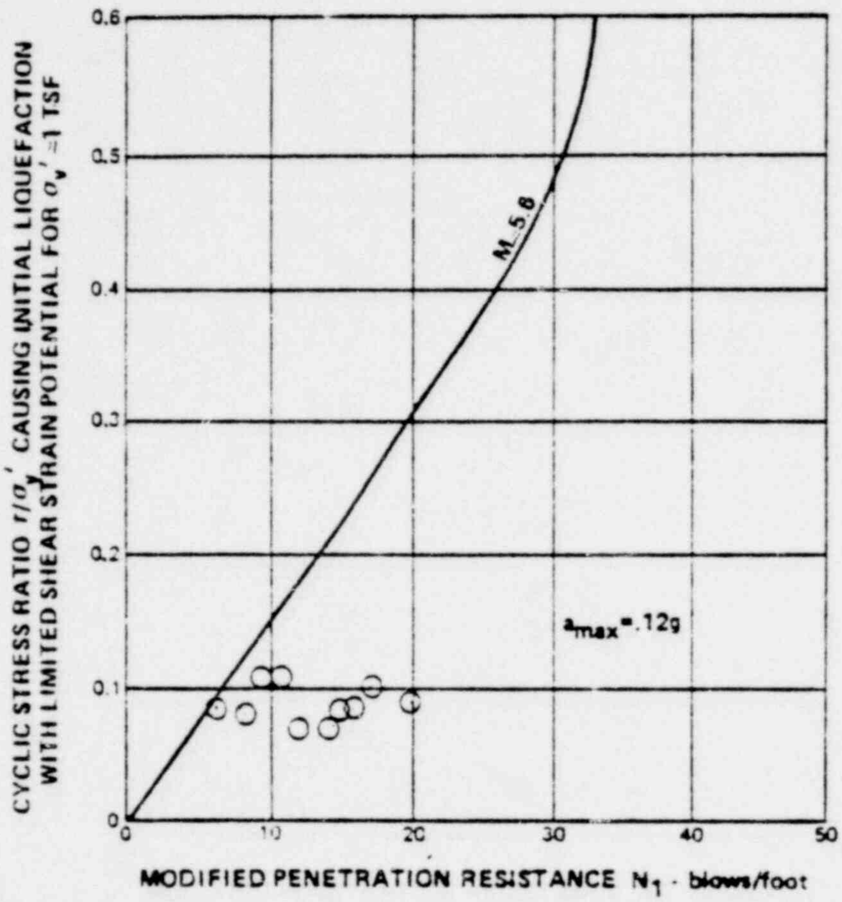

CORRELATION BETWEEN FIELD LIQUEFACTION BEHAVIOR
OF SANDS FOR LEVEL GROUND CONDITIONS AND PENETRATION RESISTANCE

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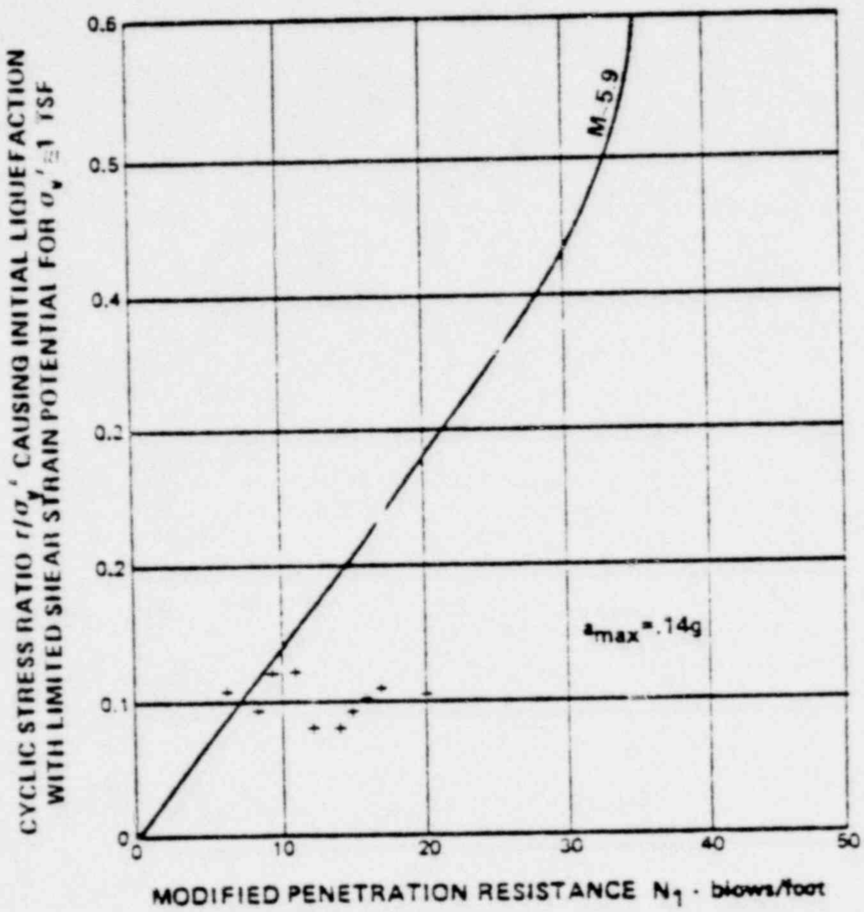
FIGURE



CORRELATION BETWEEN FIELD LIQUEFACTION BEHAVIOR
OF SANDS FOR LEVEL GROUND CONDITIONS AND PENETRATION RESISTANCE

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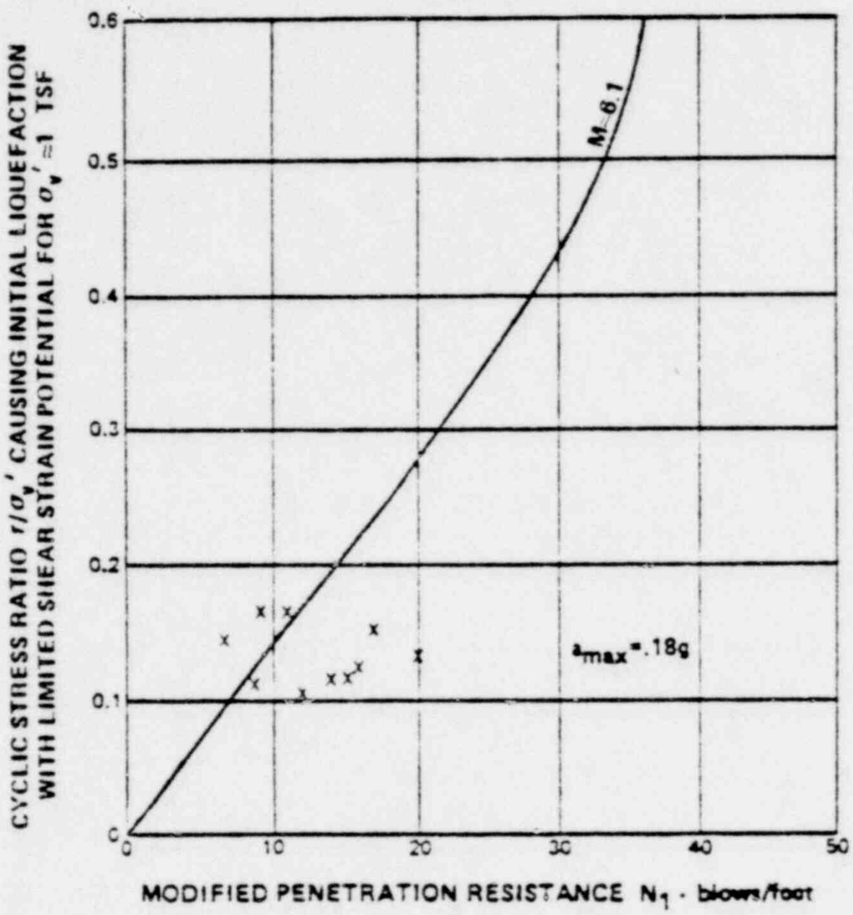
FIGURE



CORRELATION BETWEEN FIELD LIQUEFACTION BEHAVIOR
OF SANDS FOR LEVEL GROUND CONDITIONS AND PENETRATION RESISTANCE

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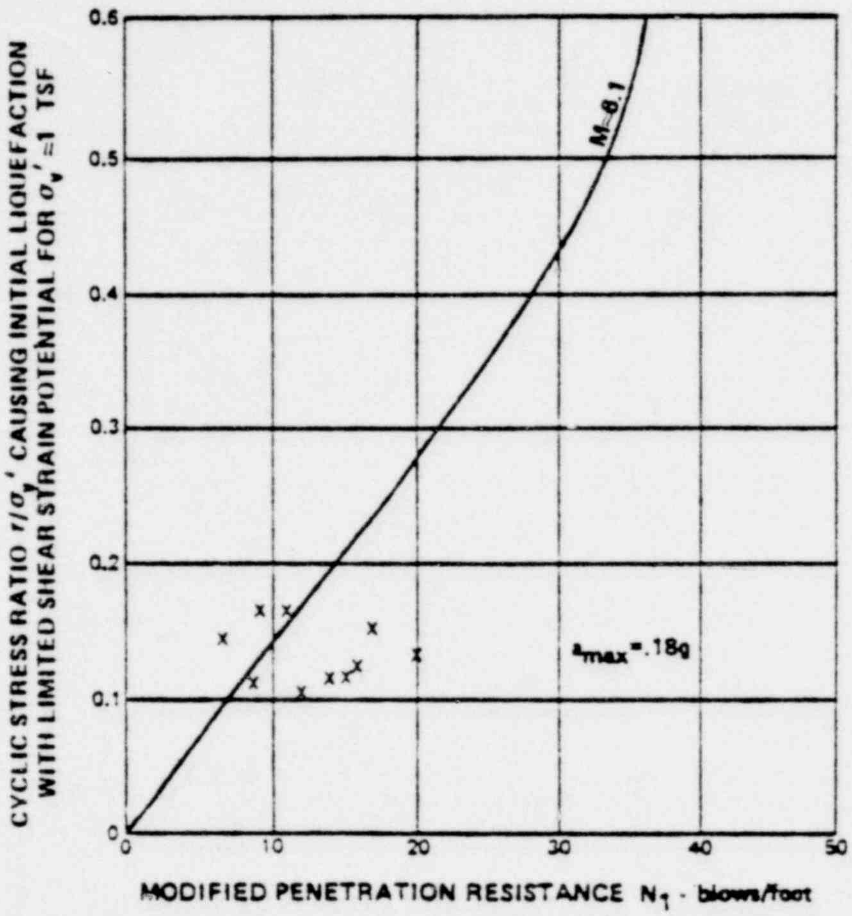
FIGURE 10



CORRELATION BETWEEN FIELD LIQUEFACTION BEHAVIOR OF SANDS FOR LEVEL GROUND CONDITIONS AND PENETRATION RESISTANCE

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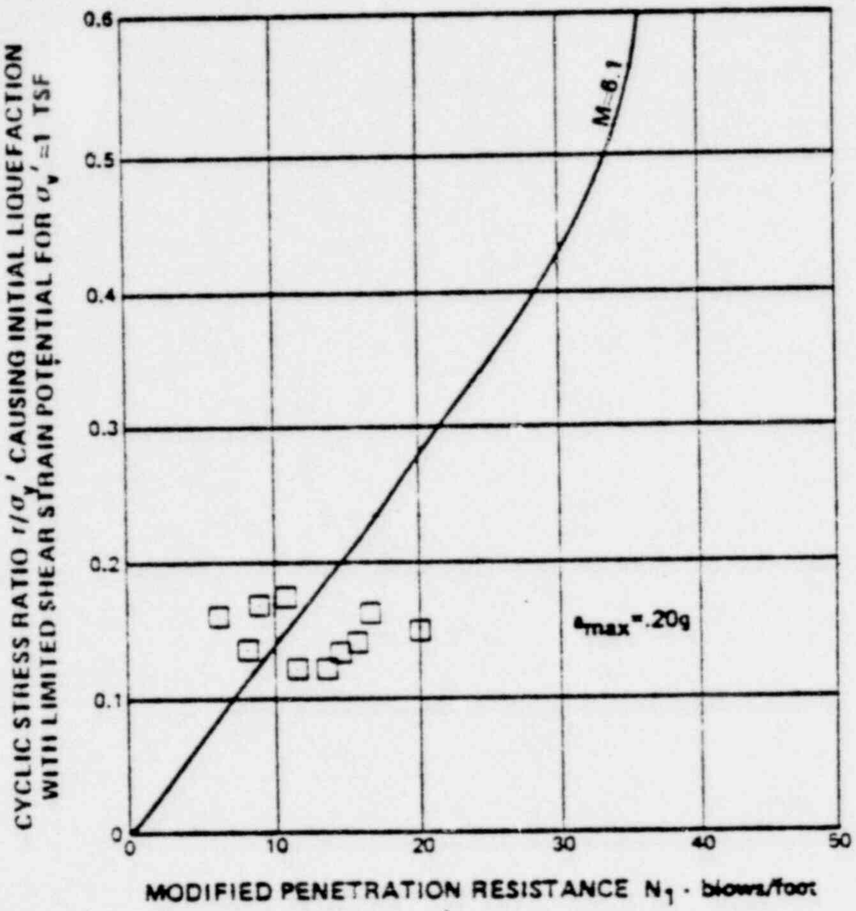
FIGURE 10



CORRELATION BETWEEN FIELD LIQUEFACTION BEHAVIOR
OF SANDS FOR LEVEL GROUND CONDITIONS AND PENETRATION RESISTANCE

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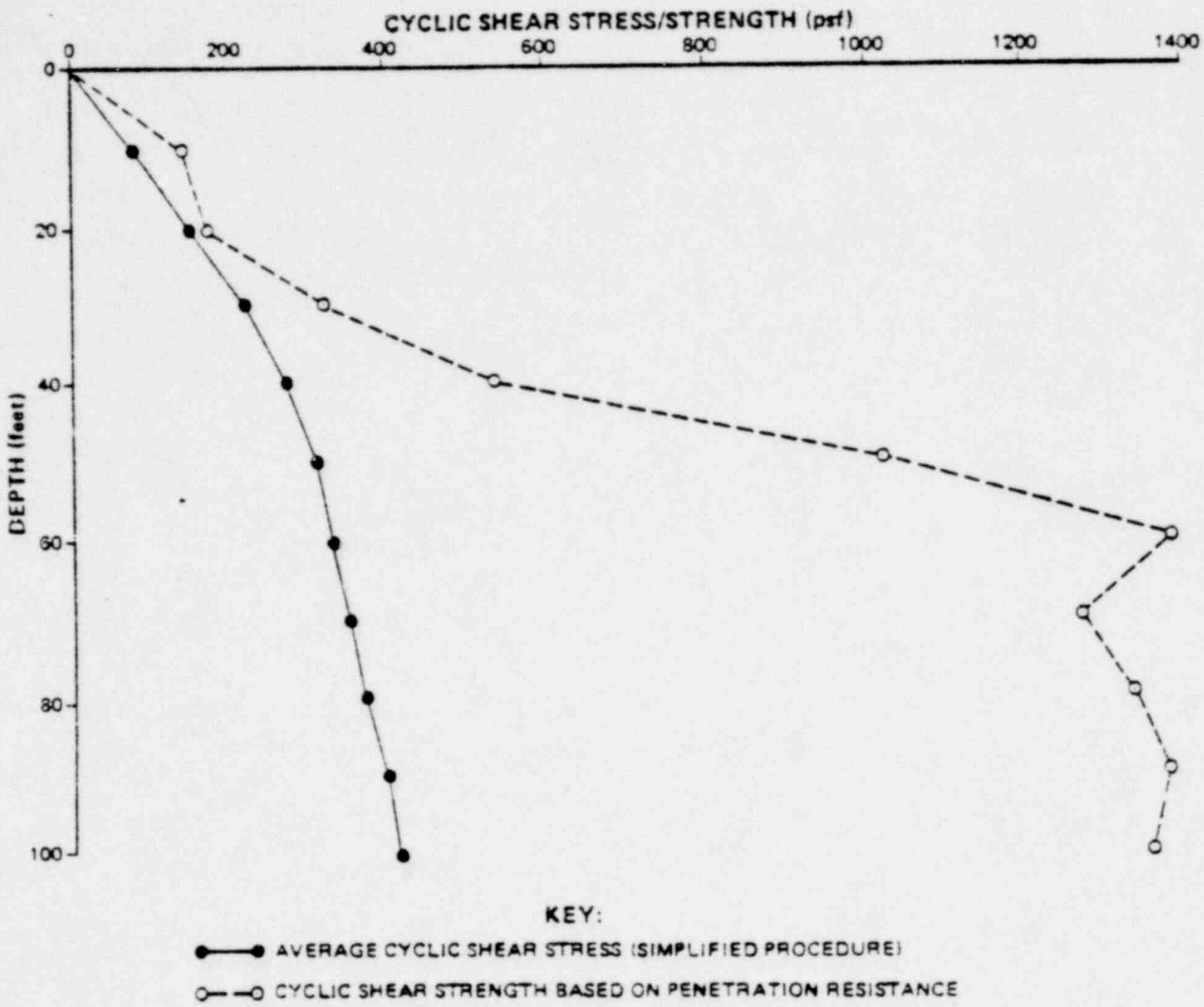
FIGURE 11



COMPARISON OF CYCLIC SHEAR STRENGTH
APPROACH 1 PROCEDURE 1 (a_{max}=0.10g)

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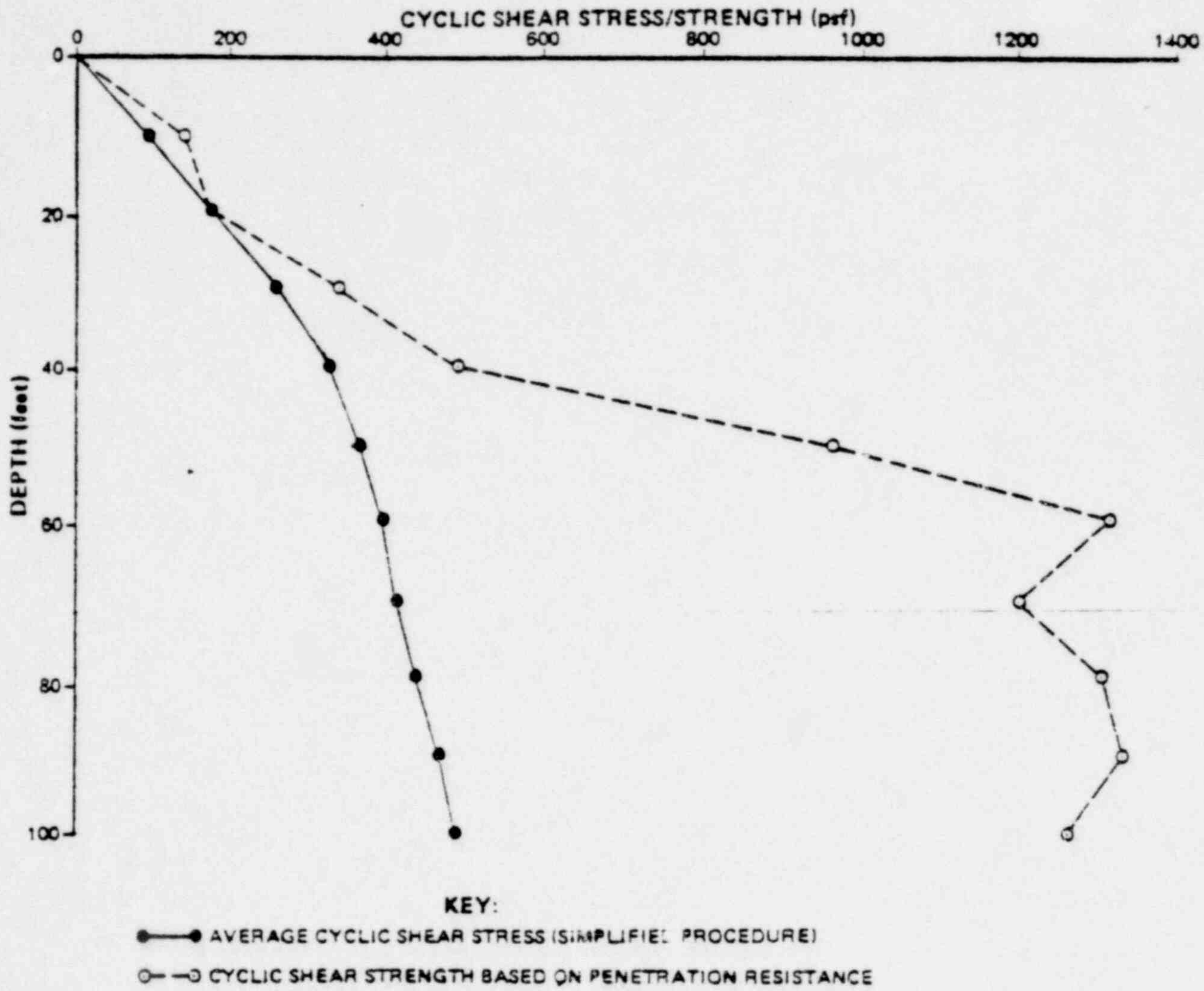
FIGURE 12



COMPARISON OF CYCLIC SHEAR STRESS AND STRENGTH
APPROACH 1 PROCEDURE 1 (a_{max}=0.12g)

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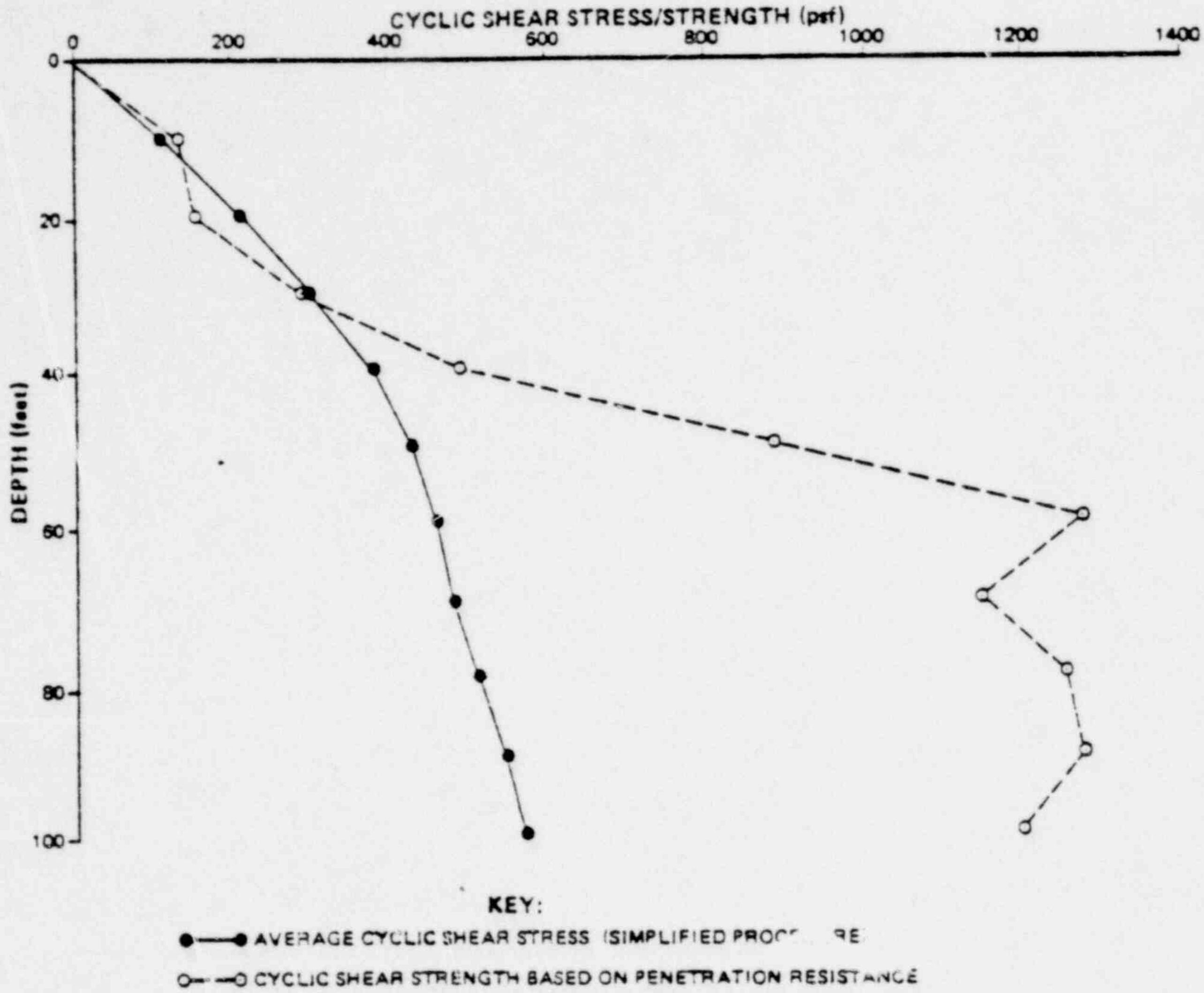
FIGURE 13



COMPARISON OF CYCLIC SHEAR STRESS AND STRENGTH
APPROACH 1 PROCEDURE 1 (a_{max}=0.14g)

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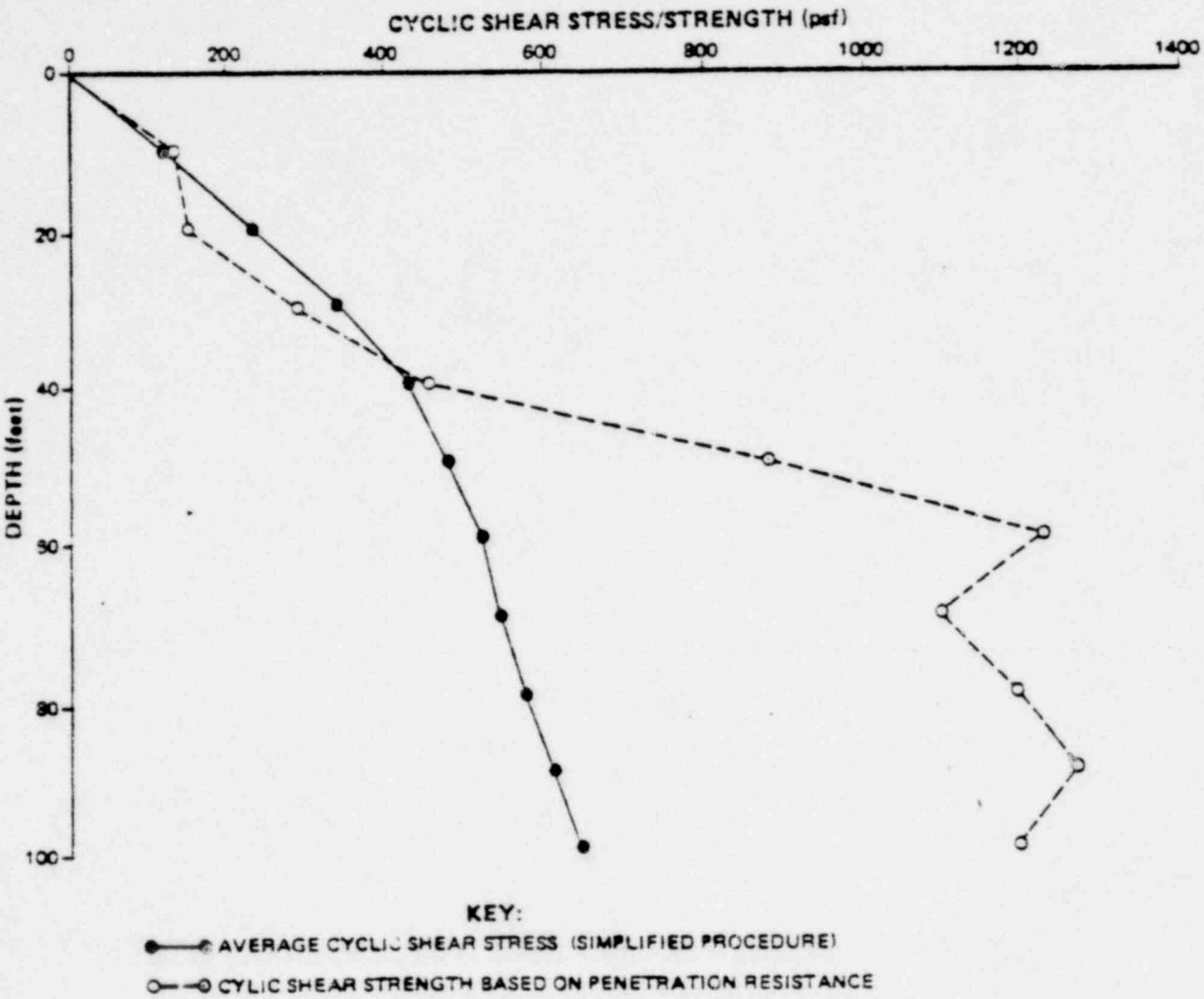
FIGURE 14



COMPARISON OF CYCLIC SHEAR STRESS AND STRENGTH
APPROACH 1 PROCEDURE 1 (a_{max}=0.16g)

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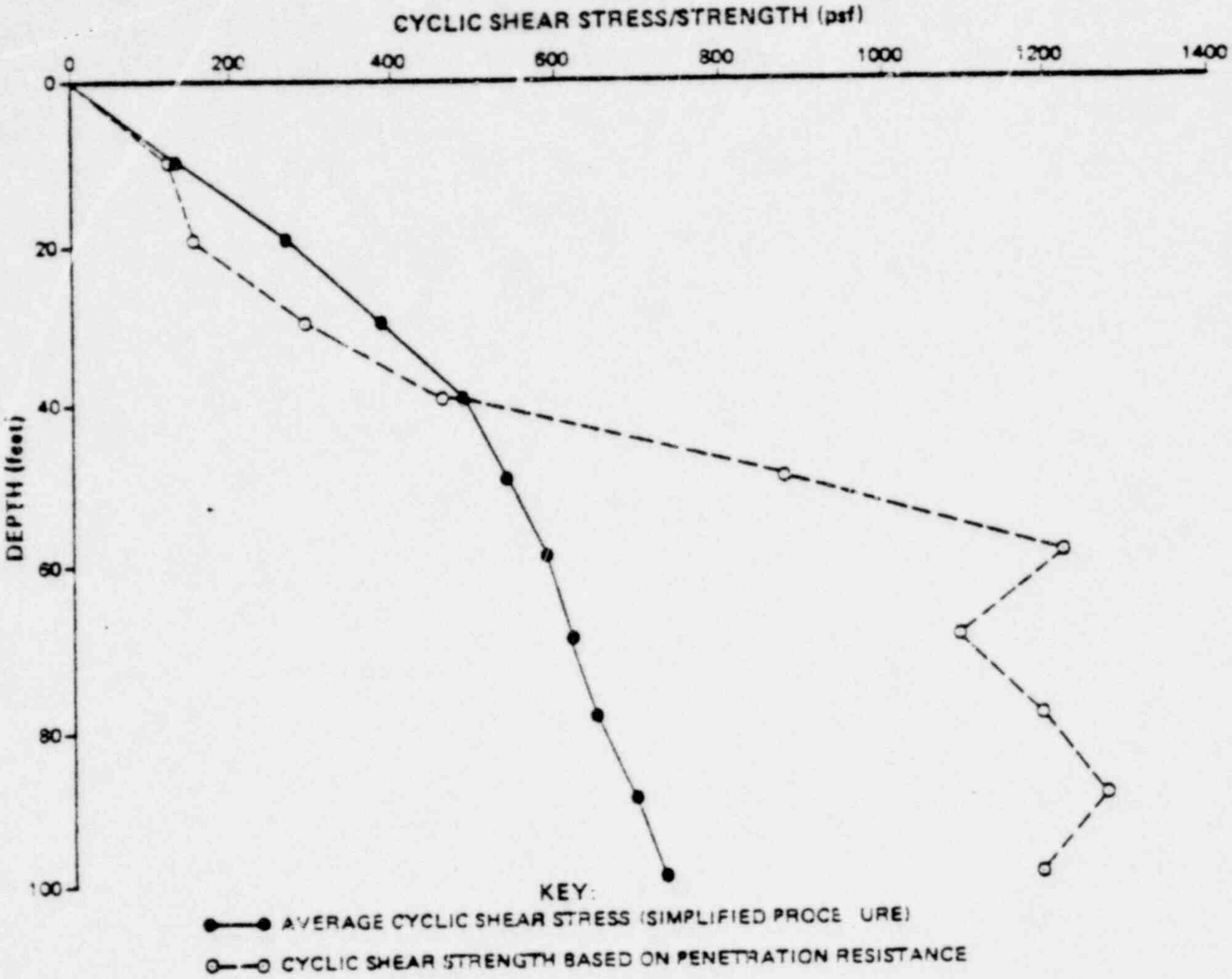
FIGURE 15



COMPARIS^N OF CYCLIC SHEAR STRESS AND STRENGTH
APPROACH 1 PROCEDURE 1 (a_{max}=0.18g)

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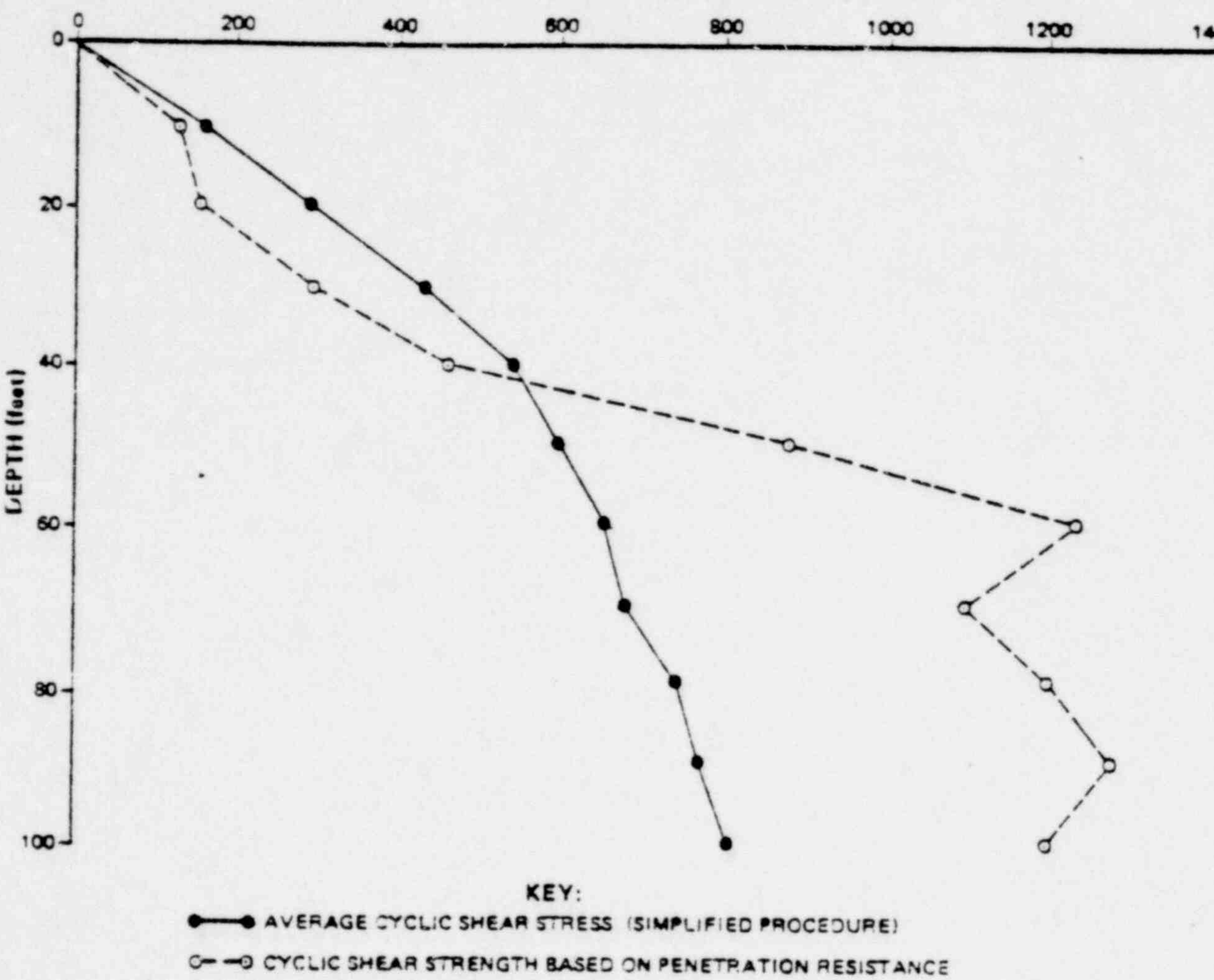
FIGURE 16



COMPARISON OF CYCLIC SHEAR STRESS AND STRENGTH
APPROACH 1 PROCEDURE 1 (a_{max}=0.20g)

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FIGURE 17

b. Estimate the cyclic shear strength using the appropriate equation:

$$R_L = 0.0042\, D_r{}^* - 0.225 \log_{10} (D_{50}/0.35)$$
for $0.04 \text{ mm} \leq D_{50} \leq 0.6 \text{ mm}$

or

$$R_L = 0.0042\, D_r{}^* - 0.05$$
for $0.6 \text{ mm} \leq D_{50} \leq 1.5 \text{ mm}$

where $D_r{}^*$ = estimated relative density from Step "a"

D_{50} = particle size in mm corresponding to 50 percent on the grain size curve

and

R_L = cyclic shear stress ratio required to cause liquefaction (triaxial test conditions).

c. Apply suitable corrections to R_L to obtain R_L^c, the corrected cyclic shear stress ratio required to cause liquefaction.

$$R_L^c = (C_1)\,(C_2)\,(C_3)\,R_L$$

where

C_1 = 0.57 (to convert triaxial test conditions to simple shear field condition)

C_2 = 1.3 to 1.5 (to account for N_{eq} = 5 rather than N_{eq} = 20 used in Japanese study

C_3 = 1 (to account for differences in failure strain criteria*).

By comparing the estimated values of cyclic shear strengths from the Japanese procedure (Ref. 8) and the average cyclic shear stresses from the Seed and Idriss procedure (Ref. 6), factors of safety against potential for liquefaction can be estimated for different depths. The cyclic shear stresses which were calculated under Procedure 1 were also used under Procedure 2. The relative densities were estimated using SPT(N) values (Figure 2) and D_{50} values (Table 1). Table 4 presents the estimated relative densities, the estimated cyclic shear strengths, the calculated cyclic shear stresses, and the resulting factors of safety at various depths for different accelerations. The stresses and strengths at different depths for all the acceleration levels considered are plotted on Figure 18. The data on Table 4 and Figure 18 suggest that there would be no liquefaction susceptibility under earthquakes producing ground surface accelerations of less than or equal to 0.16 g. Under accelerations of 0.18 g and 0.20 g LACBWR site soils between depths of 20 to 40 feet may experience liquefaction.

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TABLE 4

SUMMARY OF LIQUEFACTION ANALYSIS
APPROACH 1, PROCEDURE 2

Depth (ft)	D_r**	τ (psf)	Average Cyclic Shear Stresses and Factors of Safety for Various Accelerations*											
			a_{max} = 0.10 g		a_{max} = 0.12 g		a_{max} = 0.14 g		a_{max} = 0.16 g		a_{max} = 0.18 g		a_{max} = 0.20 g	
			τ_{av} (psf)	FS	τ_{av}	FS	τ_{av}	FS	τ_{av}	FS	τ_{av}	FS	τ_{av}	FS
10	46	161	73	2.21	88	1.83	103	1.57	118	1.37	132	1.22	147	1.10
20	41	222	145	1.53	174	1.28	203	1.09	232	0.96	261	0.85	290	0.77
30	48	365	213	1.72	255	1.43	298	1.23	341	1.07	383	0.95	426	0.86
40	53	494	270	1.83	324	1.52	378	1.31	432	1.14	486	1.02	540	0.91
50	66	745	301	2.47	361	2.06	421	1.77	482	1.54	542	1.37	602	1.24
60	75	885	328	2.70	394	2.25	459	1.93	525	1.69	590	1.50	656	1.35
70	67	1,005	343	2.93	411	2.44	480	2.09	548	1.83	617	1.63	685	1.47
80	66	1,145	363	3.16	435	2.63	508	2.25	581	1.97	653	1.75	726	1.58
90	65	1,092	389	2.81	467	2.34	545	2.00	622	1.76	700	1.56	778	1.40
100	65	1,349	409	3.30	491	2.75	573	2.35	655	2.06	737	1.83	818	1.65

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*τ_{av} = average cyclic shear stress computed using Seed & Idriss Simplified Procedure (Ref. 6).

τ = cyclic shear strength estimated from relative density (Japanese procedure), (Ref. 8).

Factor of safety (FS) = (cyclic shear strength) ÷ (average cyclic shear stress).

**D_r = relative density based on $D_r = 21 (\bar{\sigma}_v + 0.7)^{0.5}$ (Ref. 12).

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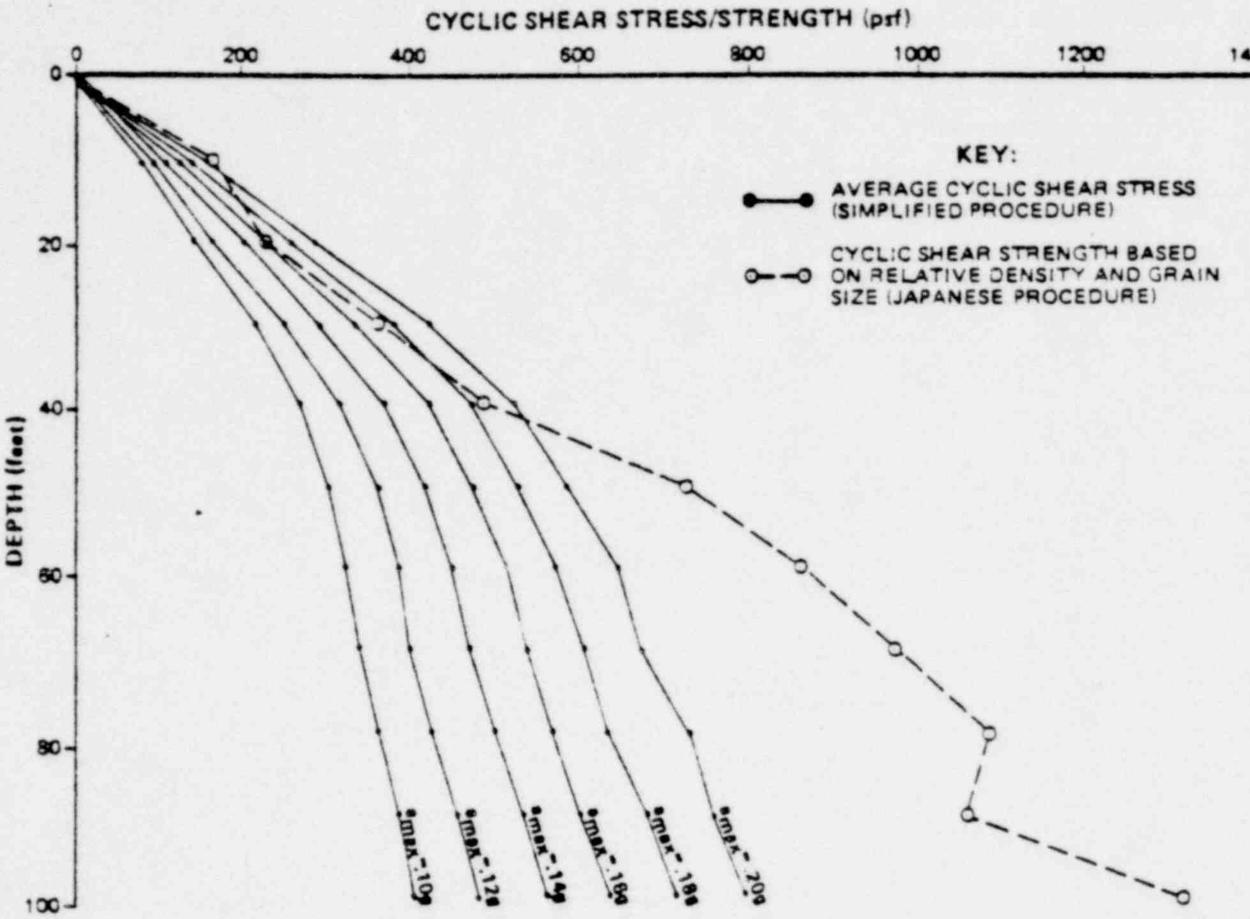

COMPARISON OF CYCLIC SHEAR STRESS AND STRENGTH
APPROACH 1 PROCEDURE 2 (a_{max}=0.10g to 0.20g)

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FIGURE 18

10.4 Evaluation of Liquefaction Potential, Approach 2

Approach 2 uses more rigorous methods and site-specific data from sophisticated laboratory results. A seismic response analysis was performed to estimate the stresses, strains, and accelerations at different depths within the soil profile resulting from SSE loading at the LACBWR site. Also, several liquefaction tests were performed on undisturbed samples to define their behavior under cyclic loading.

10.4.1 Soil Model Used in the Response Analysis. Based on a review of the data from the most recent investigation, a representative, idealized soil profile was established for the one-dimensional wave propagation analysis. This idealized soil profile corresponds to an average surface elevation of +639 feet. Table 5 presents the idealized soil profile and the soil properties that were used in the response analysis.

The upper 135 feet of the soil deposit were divided into 12 sublayers. Detailed descriptions of the soils encountered at the LACBWR site are presented in Table 5 and on the boring logs in the Appendix.

10.4.2 Soil Properties Used in the Response Analysis. The soil properties required for the wave propagation analysis are unit weight, shear modulus, damping ratio, and coefficient of earth pressure at rest.

a. Average values of unit weights are based on field and laboratory test results used in the analyses (Table 5).

b. In general, the shear modulus of a soil is influenced by several variables, including effective confining pressure, void ratio, stress history, degree of saturation, soil structure, amplitude of strain, and frequency of vibration (Ref. 14). The in situ shear modulus can be estimated by reviewing data from the geophysical survey. This value of shear modulus corresponds to low strain levels (approximately 10^{-4} percent). Shear moduli corresponding to other strain levels can be determined from strain-controlled cyclic triaxial tests and resonant column tests. The extensive field and laboratory investigations of different soils conducted by independent researchers have generally established shear modulus versus strain relationships of soils (Ref. 6). The shear modulus versus strain relationships of sands at the LACBWR site were developed by considering the generalized trends from

*The 5 to 6 percent double amplitude shear strain used in the Japanese studies was very close to the initial liquefaction criterion used in this D&M study. Therefore $C_3 = 1$ was used.

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TABLE 5

GENERALIZED SOIL PROFILE AND MODEL FOR ONE-DIMENSIONAL WAVE PROPAGATION ANALYSIS

Elevation (ft)	Depth (ft)		Design N Value	γ Wet (pcf)*	D_r**	K_2***	Shear Modulus G (ksf)
+639	0	Ground surface elevation +639					
		Hydraulic fill: brown fine- to medium sand with occasional fine gravel, trace of silt	6	115 120	45	43	376 1,162
619	20	Ground water +627 to +631					
		Gray green fine to medium sand with occasional fine gravel, trace of silt, layers of clayey silt	6-14	121 128	48	45	1,660 1,880
599	40	(Bottom of reactor vessel +610 to +618)					
		Brown fine-to-medium sand with occasional fine gravel/ coarse sand, trace of silt	14-36	126 133	66	55	2,550 2,795
579	60	(Bottom of piles approx. +580)					
559	80	Brown fine to medium sand with occasional fine gravel/ coarse sand, trace of silt	30-40	122 133 132 130	69	57	3,137 3,325 3,537 3,7[illegible]8
539	100						
		Fine-to-coarse gravel and brown fine-to-medium sand with little silt		134		90	6,244
519	115						
		Brown fine to medium sand with occasional fine gravel/ coarse sand, trace of silt		134	90	70	5,273
	135						
499		Bedrock					

*Based on field measurements.

**Based on $D_r = 21\ N/(\bar{\sigma}_v + 0.7)^{0.5}$ (Ref. 12); laboratory D_r tests, and Ref. 11.

***From expression $G = 1{,}000\ K_2\ (\bar{\sigma}_m)^{0.5}$, based on data from Ref. 6.

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the published data, the results of strain-controlled cyclic triaxial tests performed on reconstituted samples in 1973, and the relative density estimations based on SPT results.

Based on the findings of Seed and Idriss (Ref. 6), the shear modulus, at low strain levels of coarse grained soils, can be expressed by the equation

$$G = (1000)\ (K_2)\ (\bar{\sigma}_m)^{1/2}$$

where:

G = shear modulus (lb/ft^2)

$\bar{\sigma}_m$ = effective mean confining pressure (lb/ft^2)

K_2 = constant for a given compactness of soil.

The values of K_2 used in the analysis are presented in Table 5. The shear modulus varies nonlinearly with strain level. This variation was assumed to follow the pattern of average data on the coarse-grained soils presented by Seed and Idriss (Ref. 15). The nonlinear strain dependence of shear modulus that was used in this analysis is presented in Figure 19.

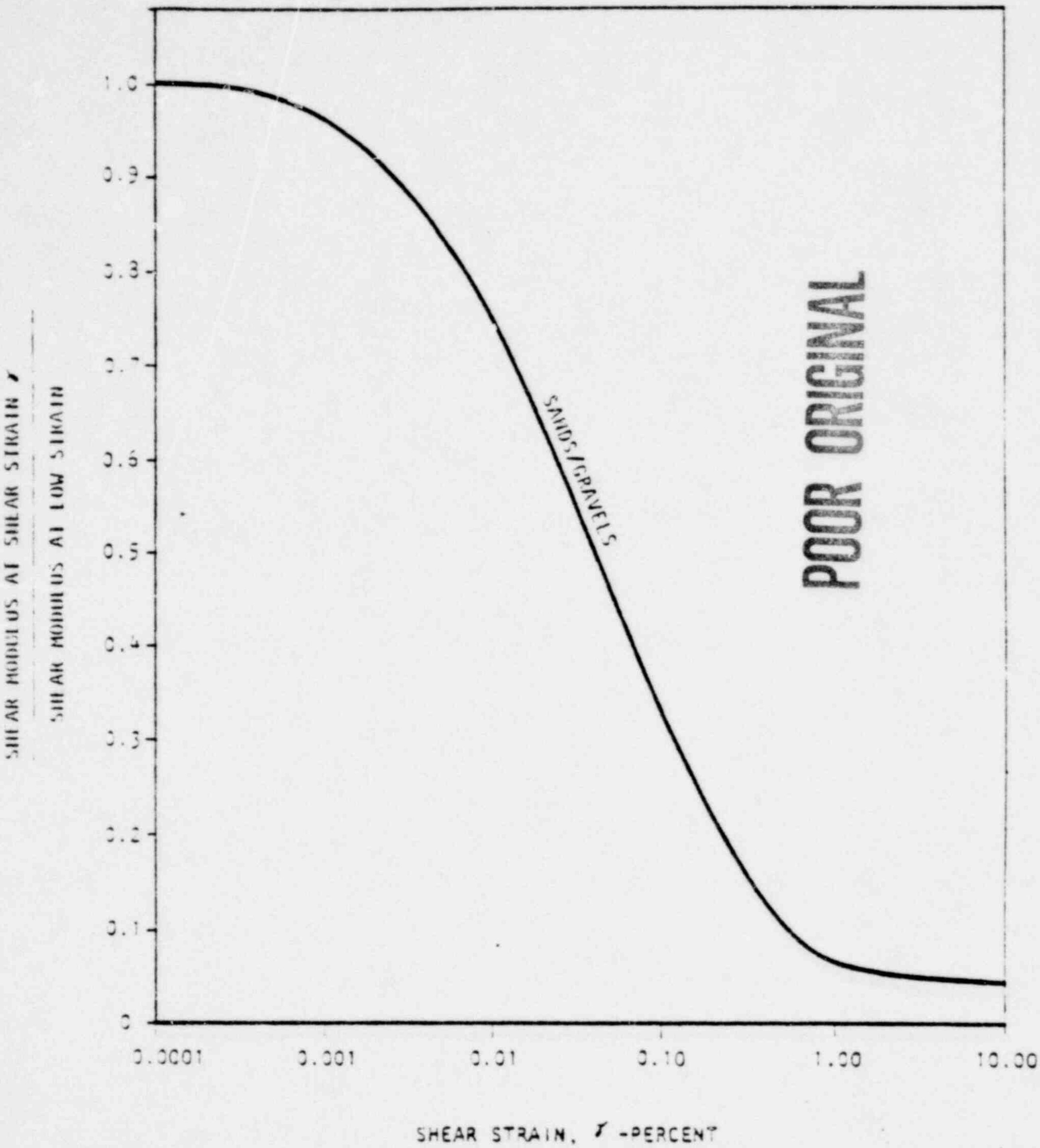
c. Most of the parameters discussed in relation to shear modulus have an opposite effect on the damping value, which increases with increasing strain amplitude, decreases slightly with increasing ambient stress, and decreases with increasing void ratio. Strain-controlled cyclic triaxial tests and resonant column tests on undisturbed samples are necessary to define experimentally the variation of the damping ratio with strain level. However, for the purposes of this study, it was considered satisfactory to assume that the damping ratio of the soils at the LACBWR site, both in magnitude and in variation with strain level, were similar to the average results found in the literature (Ref. 15). These values are the average values obtained from the experimental investigation performed by several independent researchers on typical sands. The strain-dependent values of the damping ratios for typical sands which were used in this response analysis are presented in Figure 20. (The damping ratios measured experimentally in the laboratory during the 1973 D&M investigations were also reviewed before choosing the design values).

d. A value of 0.45 was assigned for the coefficient of earth pressure for all the granular soils.

10.4.3 Design Earthquake Used in the Response Analysis. The horizontal component of a digitized acceleration-time history was used as the input motion at the surface of the soil deposit. The corresponding accelerogram is presented in Figure 21.

POOR ORIGINAL

1145 186



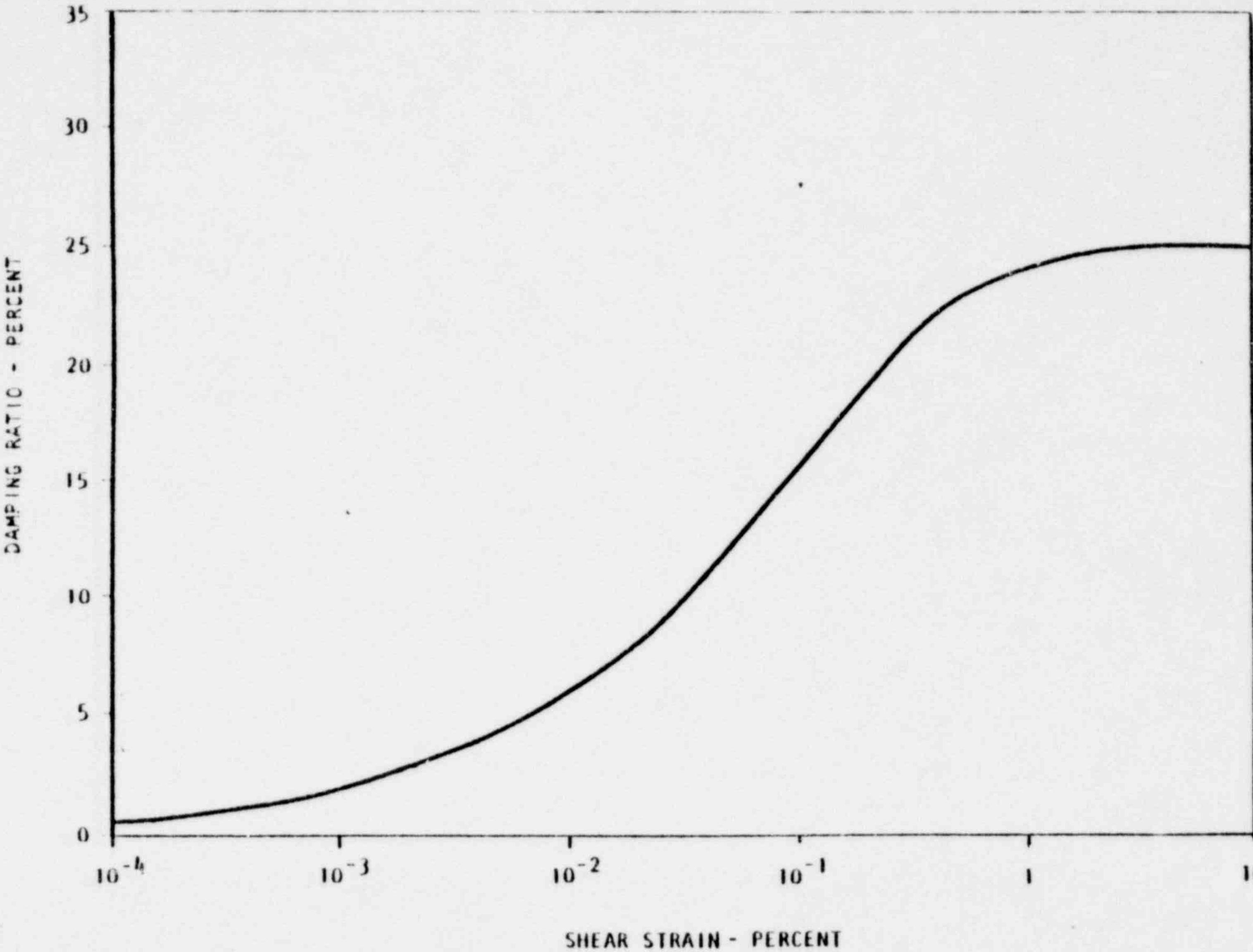
POOR ORIGINAL

1145 187

TYPICAL REDUCTION OF SHEAR MODULUS WITH SHEAR STRAIN

(SEED & IDRISS 1970)

DAMES & MOORE



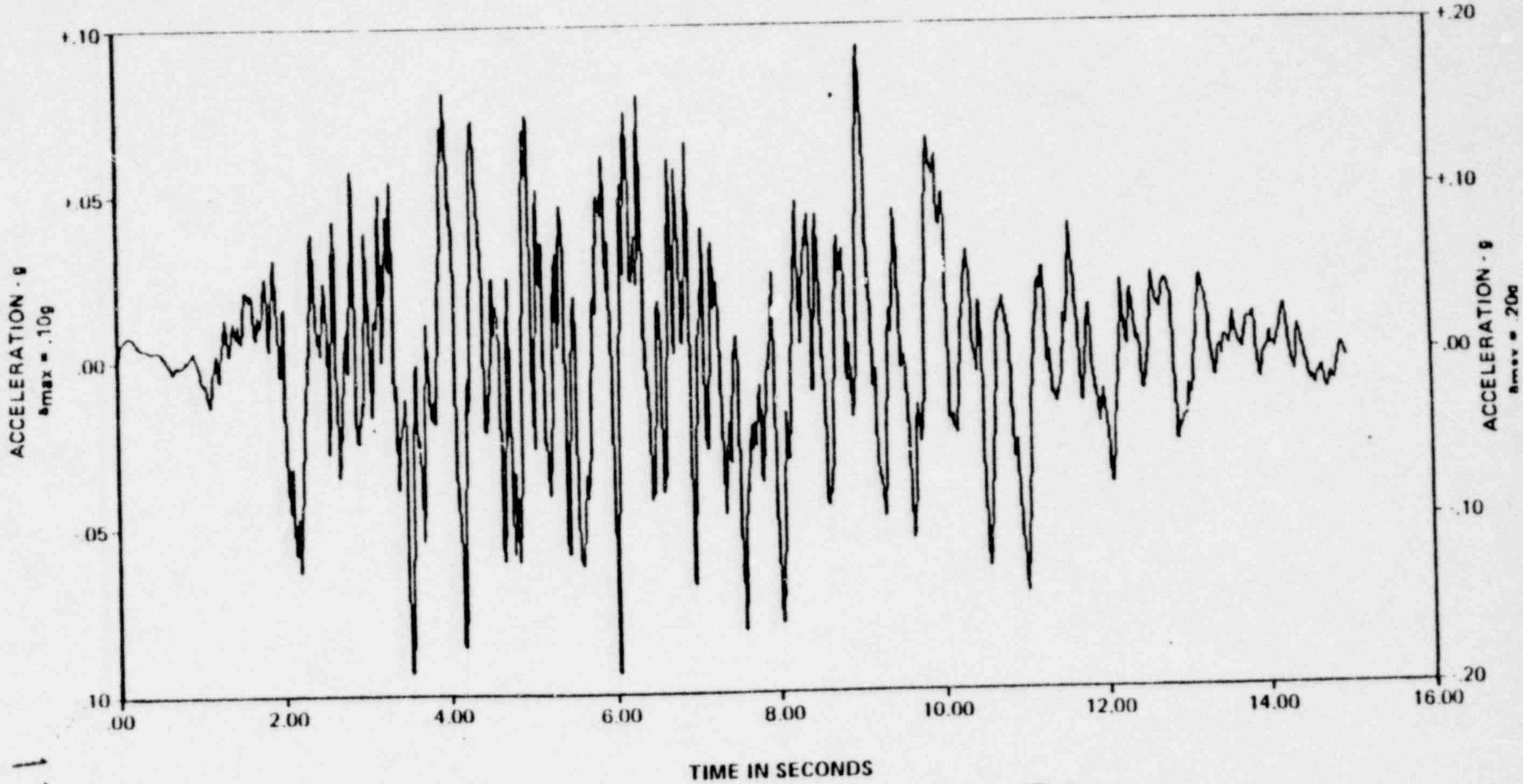
DAMPING RATIO FOR SATURATED SANDS

(SEED & IDRISS, 1

1145 188

DAMES & MOORE

FIGURE 20



POOR ORIGINAL

SSE HORIZONTAL COMPONENT (digitized time history)

1145 189

DAMES & MOORE

FIGURE 21

The duration of the design earthquake was assumed to be 15 seconds, and a range of maximum ground surface accelerations of 10 to 20 percent of gravity was used, based on the recommendations of NRC.

10.4.4 <u>One-Dimensional Wave Propogation Analysis</u>. A mathematical model was used to evaluate the response of the soils at the LACBWR site when subjected to the SSE loading. This model is based on one-dimensional strain-compatible shear wave propagation through a layered system. Each layer in the system is assumed to be isotropic, homogeneous, and of visco-elastic behavior (Ref. 16).

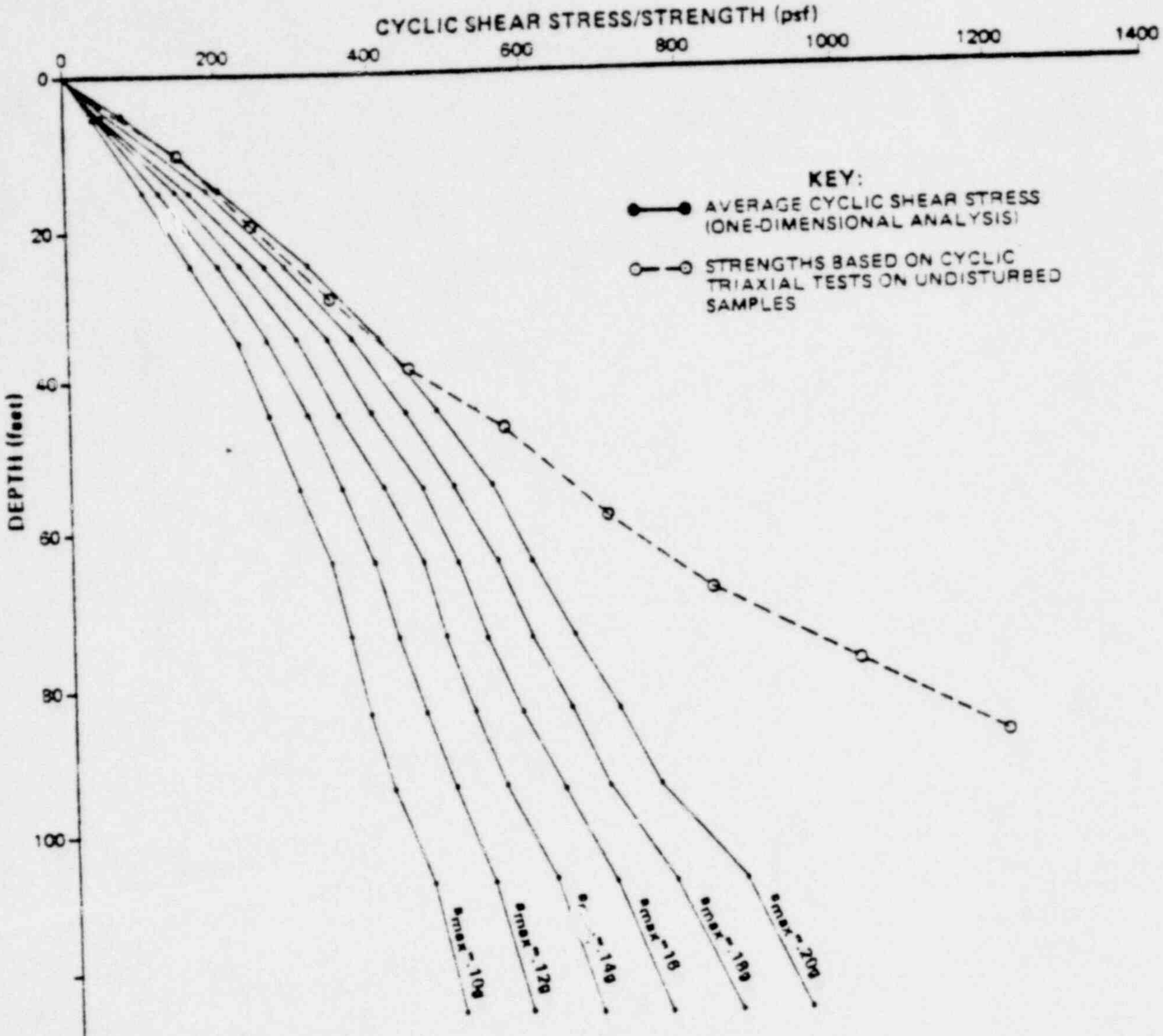
A computer program developed by Schnabel <u>et al</u>. (Ref. 16) was modified by D&M to include additional input and output. The nonlinearity of the shear modulus and damping ratio is accounted for in this program by the use of equivalent linear properties. This computer program (Ref. 17) was verified for various practical problems, certified in accordance with quality assurance requirements, and used to analyze the soils at the LACBWR site.

The average shear stress levels in the stress histories obtained by peforming one-dimensional analysis were computed assuming the ground water to be at 10 feet below ground surface. This represents the average condition; the groundwater level actually fluctuates slightly. The average cyclic shear stresses computed by performing one-dimensional analyses are plotted as functions of depth for various acceleration levels on Figure 22.

10.4.5 <u>Cyclic Shear Strength</u>. The next step in Approach 2 is to determine the cyclic shear strength of undisturbed samples obtained from various potentially liquefiable layers. Fifteen samples, representing four depths of the soil profile, were chosen for stress-controlled cyclic triaxial testing using standard procedures described in Section 9.6. Figure 3 shows the envelope of particle size curves for the samples used. The results of these tests are summarized in Table 2. The test results were plotted on a semilogarithmic plot to define the relationship between stress ratio and number of cycles required to cause initial liquefaction (Figures 23 through 27).

1145 190

POOR ORIGINAL

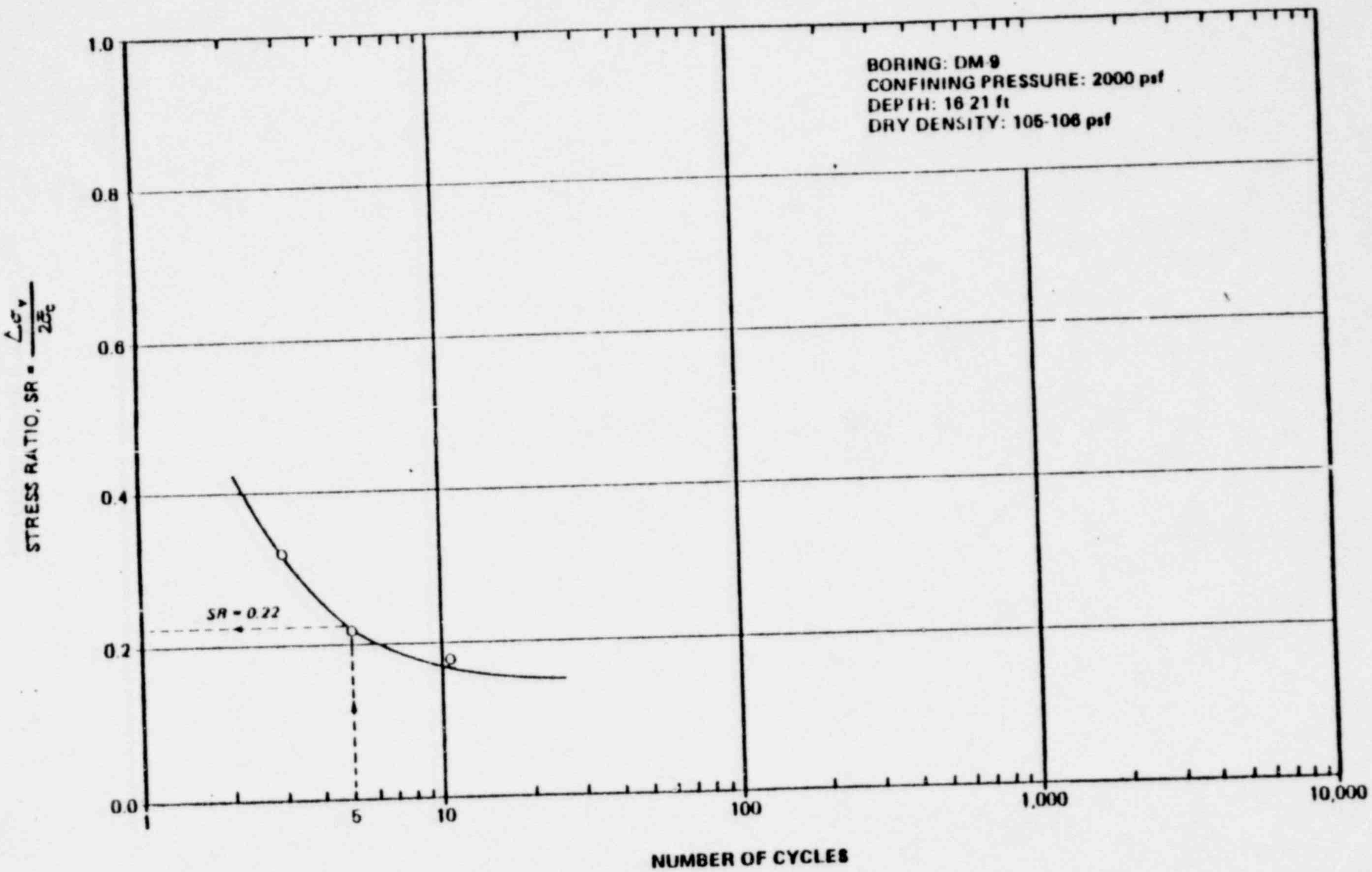


COMPARISON OF CYCLIC SHEAR STRESS AND STRENGTH
APPROACH 2 (a_{max} = 0.10g to 0.20g)

1145 191

DAMES & MOORE

FIGURE 22

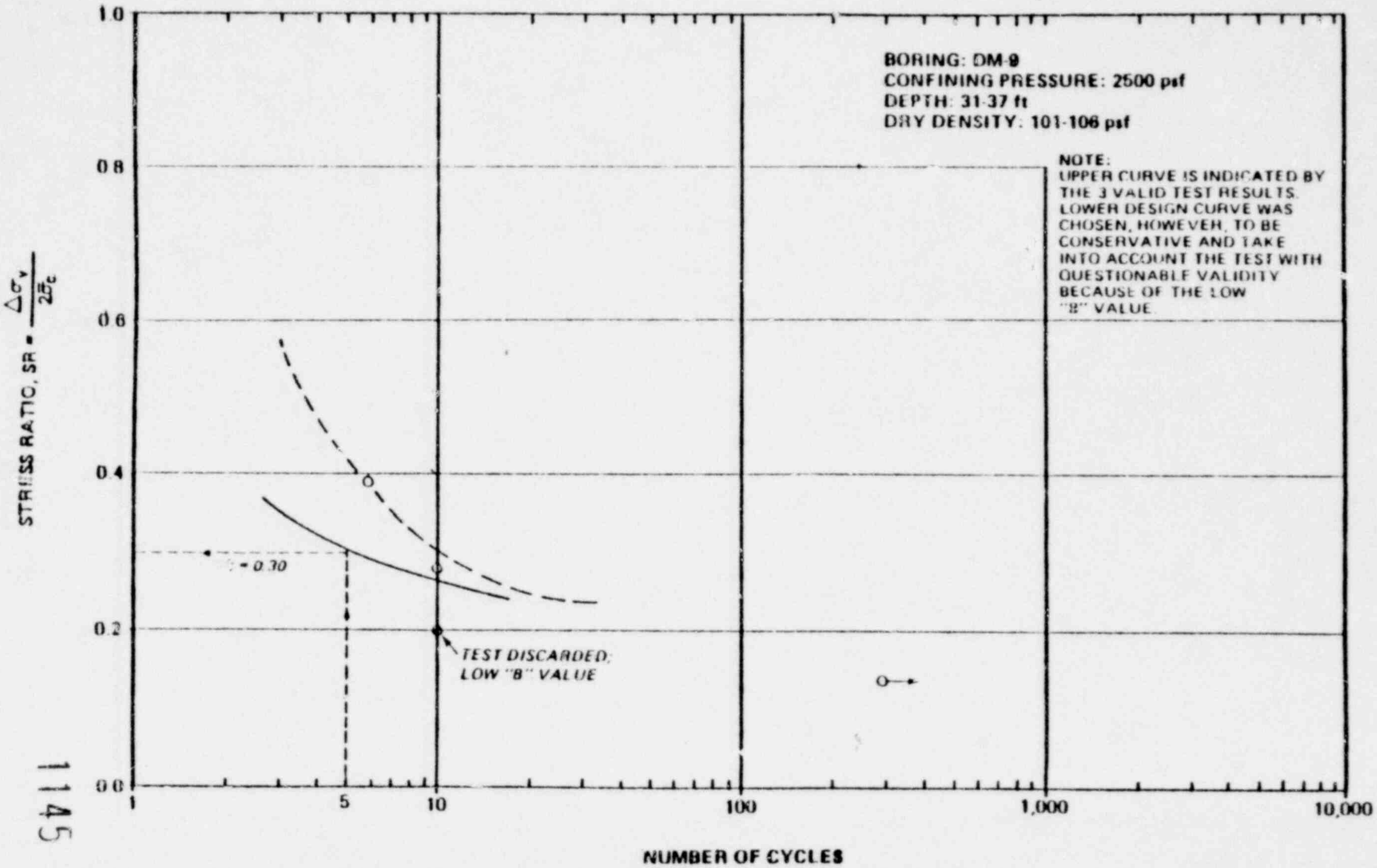


CYCLIC SHEAR STRESS RATIO VERSUS NUMBER OF CYCLES FOR INITIAL LIQUEFACTION, TEST NO. 1, DEPTH 16 TO 21 FEET

DAMES & MOORE

1145 192

FIGURE 23

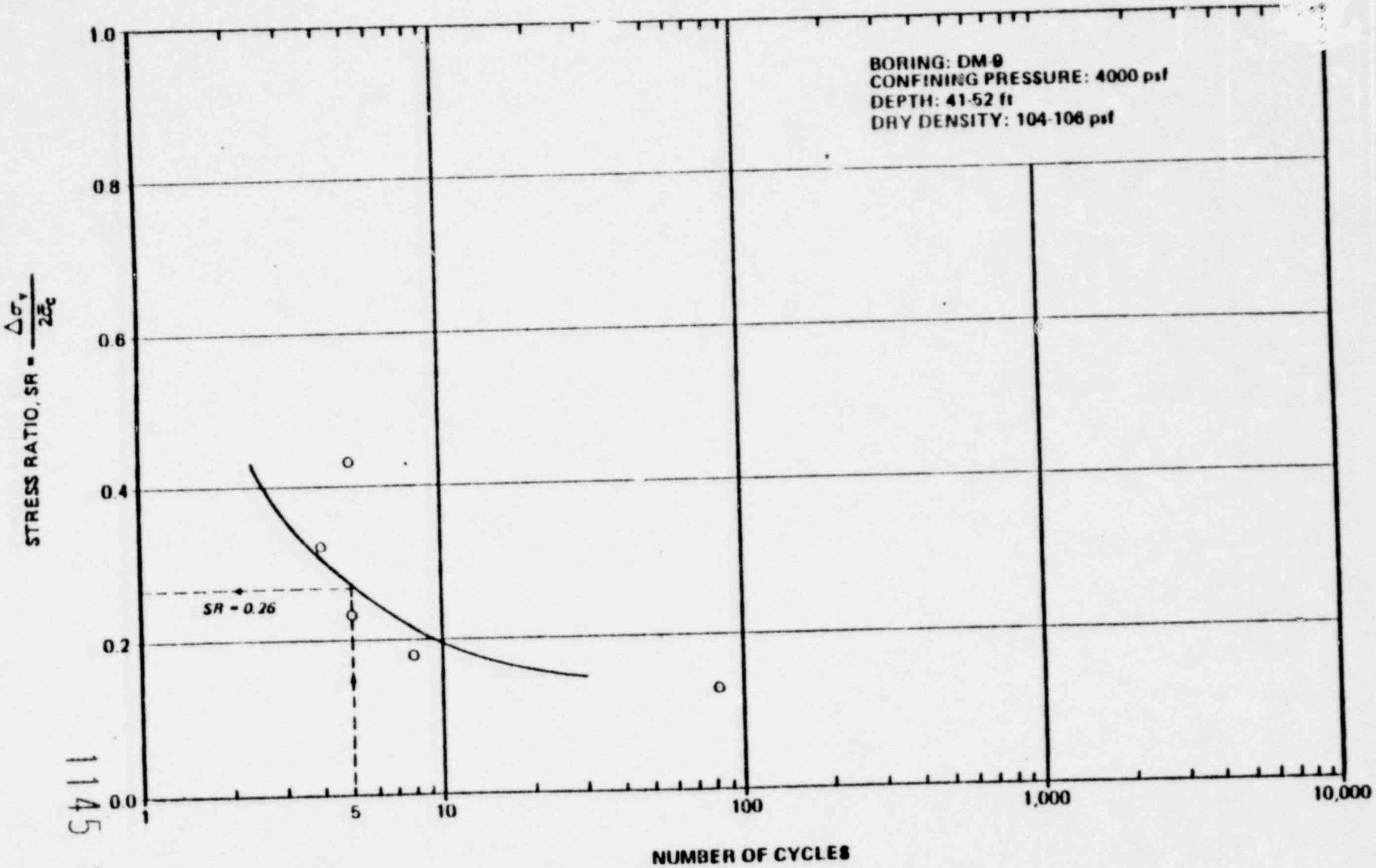


CYCLIC SHEAR STRESS RATIO VERSUS NUMBER OF CYCLES FOR INITIAL LIQUEFACTION, TEST NO. 2, DEPTH 31 TO 37 FEET

1145 193

DAMES & MOORE

FIGURE 24

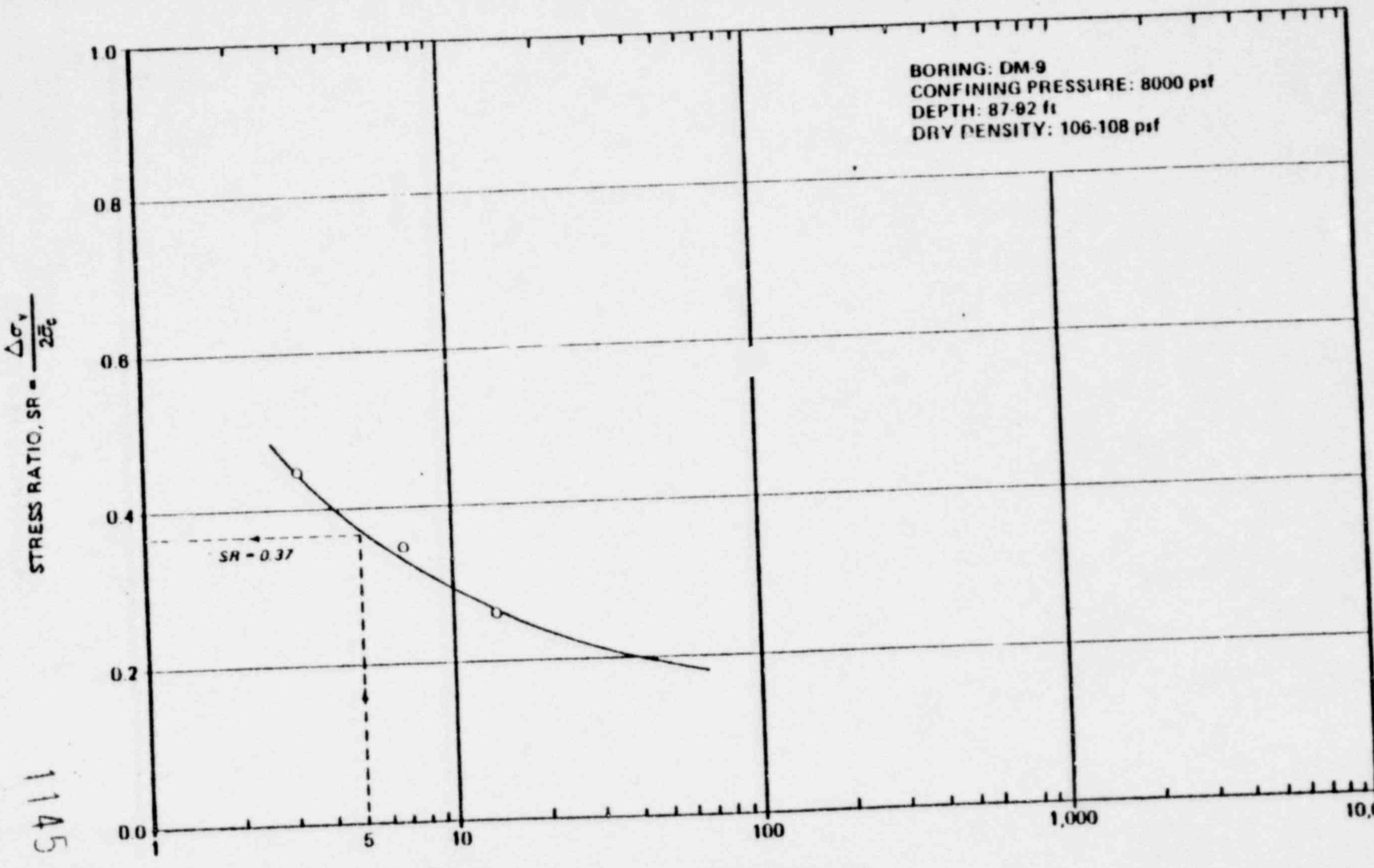


CYCLIC SHEAR STRESS RATIO VERSUS NUMBER OF CYCLES FOR INITIAL LIQUEFACTION, TEST NO. 3, DEPTH 41 TO 52 FEET

1145 194

DAMES & MOORE

FIGURE 25

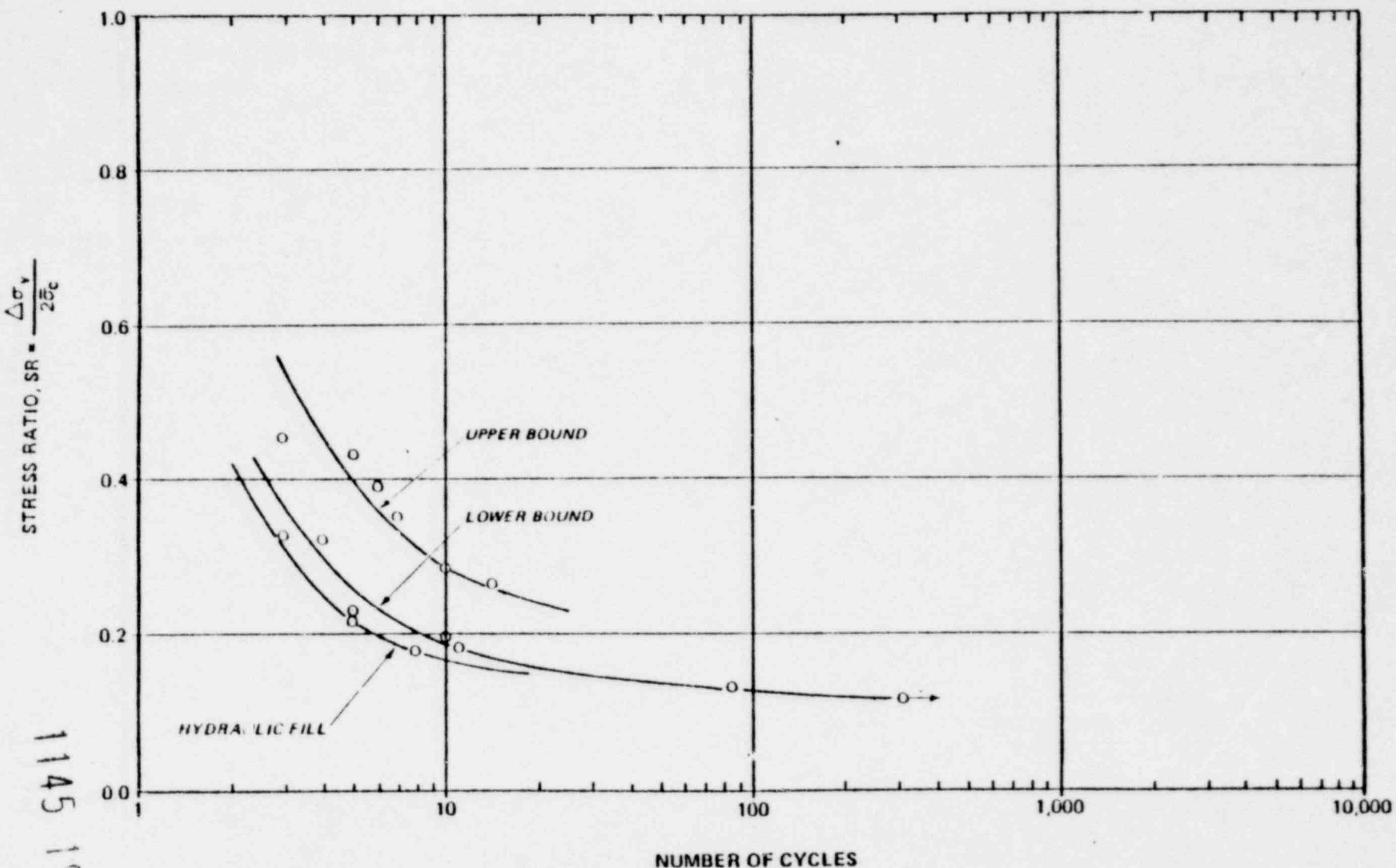


CYCLIC SHEAR STRESS RATIO VERSUS NUMBER OF CYCLES FOR INITIAL LIQUEFACTION,
TEST NO. 4, DEPTH 87 TO 92 FEET

1145 195

DAMES & MOORE

FIGURE 26



SUMMARY CURVE SHOWING EFFECTS OF DENSITY AND SOIL FABRIC ON NUMBER OF CYCLES TO 5% DOUBLE AMPLITUDE STRAIN

1145 196

DAMES & MOORE

FIGURE 27

10.4.6 Conversion of Irregular Stress History Into Equivalent Uniform Cyclic Stress Series. In Approach 2, the calculated cyclic shear stresses are compared directly with those required to cause liquefaction of representative soil samples in the laboratory. It is usually more convenient to perform laboratory tests using uniform cyclic stress applications than to attempt to reproduce a representative field stress history. Therefore, it is necessary to convert the irregular stress history that is actually developed during earthquakes into an equivalent uniform cyclic stress series.

There are three basic methods by which this conversion can be accomplished. However, it has been shown that the procedures used in this step of the analysis have little effect on the final analysis (Ref. 7). Based on the results of a statistical study of the representative numbers of cycles developed during a number of different earthquake motions, a convenient basis for selecting an equivalent uniform cyclic stress series for earthquakes of different magnitudes has been presented by Seed (Ref. 7). According to Seed, for earthquake magnitudes between 5 and 6, the number of equivalent cycles, N_{eq}, is approximately 5, corresponding to an average cyclic shear stress, τ_{av}, of 65 percent of the maximum shear stresses. Therefore, the maximum cyclic shear stresses that were obtained by performing a one-dimensional wave propagation analysis were multiplied by 0.65 to obtain the average cyclic shear stress at any point.

10.4.7 Correction Factor. The cyclic triaxial test does not directly simulate the simple shear conditions actually induced during an earthquake. Also, the effect of multidirectional shaking is not included in this testing. As a result, the stress ratio obtained in the cyclic triaxial test is higher, and a correction factor, C_r, is applied to modify these values. For normally consolidated sands (K_0 = 0.4), a value of 0.57 is considered appropriate for C_r (Ref. 7), and the stress ratio causing liquefaction in the laboratory is multiplied by 0.57 to account for the field conditions.

Figure 22 represents the summary of cyclic shear stress computation using the one-dimensional analysis and the cyclic shear strengths from the laboratory tests. The stresses poltted are 65 percent of the maximum shear stresses to correspond to an average condition. The cyclic shear strengths

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were obtained by following the procedures mentioned below. Figure 27 is a summary of all liquefaction test results. It can be seen that three distinct liquefaction curves can be drawn on the various data points--an upper bound and lower bound for natural materials below the hydraulic fill material. Three stress ratios corresponding to these three curves can be chosen for a given number of cycles, N_{eq}. For N_{eq} of 5, the three possible relations between confining pressure and the cyclic shear stress required to cause liquefaction (cyclic shear strength) are plotted on Figure 28. The shaded zone shows the scatter of data for natural soils below the hydraulic fill. However, the nonlinear effects of the relationship between confining pressure and the cyclic shear strength can best be estimated by selecting the stress ratios from data on each individual test presented on Figures 23,24, 25, and 26. These four tests represent four different confining pressures ranging from 2,000 to 8,000 psf.

A design strength curve was drawn on Figure 28 by selecting the four different stress ratios from Figures 23 through 27. It can be seen that design curve chosen represents a lower bound of strength to almost 4,000 psf of confining pressure. (This confining pressure represents the crucial depths up to 50 feet where liquefaction potential is of primary concern.) A field strength curve corresponding to 57 percent of the laboratory triaxial strength curve has been drawn on Figure 28. It is this curve that was used to determine the strengths at various depths.

10.4.8 Factor of Safety Computation. The cyclic shear stress required to cause liquefaction at a particular depth is found from the field strength curve B of Figure 28 by reading off the ordinate corresponding to the confining pressure at that depth. The variation of the cyclic shear strength, the induced average shear stress (obtained by the one-dimensional wave propogation analysis), and their ratio (that is, the factor of safety against liquefaction) with depth are summarized in Tables 6. The cyclic shear stresses and strength are also presented as a function of depth on Figure 22. Table 6 and Figure 22 show that even with a very conservative interpretation of strength data, no liquefaction is predicted up to an acceleration level of 0.20 g.

10.4.9 Discussion and Conclusions. During the current investigation, the liquefaction potential at the LACBWR site was studied using a simplified

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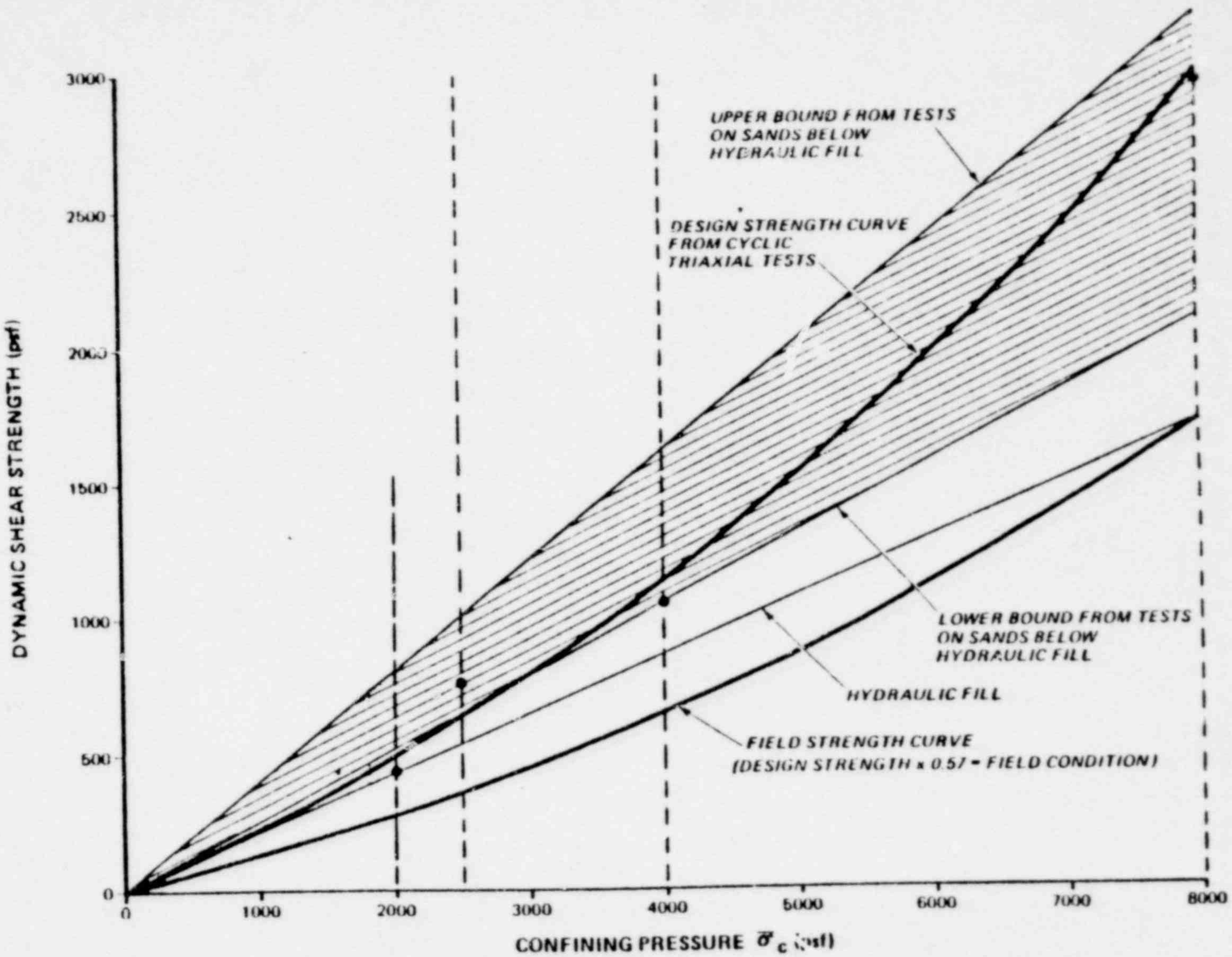

RELATION BETWEEN CYCLIC SHEAR STRENGTH AND CONFINING PRESSURE BASED ON CYCLIC TRIAXIAL TESTS

1145 199

DAMES & MOORE

FIGURE 28

TABLE 6

SUMMARY OF LIQUEFACTION ANALYSIS
APPROACH 2

		Average Cyclic Shear Stresses and Factors of Safety for Various Accelerations*											
		a_{max} = 0.10 g		a_{max} = 0.12 g		a_{max} = 0.14 g		a_{max} = 0.16 g		a_{max} = 0.18 g		a_{max} = 0.20 g	
Depth (ft)	τ psf)	τ_{av} (psf)	FS	τ_{av}	FS	τ_{av}	FS	τ_{av}	FS	τ_{av}	FS	τ_{av}	FS
10	150	72	2.08	86	1.74	101	1.49	114	1.32	122	1.17	141	1.06
20	250	134	1.87	163	1.53	189	1.32	214	1.17	238	1.05	262	0.95
30	350	194	1.80	233	1.50	268	1.31	302	1.16	340	1.03	367	0.95
40	460	244	1.89	290	1.59	332	1.39	371	1.24	411	1.12	450	1.02
50	590	284	2.08	339	1.74	387	1.52	433	1.36	479	1.23	525	1.12
60	730	322	2.27	384	1.90	439	1.66	490	1.49	539	1.35	587	1.24
70	860	356	2.42	420	2.05	478	1.80	532	1.62	585	1.47	639	1.35
80	1,050	381	2.76	449	2.34	512	2.05	573	1.83	633	1.66	695	1.51
90	1,240	407	3.05	482	2.57	550	2.25	620	2.00	684	1.81	750	1.65

POOR ORIGINAL

*τ_{av} = average cyclic shear stress = (maximum cyclic shear stress from one-dimensional analysis) x (0.65).

τ = cyclic shear strength = (triaxial cyclic shear strength) x (0.57).

Factor of safety (FS) = (cyclic shear strength) ÷ (average cyclic shear stress).

1145 200

approach and a rigorous approach. In the simplified approach, stresses were computed using empirical equations and strengths were estimated using past experience during earthquakes at various sites that liquefied and also using relative densities. In the rigorous approach, stresses were computed using a one-dimensional model and wave propagation response analysis. The strengths were measured by performing cyclic triaxial tests on undisturbed samples.

The following conclusions were based on these analyses:

a. Using Approach 1, Procedure 1 (Seed and Idriss stress and strength based on SPT, N values), no liquefaction is predicted up to a maximum acceleration level of 0.12 g. The potential for liquefaction is suggested for various maximum surface acceleration levels stated below:

a_{max}	depths prone to liquefaction
0.10 g	none
0.12 g	none
0.14 g	20 to 30 feet
0.16 g	20 to 30 feet
0.18 g	10 to 40 feet
0.20 g	10 to 40 feet.

b. Using Approach 1, Procedure 2 (Seed and Idriss stress and strength based on relative densities), no liquefaction is predicted up to a maximum acceleration level of 0.16 g. The potential for liquefaction is predicted for various maximum acceleration levels stated below:

a_{max}	depths prone to liquefaction
0.10 g	none
0.12 g	none
0.14 g	none
0.16 g	20 feet
0.18 g	20 to 30 feet
0.20 g	20 to 40 feet.

c. Using Approach 2 (stresses from one-dimensional wave propagation analysis and strength from stress-controlled cyclic triaxial tests peformed in the laboratory on undisturbed samples), no liquefaction is predicted up to a maximum acceleration level of 0.20 g. (The factor of safety against potential for liquefaction at 0.20 g is close to unity between depths of 20 to 30 feet.)

The rigorous analysis made during the current investigation is relatively more accurate than all other analyses made at the LACBWR plant site.

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We believe that a high degree of confidence can be assigned to the rigorous analysis made during the current investigations for the following reasons:

a. The test boring and sampling program was performed under carefully controlled conditions using state-of-the-art techniques.

b. The undisturbed samples were drained and frozen at the site before transporting for storage and were kept frozen until just before testing (partially eliminating sample disturbance at the site).

c. The frozen samples were carefully packaged and transported by D&M field engineers to minimize any possible sample disturbance during transport.

d. State-of-the-art testing techniques were used to determine the in situ densities and the cyclic shear strengths of samples.

e. All the field and laboratory investigations were subject to stringent quality assurance and quality control requirements of D&M, DPC, and NRC.

In summary, given our present knowledge and understanding of the seismicity of the region and the behavior of soils under dynamic loading, it is our opinion that there is little threat of liquefaction at the LACBWR site under a maximum acceleration level corresponding to a realistic design SSE that can be assigned to the site.

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11.0 SUMMARY OF LIQUEFACTION ANALYSES AT THE LACBWR SITE

Tables 3, 4, and 5 summarize the liquefaction analyses performed thus far at the LACBWR site. The D&M analysis of 1973 (Ref. 1) was conservative and concluded that the factors of safety against potential for liquefaction under the design SSE at various depths were adequate. WES performed a very conservative analysis (Ref. 3) and concluded that the minimum factor of safety was close to unity under a 0.12-g ground acceleration. As a result of D&M's review of its past work and the WES report, and reevaluation of the various analyses, it was concluded that the factors of safety were indeed adequate. However, there were certain questions that were raised by NRC regarding the lack of test data on undisturbed samples and the lack of continuous standard penetration test results. Since the existing data did not satisfy these new concerns, DPC decided to perform modest field and laboratory investigations and limited analyses to verify the earlier findings on liquefaction potential.

As a result of the new state-of-the-art investigations performed at the LACBWR plant site, it is now concluded that the LACBWR plant site has an adequate factor of safety against potential for liquefaction under any realistic design SSE that can be assigned to the plant site. However, in the absence of an NRC decision regarding a design SSE and a corresponding design acceleration level, a range of acceleration between 0.10 g and 0.20 g was assumed and factors of safety were estimated. The minimum factors of safety are listed below for the different acceleration levels:

a_{max}	Depth (ft)	Factor of Safety
0.10 g	30	1.80
0.12 g	30	1.50
0.14 g	30	1.31
0.16 g	30	1.16
0.18 g	30	1.03
0.20 g	20-30	close to unity (0.95)

Based on these results, it can be concluded that the threshold liquefaction resistance at the LACBWR site occurs for a design SSE which yields a maximum ground surface acceleration greater than 0.18 g and less than 0.20 g.

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APPENDIX

BORING LOGS

1145 206

KEY TO LOG OF BORINGS

LEGEND:

12 — INDICATES DEPTH OF STANDARD SPLIT SPOON SAMPLE.

INDICATES NUMBER OF BLOWS REQUIRED TO DRIVE STANDARD SPLIT SPOON ONE FOOT IN STANDARD PENETRATION TEST.

INDICATES DEPTH OF SPT SAMPLING ATTEMPT WITH NO RECOVERY.

P — INDICATES DEPTH OF RELATIVELY UNDISTURBED SAMPLE OBTAINED WITH OSTERBERG PISTON SAMPLER.

INDICATES THAT SAMPLE TUBE WAS PUSHED INTO SOIL BY HYDRAULIC PRESSURE.

INDICATES DEPTH OF DISTURBED SAMPLE OBTAINED WITH OSTERBERG PISTON SAMPLER.

ELEVATIONS REFER TO THE USGS MEAN SEA LEVEL DATUM.
APPROXIMATE LOCATIONS OF BORINGS ARE SHOWN ON PLOT PLAN.
CLASSIFICATION SYMBOLS REFER TO UNIFIED CLASSIFICATION SYSTEM, PLATE A-2.
DISCUSSION IN TEXT IS NECESSARY FOR COMPLETE UNDERSTANDING OF THE SUBSURFACE MATERIALS.

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SOIL CLASSIFICATION CHART

MAJOR DIVISIONS			GRAPH SYMBOL	LETTER SYMBOL	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	CLEAN GRAVELS (LITTLE OR NO FINES)		GW	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
				GP	POORLY-GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
	MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)		GM	SILTY GRAVELS, GRAVEL-SAND-SILT MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES
	SAND AND SANDY SOILS	CLEAN SAND (LITTLE OR NO FINES)		SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE				SP	POORLY-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
	MORE THAN 50% OF COARSE FRACTION PASSING NO. 4 SIEVE	SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)		SM	SILTY SANDS, SAND-SILT MIXTURES
				SC	CLAYEY SANDS, SAND-CLAY MIXTURES
FINE GRAINED SOILS	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY
				CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
				OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILTY SOILS
				CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
				OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS				PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

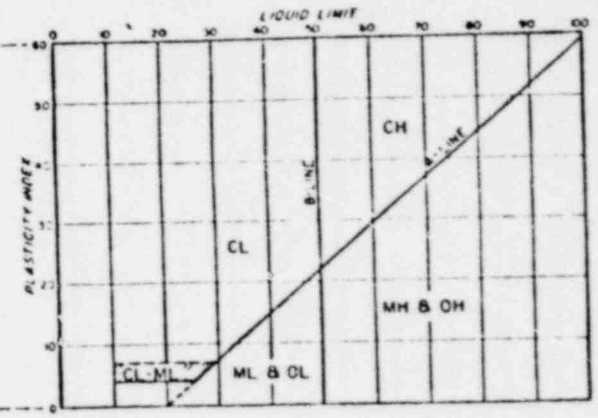
FOR LABORATORY CLASSIFICATION OF FINE-GRAINED SOILS

GRADATION CHART

MATERIAL SIZE		PARTICLE SIZE			
		LOWER LIMIT		UPPER LIMIT	
		MILLIMETERS	SIEVE SIZE*	MILLIMETERS	SIEVE SIZE*
SAND	FINE	0.074	#200	0.42	#40
	MEDIUM	0.42	#40	2.00	#10
	COARSE	2.00	#10	4.76	#4
GRAVEL	FINE	4.76	#4	19.1	3/4"+
	COARSE	19.1	3/4"+	76.2	3"+
COBBLES		76.2	3"+	304.8	12"+
BOULDERS		304.8	12"+	914.4	[illegible]

* U.S. STANDARD + CLEAR SQUARE OPENINGS

PLASTICITY CHART



NOTES

1. DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE CLASSIFICATIONS.
2. WHEN SHOWN ON THE BORING LOGS, THE FOLLOWING TERMS ARE USED TO DESCRIBE THE CONSISTENCY OF COHESIVE SOILS AND THE RELATIVE COMPACTNESS OF COHESIONLESS SOILS.

COHESIVE SOILS	APPROXIMATE SHEARING STRENGTH IN KSF	COHESIONLESS SOILS	
VERY SOFT	LESS THAN .25	VERY LOOSE	THESE ARE USUALLY BASED ON AN EXAMINATION OF SOIL SAMPLES, PENETRATION RESISTANCE, AND SOIL DENSITY DATA
SOFT	.25 TO 0.5	LOOSE	
MEDIUM STIFF	0.5 TO 1.0	MEDIUM DENSE	
STIFF	1.0 TO 2.0	DENSE	
VERY STIFF	2.0 TO 4.0	VERY DENSE	
HARD	GREATER THAN 4.0		

SAMPLES

- INDICATES UNDISTURBED SAMPLE
- INDICATES DISTURBED SAMPLE
- INDICATES SAMPLING ATTEMPT WITH NO RECOVERY
- I INDICATES LENGTH OF CORING RUN

NOTE
DEFINITIONS OF ANY ADDITIONAL DATA REGARDING SAMPLES ARE ENTERED ON THE FIRST LOG ON WHICH THE DATA APPEAR

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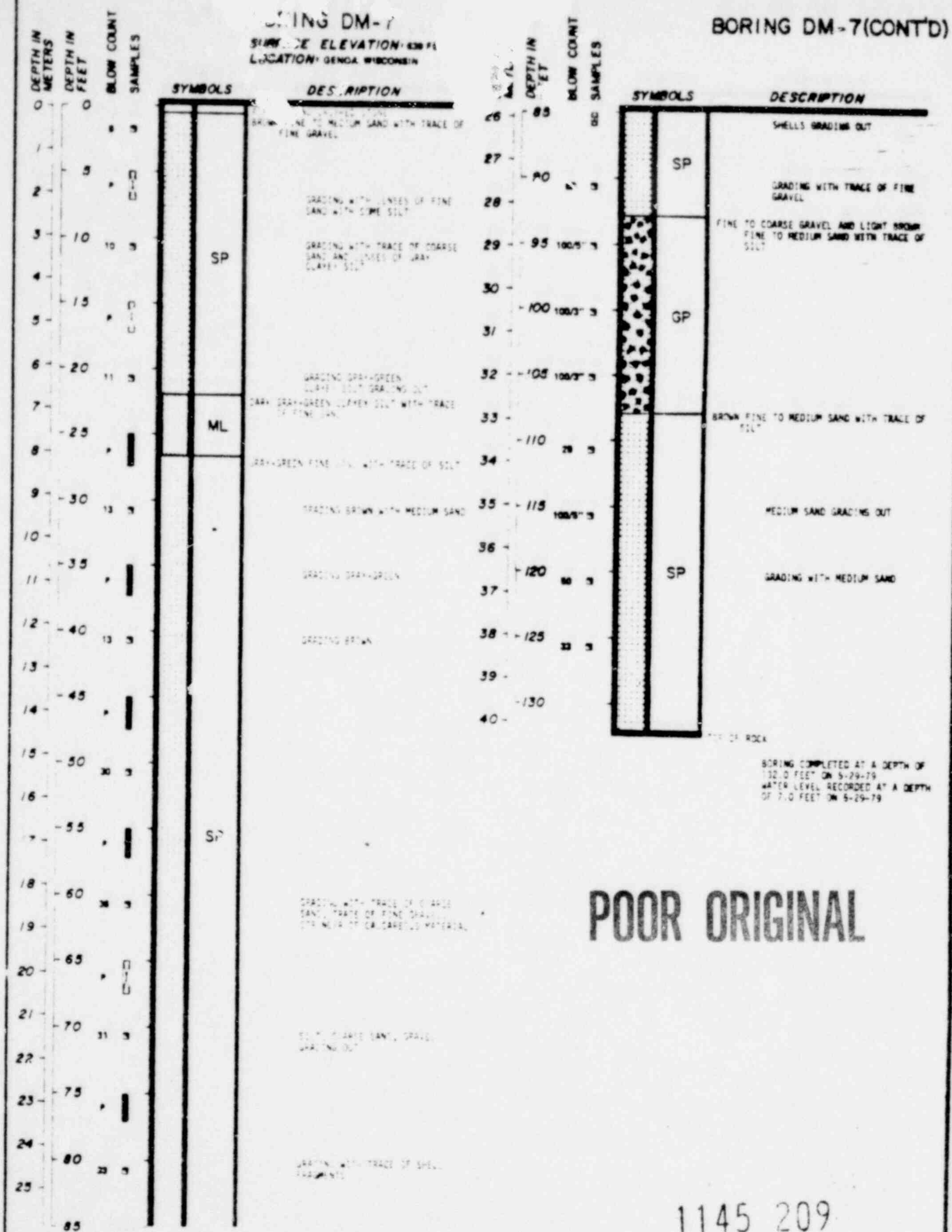
UNIFIED SOIL CLASSIFICATION SYSTEM

POOR ORIGINAL

DAMES & MOORE

BORING DM-7



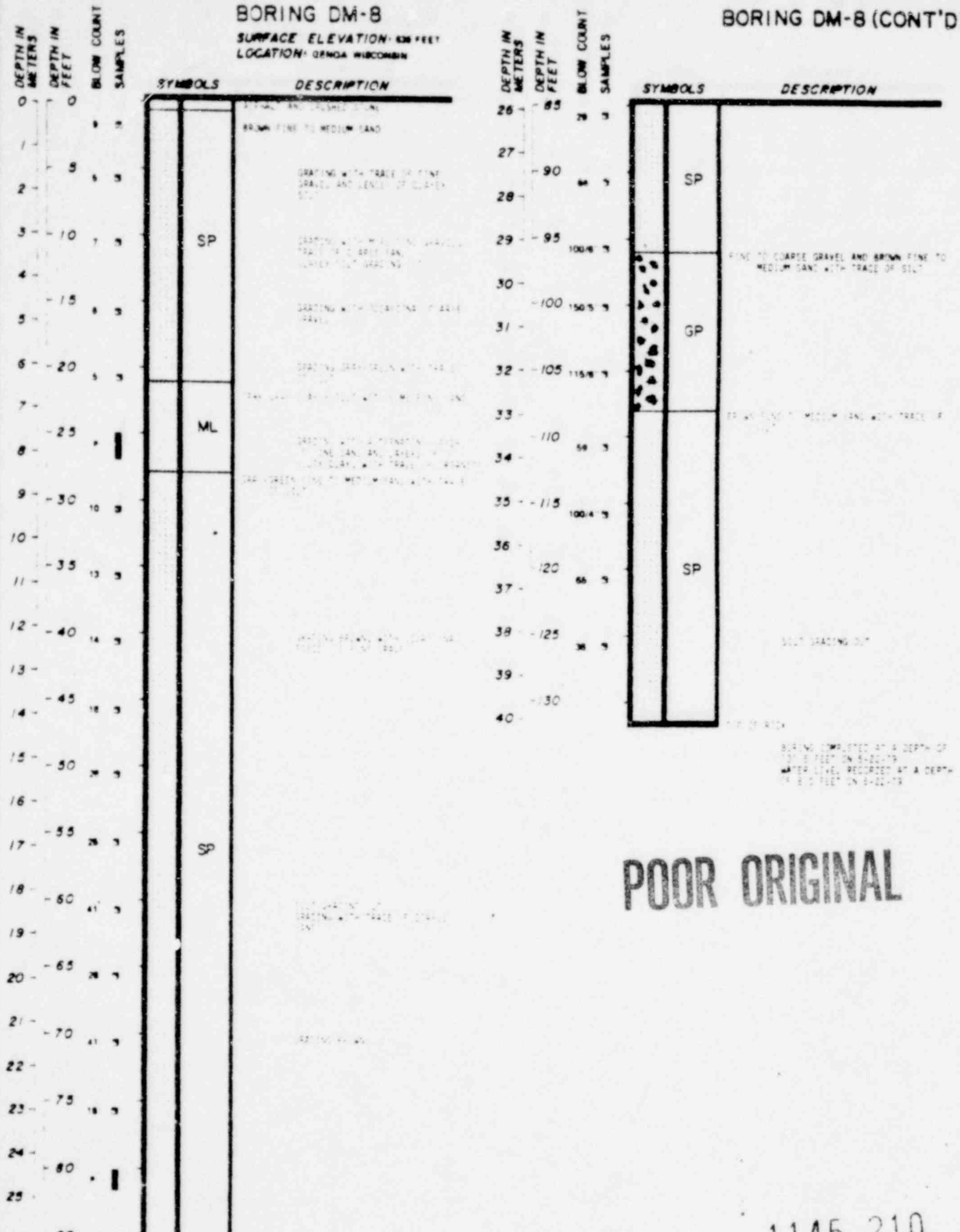
POOR ORIGINAL

1145 209

LOG OF BORING

DAMES & MOORE

PLATE A-3



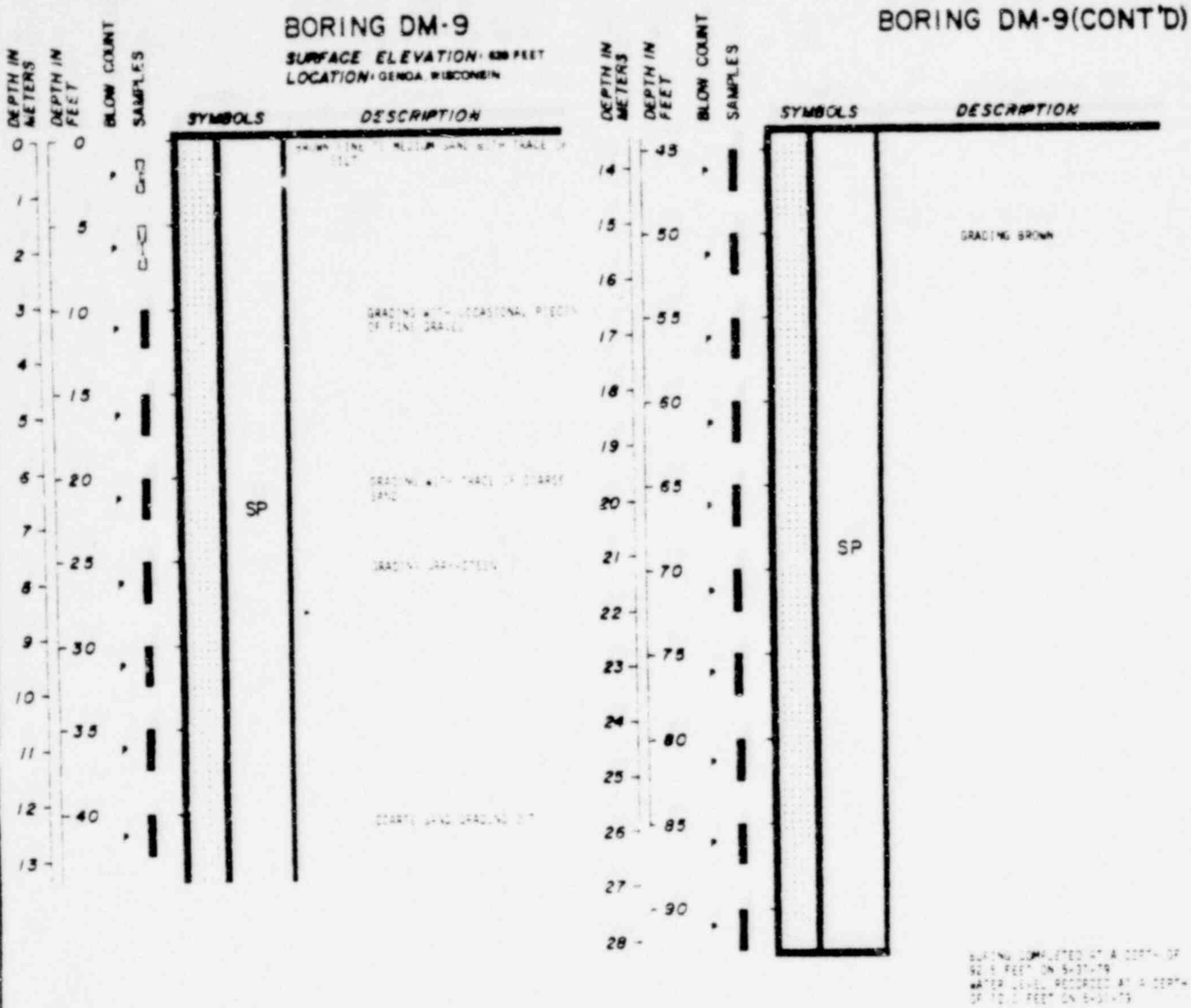
POOR ORIGINAL

1145 210

LOG OF BORING

DAMES & MOORE

PLATE A-4



POOR ORIGINAL

LOG OF BORING

1145 211

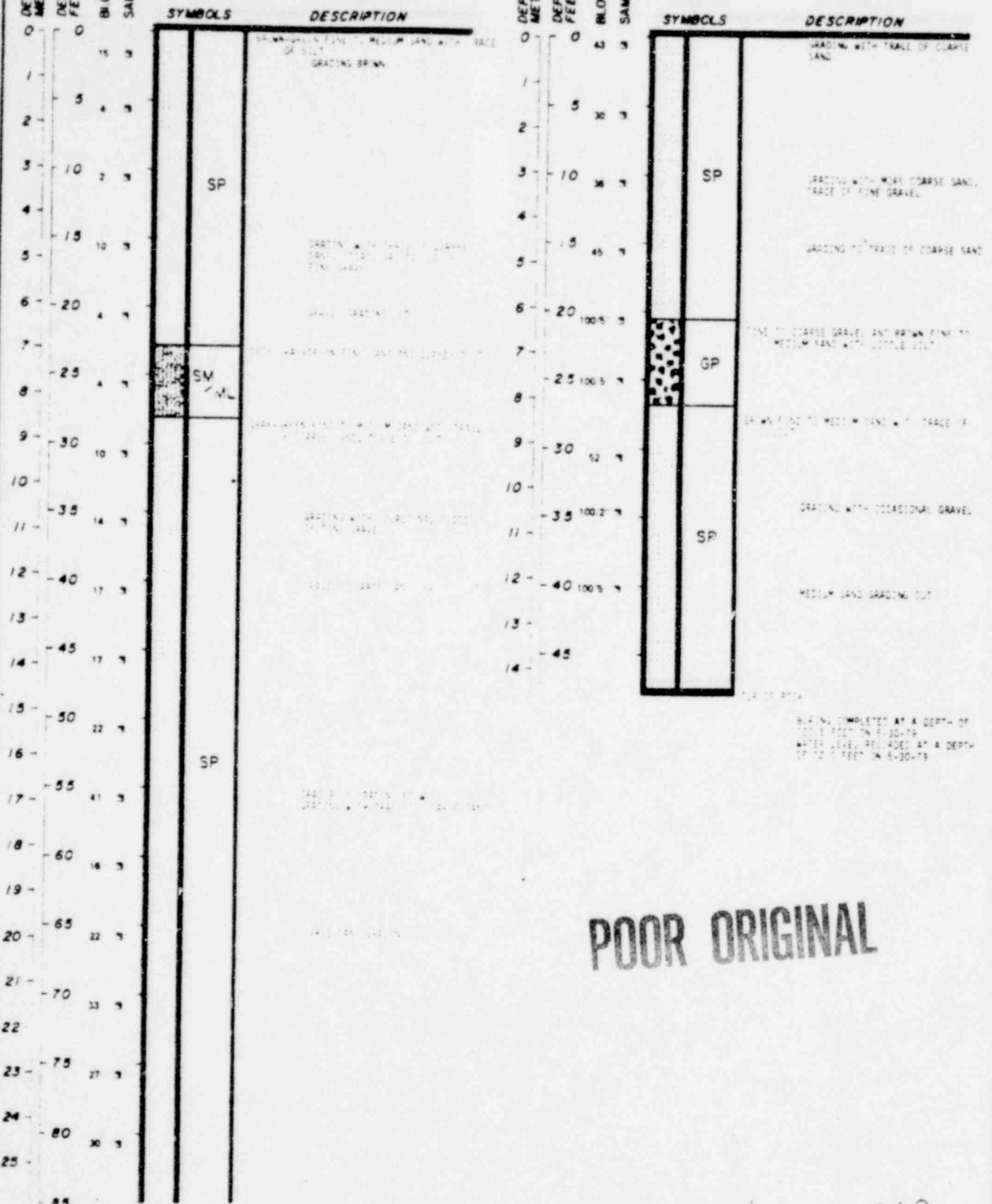
DAMES & MOORE

PLATE A-5

BORING DM-10

SURFACE ELEVATION: 630 FEET
LOCATION: GENOA WISCONSIN

BORING DM-10 (CONT'D)

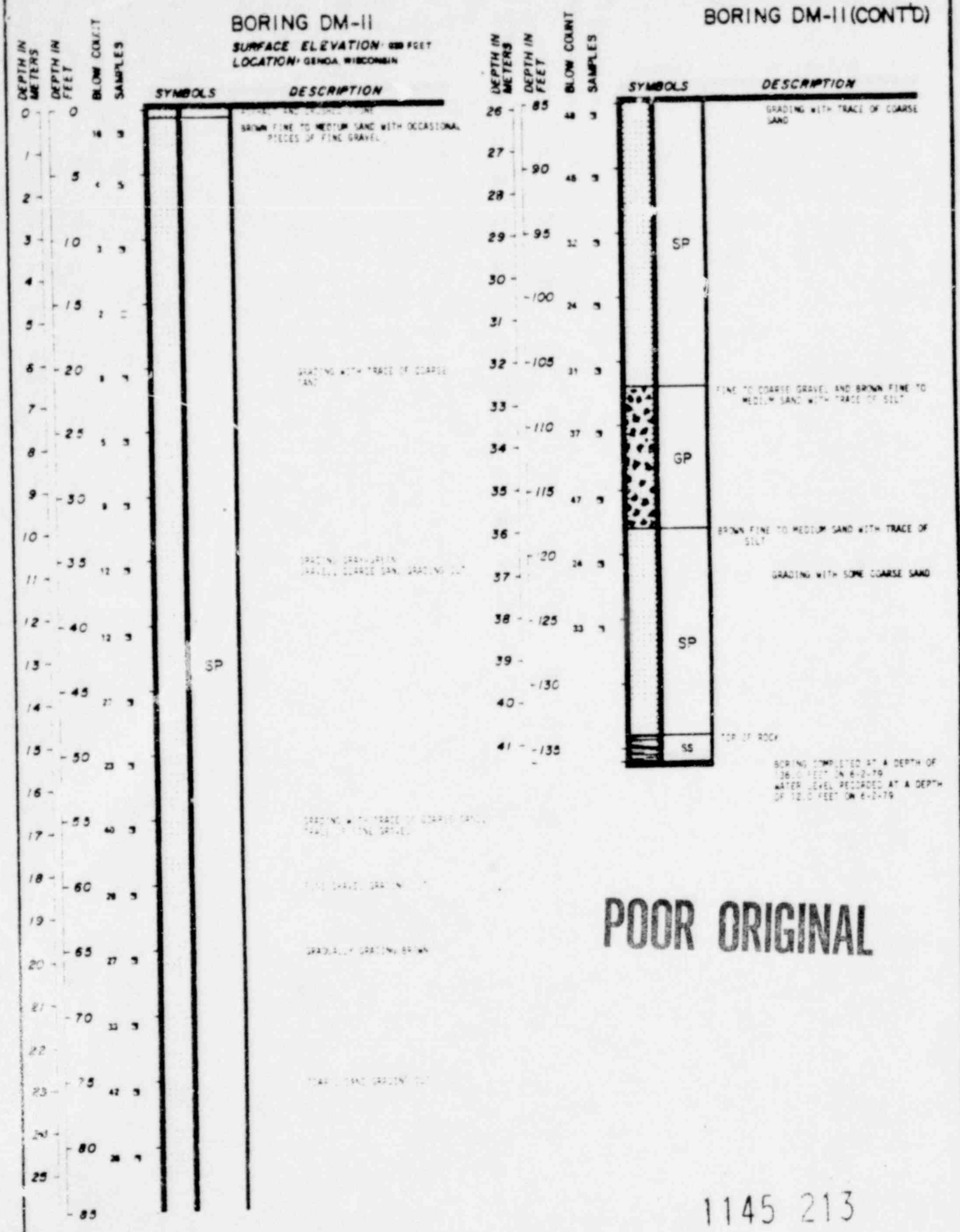


1145 212

LOG OF BORING

DAMES & MOORE

PLATE A-6



POOR ORIGINAL

1145 213

LOG OF BORING

DAMES & MOORE

PLATE A-7