January 27, 2000

Template #NRR-088

Mr. D. E. Young, Vice President
Carolina Power & Light Company
H. B. Robinson Steam Electric Plant,
Unit No. 2
3581 West Entrance Road
Hartsville, South Carolina 29550

SUBJECT: H. B. ROBINSON STEAM ELECTRIC PLANT UNIT 2 - ISSUANCE OF REQUEST FOR ADDITIONAL INFORMATION ON INDIVIDUAL PLANT EXAMINATION OF EXTERNAL EVENTS (IPEEE) FOR SEVERE ACCIDENT VULNERABILITIES, 10 CFR 50.54(F) (TAC NO. M83688)

Dear Mr. Young:

As a result of ongoing review of your submittals, particularly the latest one dated February 5, 1999, in response to our request for additional information (RAI) and discussion during the staff visit to H. B. Robinson 2 on September 23-24, 1998, the staff at this time is unable to conclude that you have met the intent of Supplement 4 to Generic Letter 88-20. Therefore, the staff, in consultation with Sandia National Laboratories (SNL), our consultant on the subject, has developed an RAI with special emphasis in the area of seismic analysis. In order to help you understand the staff's concern with your analysis, we have enclosed a copy of a draft technical evaluation report from SNL which provides highlights on the differences between the staff and your approach.

We request that you provide your written response to this RAI within 60 days of the receipt of this letter.

If you have any questions regarding this request, please do not hesitate to contact me at (301) 415-1478.

Sincerely,

/RA/

Ram Subbaratnam, Project Manager, Section 2
Project Directorate II
Division of Licensing Project Management
Office of Nuclear Reactor Regulation

Docket No. 50-261

Enclosures:
1. Robinson Supplemental RAI
2. Draft Technical Evaluation Report

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ACRS
OPA
OC/LFDCB
B. Bonser, RII

cc: H. B. Robinson 2 Service List

UNITED STATES
NUCLEAR REGULATORY COMMISSION
WASHINGTON D C 20555-0001

January 27, 2000

Mr. D E Young, Vice President
Carolina Power & Light Company
H. B. Robinson Steam Electric Plant,
Unit No. 2
3581 West Entrance Road
Hartsville, South Carolina 29550

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cc w/encls: See next page

cc.

Mr. William D Johnson
Vice President and Corporate Secretary
Carolina Power & Light Company
Post Office Box 1551
Raleigh, North Carolina 27602

Ms. Karen E. Long
Assistant Attorney General
State of North Carolina
Post Office Box 629
Raleigh, North Carolina 27602

U.S. Nuclear Regulatory Commission
Resident Inspector's Office
H B. Robinson Steam Electric Plant
2112 Old Camden Road
Hartsville, South Carolina 29550

Mr. T. D. Walt
Plant General Manager
Carolina Power & Light Company
H. B. Robinson Steam Electric Plant, Unit No. 2
3581 West Entrance Road
Hartsville, SC 29550

Mr. J. W. Moyer
Director of Site Operations
Carolina Power & Light Company
H. B. Robinson Steam Electric Plant, Unit No. 2
3581 West Entrance Road
Hartsville, South Carolina 29550

Public Service Commission
State of South Carolina
Post Office Drawer 11649
Columbia, South Carolina 29211

Mr. H. K. Chernoff
Supervisor, Licensing/Regulatory Programs
Carolina Power & Light Company
H. B. Robinson Steam Electric Plant,
Unit No. 2
3581 West Entrance Road
Hartsville, South Carolina 29550

Mr. Mel Fry, Director
N C Department of Environment
and Natural Resources
Division of Radiation Protection
3825 Barrett Dr
Raleigh, North Carolina 27609-7721

Mr Robert P Gruber
Executive Director
Public Staff - NCUC
Post Office Box 29520
Raleigh, North Carolina 27626-0520

Mr. Virgil R. Autry, Director
South Carolina Department of Health
Bureau of Land & Waste Management
Division of Radioactive Waste Management
2600 Bull Street
Columbia, South Carolina 29201

Mr. Terry C. Morton
Manager
Performance Evaluation and
Regulatory Affairs CPB 7
Carolina Power & Light Company
Post Office Box 1551
Raleigh, North Carolina 27602-1551

Mr. John H. O'Neill, Jr.
Shaw, Pittman, Potts & Trowbridge
2300 N Street, NW.
Washington, DC 20037-1128

Mr. R. L. Warden
Manager - Regulatory Affairs
Carolina Power & Light Company
H. B. Robinson Steam Electric Plant,
Unit No. 2
3581 West Entrance Road
Hartsville, South Carolina 29550-0790

Robinson 2 IPEEE Supplemental Request for Additional Information Related to Postulated Seismic Events

The licensee's February 5, 1999, response to the staff's request for additional information (RAI) based on the site visit in the seismic area was insufficient to allow the staff to conclude that Robinson 2 has met the intent of Supplement 4 to Generic Letter 88-20 Responses to the following question (follow-up to previous RAIs) is necessary in order to complete our review

Supplemental RAI·

Experience from past earthquakes has shown soil failures, including soil liquefaction and slope instabilities, to be a significant concern An issue has been raised with regard to the H B. Robinson - Unit 2 (HBR) IPEEE soil failure and liquefaction analyses

H. B Robinson is a deep soil site The submittal characterizes the top 50 ft of soil at the Robinson site as containing various beds of moderate to dense sands interspersed with layers of relatively weak to moderate strength silty sands, sandy silts, and silty clays. The submittal has indicated, from an initial iteration of liquefaction evaluation, that localized soil lenses which might be subject to liquefaction are likely to occur for a Mw 5.5 earthquake producing Review Level Earthquake (RLE) motions; and based on an updated liquefaction analysis, a "statistically insignificant number of data points" indicated liquefaction. Thus, for an analysis based on higher magnitudes (for instance, MW, 6.0) significant liquefaction may be possible.

Earthquake experience has identified seismically induced breaks of piping buried in soft soil or soils that have experienced liquefaction to be a significant concern. This possibility and the increased susceptibility associated with deterioration of piping (which the IPEEE indicates has been observed at Robinson 2 for buried service water piping), for the various possible site soil characteristics and beyond-design-basis magnitudes, have not been adequately addressed in Response A.2.3. The evaluation of the potential and effects of seismically induced failures of buried piping (e.g., service water piping and fuel oil transfer lines) founded in soil should be performed for critical (i.e., the most susceptible) soil conditions/locations subjected to RLE motions.

The utility submittal concluded that there are no concerns pertaining to soil failures at Robinson 2, and that, with respect to all soil failure modes and all earth structures, a High-Confidence-of-low-probability of failure in excess of the 0.3g RLE exists. As noted above, these findings have not yet been reasonably justified.

To ensure that the investigation of soil failures (particularly for analyses of liquefaction susceptibility, dynamic instability, etc.,) is relevant to RLE motions, a consistent earthquake magnitude should be used. Alternatively, sensitivity investigations for various magnitudes (e.g., MW 5.5, 6.0, etc.) can be performed. The magnitude and extent of liquefaction-induced soil shear strains at the plant site, within the slope and foundation materials of Lake Robinson Dam, and along submerged embankments, should be addressed based on the critical observed soil properties. Loss of soil strength and consequential reductions in lateral resistance of foundation pile systems of essential structures should be assessed. Impacts of the potential for pipe breaks and differential soil settlements/displacements on essential structures and components, for representative critical cases, should be examined. Seismic-induced stresses in buried piping, accounting for deterioration in piping materials, should also be investigated for critical soil characteristics/locations.

As discussed in the attached draft Technical Evaluation Report (TER), the staff is concerned that the appropriate factors have been applied in the seismic analysis The draft TER provides a detailed discussion and comparison of the Robinson 2 analysis, as submitted, with that of our contractor This comparison highlights the differences When the licensee re-evaluates the buried piping, the licensee should give careful consideration to the analyses in the TER.

With this background, please provide your related findings as to whether or not there are any soil failure issues of concern for Robinson 2, with respect to the RLE Please discuss fully the data (soil properties, seismic capability characteristics of structures or components, plant configuration, dam configuration, slope configurations, earthquake characteristics, etc.), the methodological details, and the results of your soil failure assessments which support your specific findings and conclusions For any piping system which is shown to not be qualified for the RLE, please specify the maximum ground acceleration that the ground and piping can withstand without resulting in pipe failure or soil liquefaction-induced piping failure

SUPPLEMENTARY

TECHNICAL EVALUATION REPORT

BASED ON THE

SITE VISIT AND STEP-2 REVIEW OF THE

INDIVIDUAL PLANT EXAMINATION OF EXTERNAL EVENTS SUBMITTAL

FOR THE

H. B. ROBINSON (UNIT 2) PLANT

FINAL DRAFT REPORT

October 11, 1999

Michael P. Bohn
U. S. NRC Senior Review Board Seismic Consultant
Sandia National Laboratories
Albuquerque, NM 87103

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1.0 INTRODUCTION

The initial "submittal only" review [1] of the Individual Plant Examination of External Events (IPEEE) submittal for the H. B. Robinson - Unit 2 plant [2] raised some concerns that required further consideration before a satisfactory review could be completed. A request for additional information (RAI) was sent to the licensee [3] requesting the information needed to complete the review. The licensee provided responses to the initial RAIs [4], but the responses in certain areas were not adequate to resolve the concerns. Hence, the US NRC determined that a Level 2 review involving a site audit would be conducted. A supplementary set of RAI questions [5] was sent to the utility, as well as a description of the planned site audit [6]. The site audit was focussed primarily on (a) the seismic fragility analyses (HCLPF calculations), (b) the soil liquefaction analysis, and (c) a concern relating to an interfacing systems LOCA issue. This report summarizes the results of the Level 2 review of the remaining issues raised in the supplemental RAIs based on information obtained during the site visit and further information provided by the licensee [7] following the visit.

2.0 PRINCIPAL ISSUES INVESTIGATED DURING THE SITE VISIT

The principal issues investigated during the site audit were documented in the supplemental round of RAIs sent to the licensee [5], in the (draft) technical evaluation report [1] resulting from the submittal-only review and in the Site Audit Plan [6]. The supplemental RAIs are listed (*verbatim*) below:

Supplemental RAI No. 1 Soil Failure, Soil Liquefaction and Slope Instability Analyses

Experience from past earthquakes has shown soil failures, including soil liquefaction and slope instabilities, to be a significant concern. A number of issues have been raised with regard to the H. B. Robinson - Unit 2 (HBR) IPEEE soil failure and liquefaction analyses.

Of particular concern is the fact that soil liquefaction and seismically induced deformations of embankments or dams are sensitive to assumed earthquake magnitude and strong motion duration. The earthquake magnitude used for the HBR IPEEE submittal soils evaluation has not been determined to be consistent with the review level earthquake (RLE). (The selected magnitude is based on the mean magnitude for the Savannah River site.)

H. B. Robinson is a deep soil site. The submittal characterizes the top 50 ft of soil at the Robinson site as containing various beds of moderate to dense sands interspersed with layers of relatively weak to moderate strength silty sands, sandy silts, and silty clays. The submittal has indicated, from an initial iteration of liquefaction evaluation, that localized soil lenses which might be subject to liquefaction are likely to occur for a M_w 5.5 earthquake producing RLE motions; and based on an updated liquefaction analysis, a "statistically insignificant number of data points" indicated liquefaction. Thus, for an analysis based on higher magnitudes (for instance, M_w 6.0) significant liquefaction may be possible. Also, as indicated in response A.2 4, an "equivalent static" factor of safety of Lake Robinson Dam, for SSE input (0.2g), was earlier assessed as being 1.08. For RLE motions, the factor of safety would likely be below unity, with resulting transient and permanent deformations. Again, considering higher magnitudes (e.g , M_w 6.0), it is likely that significant displacements of the dam would result. Furthermore, the treatment of submerged slopes and dispersal of lake sediments was based on "extrapolation of the soil boring logs" to conclude that the lake-bed sands are not susceptible to liquefaction. However, no justification was provided in the response.

Earthquake experience has also identified seismically induced breaks of piping buried in soft soil or soils that have experienced liquefaction to be a significant concern. This possibility and the increased susceptibility associated with deterioration of piping (which the IPEEE indicates has been observed at Robinson-2 for buried service water piping), for the various possible site soil characteristics and beyond-design-basis magnitudes, have not been adequately addressed in Response A.2.3. The evaluation of the potential and effects of seismically induced failures of buried piping (e.g., service water piping and fuel oil transfer lines) founded in soil should be performed for critical (i.e., the most susceptible) soil conditions/locations subjected to Review Level Earthquake (RLE) motions.

The utility submittal concluded that there are no concerns pertaining to soil failures at Robinson-2, and that, with respect to all soil failure modes and all earth structures, a HCLPF in excess of the 0.3g RLE exists. As noted above, these findings have not yet been reasonably justified.

To insure that the investigation of soil failures (particularly for analyses of liquefaction susceptibility, dynamic instability, etc.,) is relevant to RLE motions, a consistent earthquake magnitude should be used. Alternatively, sensitivity investigations for various magnitudes (e.g., M_w 5.5, 6.0, etc.) can be performed. The magnitude and extent of liquefaction-induced soil shear strains at the plant site, within the slope and foundation materials of Lake Robinson Dam, and along submerged embankments, should be addressed based on the critical observed soil properties. Loss of soil strength and consequential reductions in lateral resistance of foundation pile systems of essential structures should be assessed. Impacts of the potential for pipe breaks and differential soil settlements/displacements on essential structures and components, for representative critical cases, should be examined. Seismic-induced stresses in buried piping, accounting for deterioration in piping materials, should also be investigated for critical soil characteristics/locations.

With this background, please provide your related findings as to whether or not there are any soil failure issues of concern for Robinson-2, with respect to the RLE. Please discuss fully the data (soil properties, seismic capability characteristics of structures or components, plant configuration, dam configuration, slope configurations, earthquake characteristics, etc.), the methodological details, and the results of your soil failure assessments which support your specific findings and conclusions.

Supplemental RAI No. 2 Capacity (HCLPF) Calculations

In your July 1, 1996 letter, you did not provide the requested information for RAI A.2.9 related to capacity calculations. Please provide HCLPF calculations, completed screening evaluation worksheets (SEWS), walkdown notes/checklists and photographs for the following components

- Motor Operated Valves RHR-750 and RHR-751
- Diesel Fuel Oil Storage Tank, RWST, CST
- Service Water Pumps
- 125 VDC MCCs A & B.

Supplemental RAI No. 3 Concern With Combined Failures Of Two Motor-Operated Valves (With Cast-Iron Yokes)

The seismic IPEEE identified a significant concern associated with potential combined failures of two low-capacity motor-operated valves (having cast-iron yokes) that may lead to an interfacing systems LOCA (ISLOCA) outside containment. An estimate of the frequency of this seismically induced ISLOCA was made in the IPEEE submittal. However, the approach used for calculating this frequency was crude and inaccurate. No specific modifications have been proposed with respect to the two motor-operated valves

The licensee originally committed to implement (by December 1998) related procedural enhancements in accordance with severe accident management guidelines. Subsequently, it was apparently determined that such procedural enhancements were unnecessary.

Please identify and justify what actions, if any, will be taken to mitigate the ISLOCA concern. If procedural actions are being considered, evaluate the effectiveness of these actions. Identify what operator actions would be required, where the actions would need to take place, and the failure rates associated with such actions (in consideration of the potential for seismically induced failures that may interfere with such actions).

Supplemental RAI No. 4 Containment Walkdown Results

In your July 1, 1996 letter, Response A.2.6 states "We will perform a walkdown of the HBRSEP, Unit No. 2 containment heat removal systems and their anchorages, including the fan coolers, and report the findings." Please submit this report, and describe the walkdown approach and the observed configuration of containment heat removal systems.

3.0 SITE VISIT

The site visit took place on Sept 23-24, 1998 The audit team consisted of Alan M Rubin, John T. Chen, Roger M. Kenneally (US NRC/RES) and Michael P. Bohn (Sandia National Laboratories). The principal Carolina Power and Light (CP&L) contacts were Peter Yandow (CP&L/Reg. Affairs), James Paul (CP&L/RESS) and Ron Knott (CP&L/CES). CP&L consultants were Doug Honegger (self-employed consultant) and Robin McGuire (EQE Inc). The site visit included:

- An entrance/orientation meeting with CP&L plant management and staff to discuss the site audit objectives, approaches and specific needs,
- An onsite audit of "Tier 2" IPEEE information,
- Discussions with licensee personnel and licensee contractors familiar with the various facets of the issues being reviewed,
- A plant walkdown to review and evaluate the appropriateness of IPEEE screening decisions, modeling assumptions, and identification of potential plant improvements,
- Identification of specific confirmatory information required to document the rationale for resolution of the remaining open issues, and
- An exit meeting with CP&L plant management and staff summarizing the results of the site audit.

As part of the visit, the licensee provided a well-organized notebook (Reference 8) containing the following documentation (requested in the Site Audit Plan) in support of the site audit of the H. B. Robinson (Unit 2) seismic IPEEE:

a) Written response to NRC Supplemental RAI No. 1 on soil failure issues,

b) Screening evaluation work sheets (SEWS) and engineering capacity calculation sheets for:

   - Motor Operated Valves RHR-750 and RHR-751
   - Diesel Fuel Oil Storage Tank, RWST, CST
   - Service Water Pumps
   - 125 VDC MCCs A & B,

c) Photos of all components except valves,

d) Letter describing the seismic IPEEE containment walkdown findings (Reference 9)

During the visit, a seismic walkdown was performed which included all the above items except Motor Operated Valves RHR-750 and RHR-751 (which were in containment and not accessible). In addition, the walkdown included a general tour of the site and buildings, with emphasis on the Rad Waste building.

4.0 TECHNICAL FINDINGS

4.1 Soil Failure, Soil Liquefaction and Slope Instability Analyses

With reference to Supplemental RAI No. 1, it can be seen that there are essentially three issues which must be addressed:

(a) Selection of characteristic earthquake magnitude corresponding to the 0 3g RLE

(b) Evaluation of the potential for liquefaction at different locations on the site

(c) Evaluation of the impact of any liquefaction-induced displacements on structures or piping

These issues are discussed below for the H. B. Robinson site.

As will be seen, the H. B. Robinson IPEEE soil failure analysis departed from the (NRC approved) approach presented in EPRI 6041 (Ref. 10) in one important aspect (selection of the Magnitude Scaling Factors, which significantly reduced the calculated likelihood of liquefaction). During the site visit, CP&L consultants justified use of the more recent (and less conservative) data based on the findings of a 1997 workshop involving 20 well-known practitioners in the area of liquefaction prediction. The results of this workshop, which includes a consensus set of recommendations for a state of the art assessment of soil liquefaction, are published in the **Proceedings of the 1997 NCEER Workshop on Evaluation of Liquefaction Resistance of Soils** (Reference 11) sponsored by the National Center for Earthquake Engineering Research (now the Multidisciplinary Center for Earthquake Engineering Research). Inasmuch as this 1997 workshop will be referenced frequently, it will be referred to as the "1997 NCEER Workshop Proceedings" below.

4.1.1 Earthquake Magnitude Selection for Soils Evaluations

As discussed in the initial IPEEE submittal and response to the initial RAIs, the determination of the appropriate mean magnitude (corresponding to a 0.3g RLE at a 10^{-4} exceedance frequency) was based on a study for the Savannah River site (Ref. 12). (This was felt appropriate since it was judged that the Robinson site was in the same tectonic province as the Savannah River site.) The study, which was based on the 1993 LLNL hazard analyses for the central and eastern United States (Ref. 13 and 14), indicated a mean magnitude of 5.9 m_b. Using the corresponding EPRI hazard study (Ref. 15), a very similar value of 5.8 m_b was obtained. In their response to the supplemental set of RAIs, additional arguments were presented based on the more recent NUREG/CR-6606 study of Bernreuter *et al* (Ref. 16) which included hazard deaggregation results specific to the Robinson site. Based on these studies, CP&L judged that the appropriate mean earthquake magnitude to use in their liquefaction analyses was 5.9 m_b (where m_b was defined as the short-period body-wave magnitude).

[Note that this reviewer did not have access to either the Savannah River site study or the recent NUREG/CR-6606 study during this review. **It is recommended that NRC Geosciences Branch personnel review these two studies and the validity of the conclusions drawn as to the appropriate mean magnitude to use at Robinson site.]**

As stated in the RAI response dated September 10, 1998 (page 1),

> "The mean magnitude from this study was 5.9 m_b (short-period body-wave magnitude). To be consistent with the magnitude measure required for the ground-failure analyses, this magnitude was converted to an equivalent moment magnitude of 5.5 M_w using three published relationships relating M_w to m_b."

The result that a short period body wave magnitude value of 5.9 m_b corresponds to a **(smaller)** moment magnitude of 5.5 M_w is opposite to that which would result from commonly-used conversion relations. Figure 1 shows a plot of the relationship between the moment magnitude M_w and other magnitude scales due to Heaton *et al.* (1986, Reference 17) which is recommended by the 1997 NCEER Workshop Proceedings and also presented in **Earthquake Hazard Analysis: Issues and Insights**, by L. Reiter, 1990 (Reference 18). This figure shows that, for all moment magnitudes greater than 5.0 M_w, the moment magnitude M_w **is greater than** the corresponding short period body wave magnitude m_b. From this figure, a short period body wave magnitude of 5.9 m_b corresponds to a moment magnitude of 6.3 M_w.

It should be noted that the mean magnitude chosen for use in seismic liquefaction analyses is crucial. From the point of view of the actual observational experience, there are only limited observations of liquefaction for earthquakes with magnitudes around 5.5 M_w. [That is, in the database from which the liquefaction/no liquefaction boundary shown in Figure 2 was constructed (Ref. 21), only eight data points (out of 125 total) came from 5.5 M_w earthquakes (These were the 1957 San Francisco earthquake and the 1981 Westmoreland earthquake)]. By contrast, there are a significant number of data points (observed liquefaction occurrences) due to earthquakes in the range 6.0 – 6.7 M_w.

Conclusion: The moment magnitude value (5.5 M_w) used for the H. B. Robinson IPEEE soil failure assessment seems to be based on an incorrect magnitude conversion. The appropriate conversion leads to a moment magnitude between 6.0 and 6.5 M_w. This has a significant effect on prediction of regions of potential liquefaction and magnitude of the resulting soil displacements.

4.1.2 Identification of Regions of Potential Liquefaction

Soil liquefaction is a condition wherein the local pore water pressure in the soil is sufficiently great so that it overcomes the pressure due to the soil column above, and the soil loses its load-bearing strength. As a column of sand or silty sand is vibrated, laboratory tests show that the pore water pressure increases with the number of cycles of shaking. After a sufficient number of cycles, the pore water pressure exceeds the overburden pressure and a liquefaction state exists. Fundamentally then, the potential for

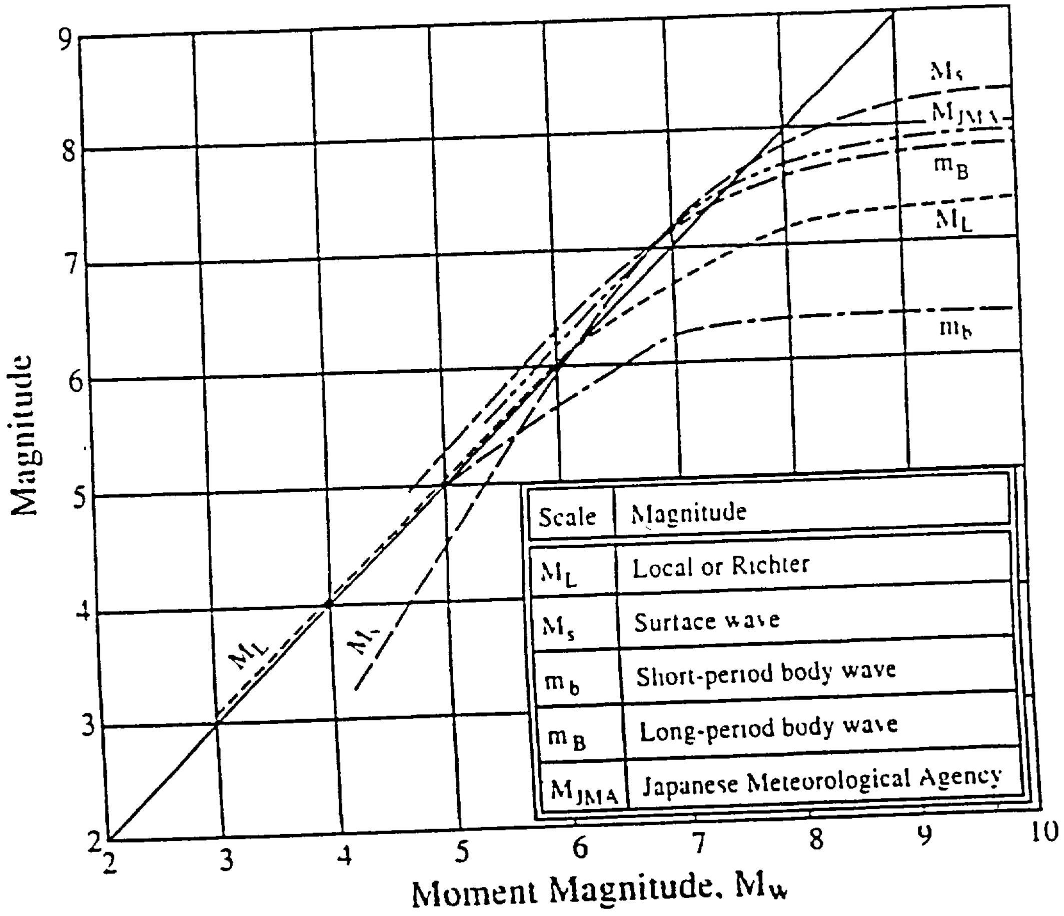


Figure 1 Relationships Between Moment Magnitude and Various Magnitude Scales (After Heaton *et al*, 1984)

liquefaction is determined by the magnitude of the soil shaking, the effective overburden pressure, and the number of cycles of vibration.

Early laboratory tests showed that the limits of liquefaction could be expressed in terms of the Cyclic Stress Ratio (CSR) defined as the ratio of the average maximum horizontal cyclic shear stress (at the depth in question) to the effective vertical overburden pressure:

$$CSR = \frac{\tau_{hv}}{(\sigma_o)_{effective}}$$

The cyclic stress ratio includes the magnitude of the soil shaking and the effective overburden pressure (which is a function of the soil column density, water density, depth of soil column, and depth of water table). Indeed, early laboratory studies of soil samples taken in regions which had experienced a specific earthquake led to correlations between the CSR and the number of cycles of shaking for sands that had liquefied and those that had not liquefied.

However, as more strong motion recording instruments came in the use, and data became available on the properties of soils where liquefaction was or was not observed due to different earthquakes, it became possible to develop a correlation between the CSR and the local soil properties, namely,

- relative density
- soil structure
- soil grain cementation (aging effect)
- lateral earth pressure

and that these factors affected the cyclic loading characteristics of the soil and the penetration resistance of the soil (as measured in a standard penetration test) in the same general way (Reference 19). Specifically, it was concluded that, for a given earthquake magnitude, a meaningful bounding correlation predicting the occurrence of liquefaction could be generated in terms of the CSR and the blow count obtained from a standard penetration test (SPT) at the location (and depth) in question. Figure 2 (from Reference 20) shows this correlation and the data on which it is based for magnitude 7 ½ earthquakes and for clean sands. (This figure is the same as Figure 9 in Appendix C of EPRI 6041). The data points are segregated into points for which liquefaction was observed (solid points), points for which marginal liquefaction was observed (partially filled points), and points for which no liquefaction was observed (open points). The solid line shown can be seen to represent a lower bound to the points for which some evidence of liquefaction was observed.

The ordinate on this plot is the Cyclic Stress Ratio described above, while the abscissa is $(N_1)_{60}$, the (modified) blow count required to drive the penetrator 12 inches (at the depth in question) obtained from a Standard Penetration Test. This figure is thus the basis for determining whether or not liquefaction is likely for magnitude 7 ½ earthquakes at some arbitrary location (and depth) consisting of clean sands (Reference 21). That is, if the

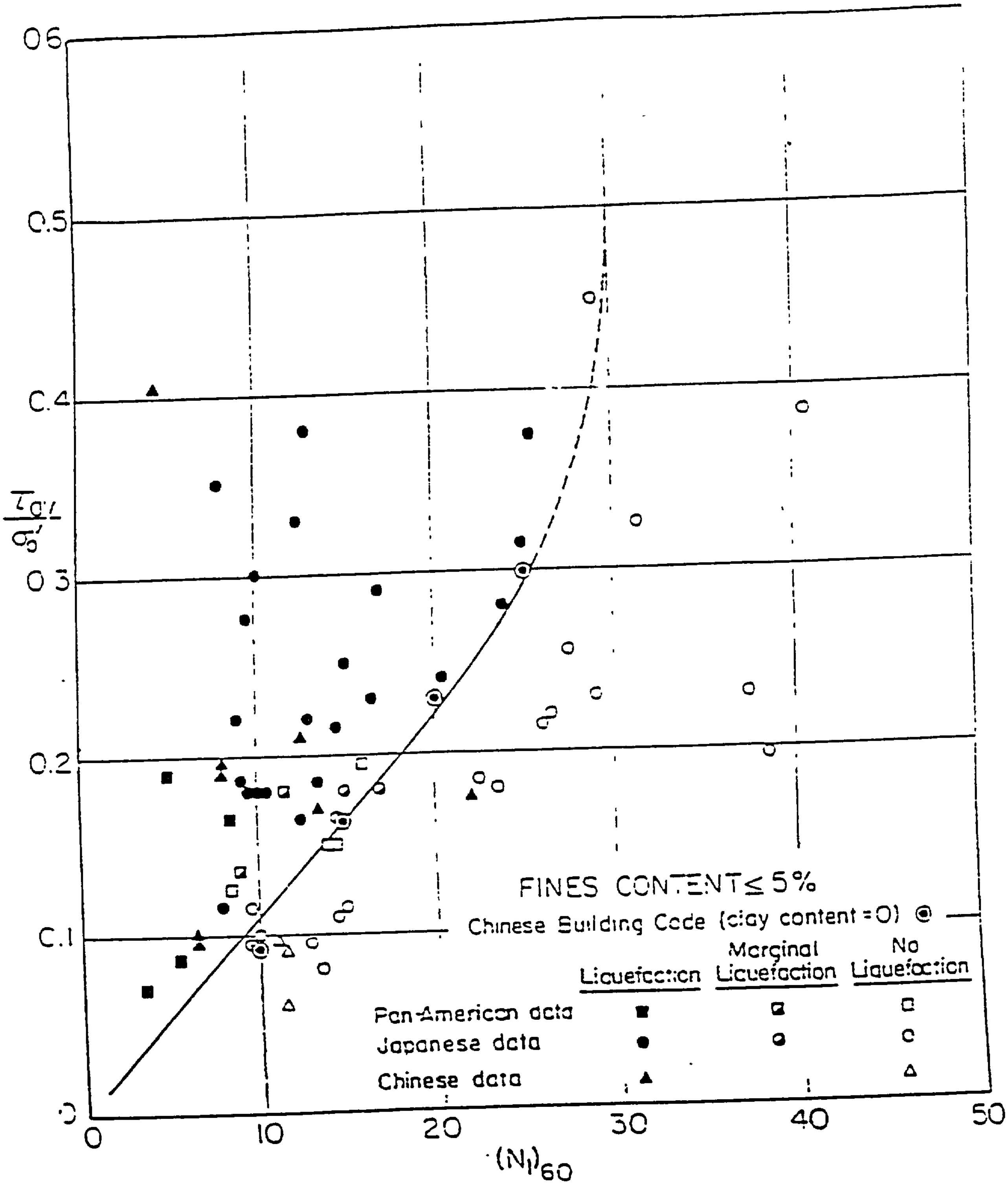


Figure 2 Relationship between stress ratios causing liquefaction and N_1 values for clean sands for M = 7 ½ earthquakes

CSR and the modified blow count $(N_1)_{60}$ are computed for the site (and depth) in question, and if the plotted point falls above the solid line in this figure, then the possibility of liquefaction would be presumed.

As noted above, Figure 2 is for clean sands. Field observation and laboratory testing show that liquefaction is less likely in silty sands. Figure 3 shows the same type of plot when data including silty sands (of different fines content) are included. Lower bound correlation curves (separating points with observed liquefaction and no liquefaction) are shown for soils of different degrees of siltyness (different fines content). (Figure 3 is identical to Figure 10 in Appendix C of EPRI 6041). As with Figure 2, Figure 3 applies only for magnitude 7 ½ earthquakes, but it is used in the same way. [Note that the bounding curves for increasing fines content are nearly parallel with the bounding curve. for clean sands (fines content < 5%). This observation forms the basis for one method of correcting the blow count values to account for silty sands]

Calculation of CSR and $(N_1)_{60}$

As described above, the CSR is computed from the effective maximum horizontal shear stress at the depth in question and from the effective overburden pressure acting on the soil element being considered. The effective maximum horizontal shear stress can be computed from dynamic soil analyses. Alternatively, a simple expression was derived by Seed (Ref. 19) given by:

$$\tau_{hv} = 0.65\ pga * \gamma_{soil} * h * r_d / g$$

where

τ_{hv} = effective maximum horizontal shear stress
pga = local peak ground acceleration
γ_{soil} = weight density of the soil
h = depth of soil element under consideration
r_d = correction factor to account for simplified model of dynamic soil column response used in developing this expression for soil shear stress at depth (shown in Figure 4)

In this expression, $\gamma_{soil} * h$ is the total weight of a column of soil (of unit area) of height h, assuming uniform soil density. Under the same assumptions, $\gamma_{soil} * h$ also represents the vertical overburden pressure σ_o at depth h, so the equation for τ_{hv} is often written as

$$\tau_{hv} = 0.65\ pga * \sigma_o * r_d / g$$

The effective vertical overburden pressure, which includes the depth and density of the soil column and the level of the water table, is given by:

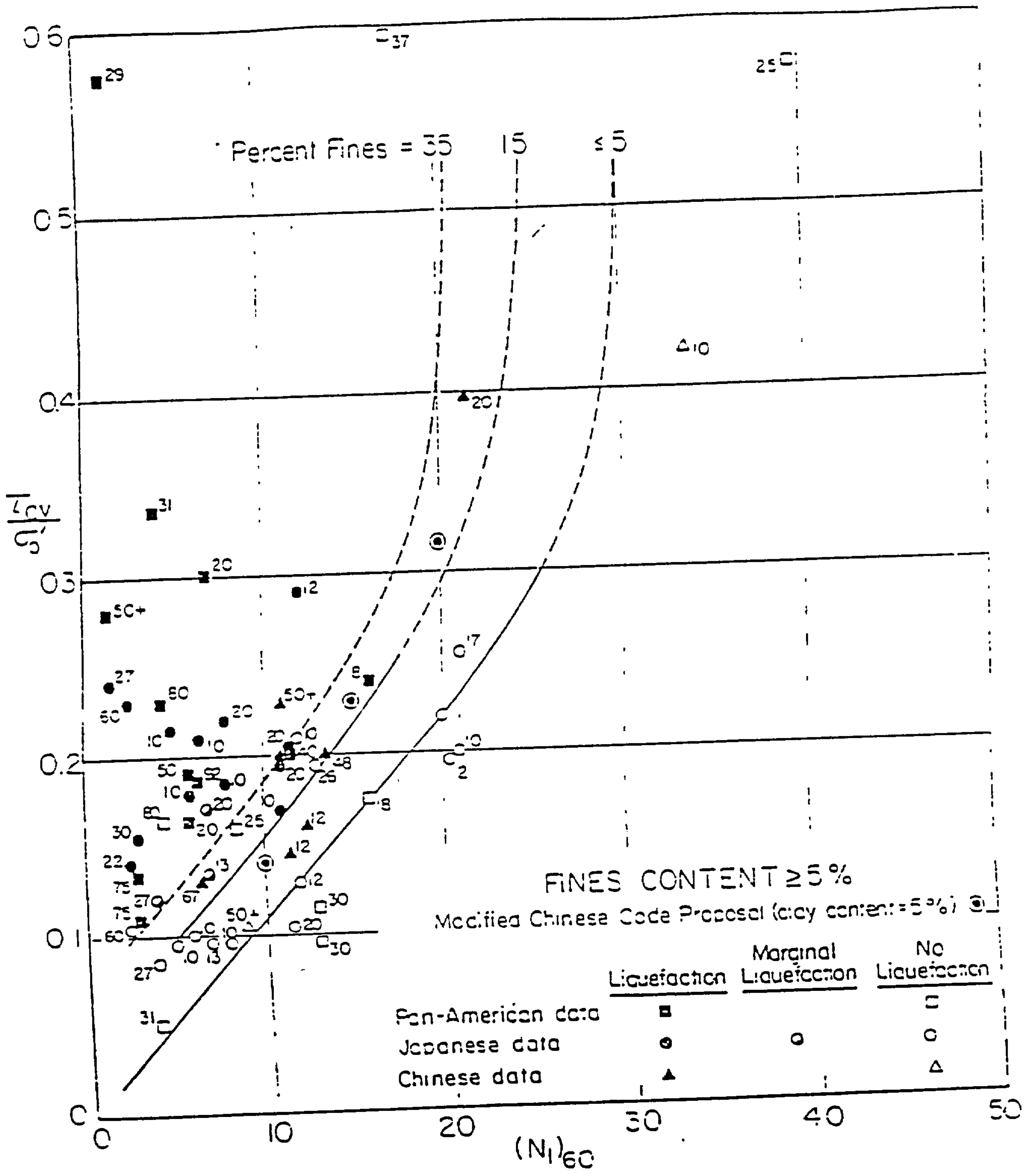
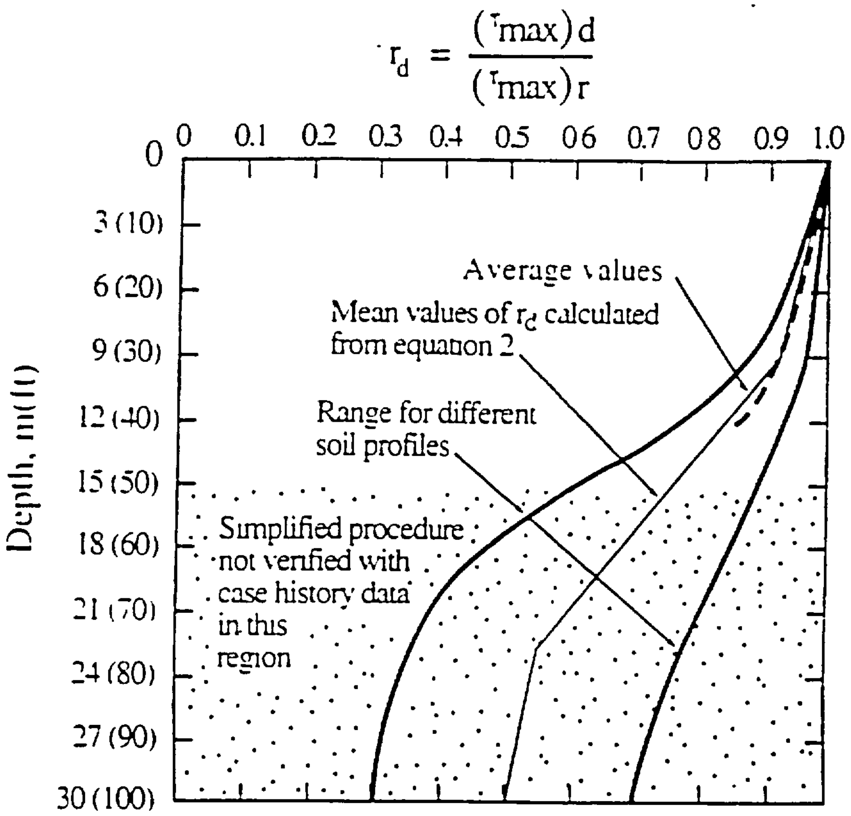


Figure 3 Relationship between stress ratios causing liquefaction and N_1 values for silty sands for M = 7 ½ earthquake



Dynamic Soil Response Correction Factor	Range
$r_d = 1.0 - 0.00765*z$	for $z \leq 9.15$ m
$r_d = 1.174 - 0.0267*z$	for $9.15 < z \leq 23.0$ m
$r_d = 0.744 - 0.008*z$	for $23.0 < z \leq 30.0$ m
$r_d = 0.50$	for $z > 30.0$ m

Figure 4 Dynamic soil response correction factor r_d versus depth (from Seed and Idriss, 1971 taken from Ref. 11) and equations recommended in Ref. 11

$$(\sigma_o)_{effective} = \gamma_{soil} * h - (h - h_{wt}) * \gamma_{water}$$

$$= \gamma_{soil} * h_{wt} + (h - h_{wt}) * (\gamma_{soil} - \gamma_{water})$$

where

$(\sigma_o)_{effective}$ = effective vertical overburden pressure
γ_{soil} = weight density of soil
γ_{water} = weight density of water
h = depth of soil element under consideration
h_{wt} = depth of water table under ground surface

Hence the expression for the cyclic stress ratio is often written as

$$CSR = \tau_{hv}/(\sigma_o)_{effective} = 0.65\ (pga/g)*[\sigma_o/(\sigma_o)_{effective}]*r_d$$

The cyclic stress ratio is a measure of the **demand** (load) which may or may not cause liquefaction.

The **resistance** to liquefaction is measured in terms of the soil's penetration resistance in terms of "blow counts" observed in a Standard Penetration Test (SPT). The modified blow count $(N_1)_{60}$ is obtained from the raw blow count measured in a Standard Penetration Test by normalizing it to an overburden pressure of 1 ton/ft^2 and then correcting the blow count for nonstandard test conditions. The correction factors listed in Appendix C of EPRI 6041 (as taken from Seed *et al*, Ref. 21) are as follows:

$$(N_1)_{60} = N*(ER/60)*C_N*C_S*C_Z$$

where

$(N_1)_{60}$ = modified blow count normalized to 1 tsf overburden pressure

N = raw blow count from the SPT

ER = energy ratio expressed as a percent of theoretical free fall energy delivered to the drill system in performing the SPT:

Safety hammer with 2 rope wraps around the pulley, ER = 45%
Donut hammer with 2 rope wraps around the pulley, ER = 60%

C_N = correction factor for overburden pressure (see Figure 5)

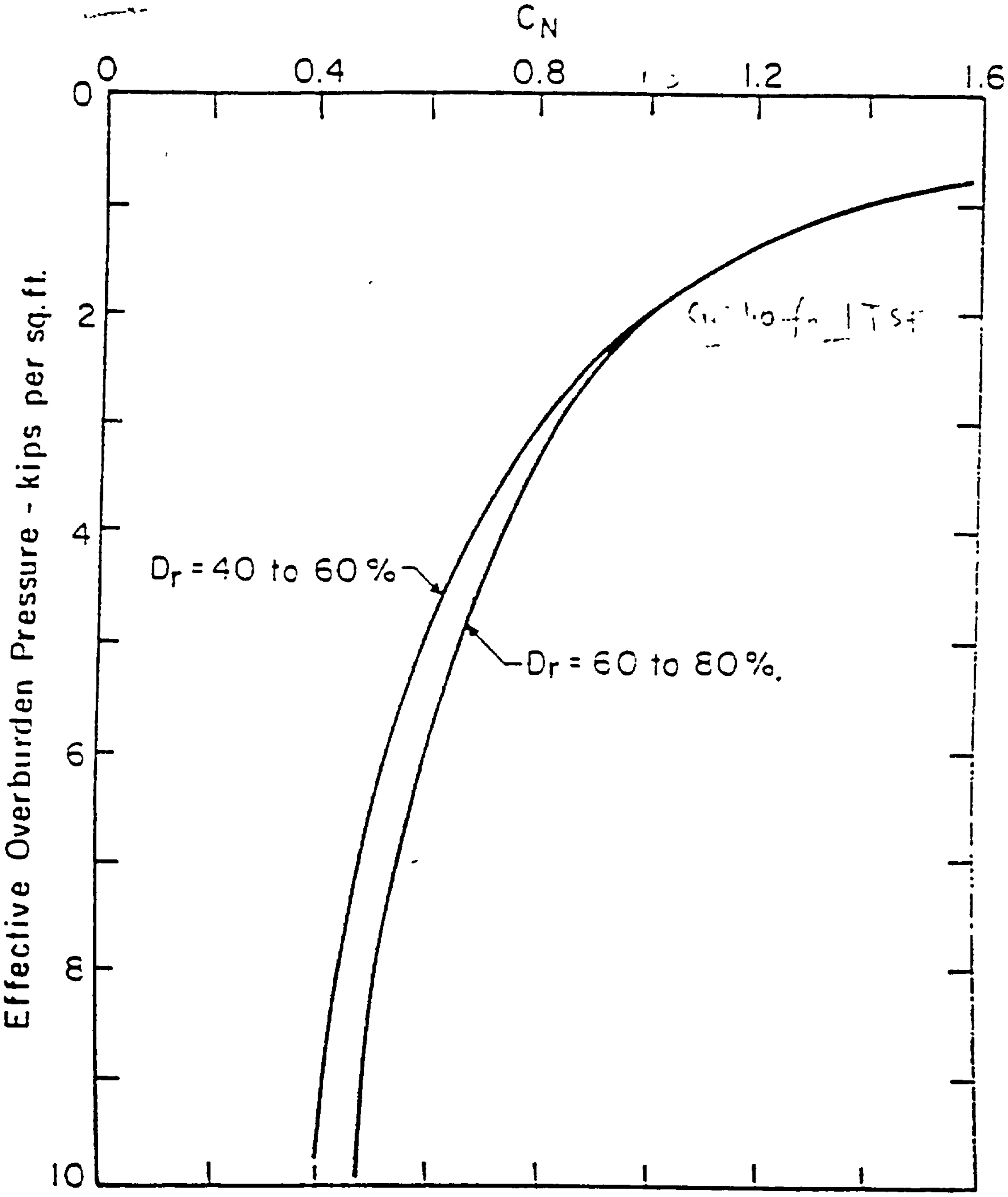


Figure 5 Overburden Correction Factor C_N (from Ref 21)

C_S = correction factor for type of sampling tube For a split spoon sampler without liner (with ID= 1.5 in, OD = 2.0 in).

For N< 10, C_S = 1.1
For N≥ 10, C_S = 1.2

C_Z = correction factor for the length of the drill rod used in the SPT:

For h<10 ft, C_Z = 0.75
For h>10 ft, C_Z = 1.00

The above factors are those given in EPRI-6041. The numerical correction factors which were used in the H. B. Robinson IPEEE liquefaction analyses were not given.

Incorporation of Silt Content (Fines Count)

As noted above in regards to Figure 3, the liquefaction/no liquefaction boundaries for silty sands (of different fines content, denoted FC) are roughly parallel to the boundary for clean sands This is typically accounted for by adding a correction factor to the modified (corrected) blow count $(N_1)_{60}$ to obtain an equivalent clean sand blow count value and then using the clean sand liquefaction/no liquefaction boundary for determination of the liquefaction potential. The original set of correction factors recommended by Seed (Ref. 22) are shown below:

Fines Content (%)	Blow Count Correction
10	1
25	2
50	4
75	5

The 1997 NCEER Workshop Proceedings (p. 63) recommends a correction in the form of

$$(N_1)_{60,cs} - \alpha + \beta^* (N_1)_{60}$$

where

$(N_1)_{60,cs}$ = equivalent clean sands blow count

$(N_1)_{60}$ = SPT (corrected) blow count taken in silty sands

FC = Fines content in %

and the two coefficients are given below:

Coefficient α	Coefficient β	Range of FC (%)
$\alpha = 0.0$	$\beta = 1.0$	$FC \leq 5\%$
$\alpha = \exp[1.76 - (190/FC^2)]$	$\beta = \exp[0.99 + (FC^{1.5}/1000)]$	$5\% < FC < 35\%$
$\alpha = 5.0$	$\beta = 1.2$	$FC \geq 35\%$

Quoting Reference 23, the Robinson IPEEE submittal used additive correction factors of 2.5 "for sands identified as SP on the boring logs (consistent with a fines content of 10 %) and 5 "for sands identified as SM on the boring logs (consistent with a fines content of 20%)".

A comparison of the equivalent clean sand blow counts given by the Seed correction model, the 1997 NCEER Workshop Proceedings correction model, and the model used by CP&L is shown on Figure 6. It can be seen that the values used for Robinson are somewhat higher than those of Seed and NCEER at a fines content of 10%, but the values are close to those of NCEER at a fines content of 20%. Thus the Robinson equivalent clean sands blow counts are somewhat non-conservative with respect to the other two models, especially for measured blow counts less than 10 (which is when liquefaction might be expected).

Consideration of Magnitudes Other Than 7 ½

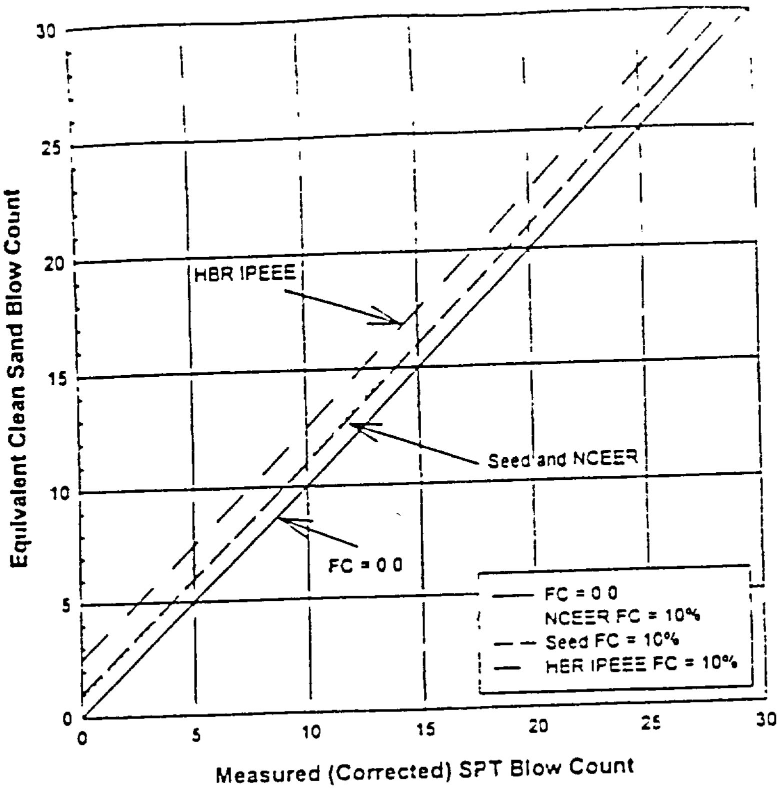
In the original Seed and Idriss (1982) approach described in EPRI 6041, modifications to account for different earthquake magnitudes are accomplished by dividing the CSR ratio by a magnitude scaling factor (MSF). EPRI 6041 provides the original MSF values derived by Seed and Idriss (1982). These are shown in Column 2 on Table 1 (from the 1997 NCEER Workshop Proceedings). As stated in the Summary Report section of these Proceedings (p. 29),

> "The workshop participants reviewed the MSF (values) listed in Table 1 and all but one (S S.C. Liao) agreed that the original factors (**nb.** of Seed and Idriss, 1982) were too conservative and that an increase is warranted for engineering practice for magnitudes less than 7.5."

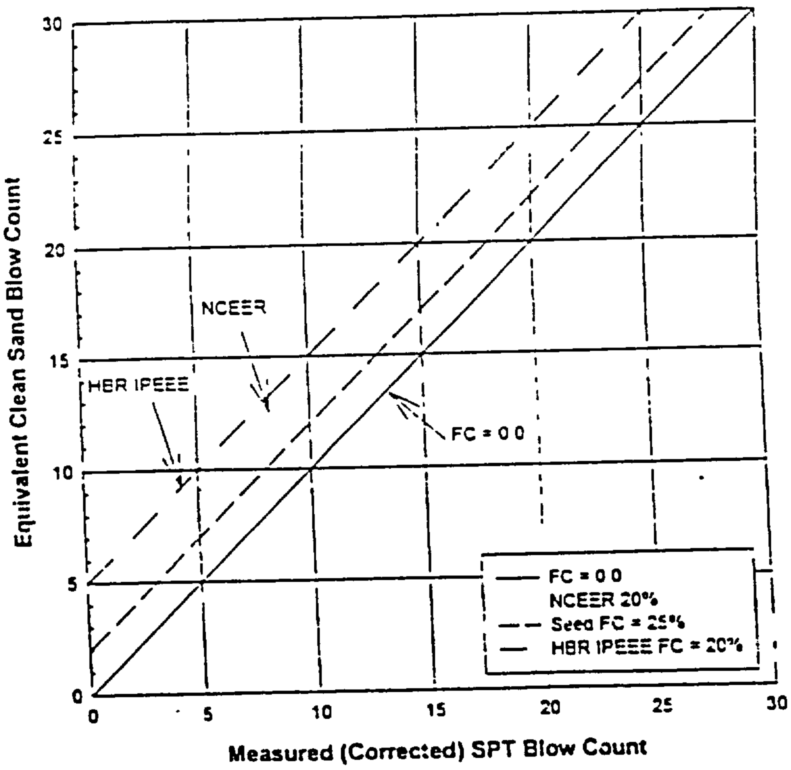
Based on a detailed review of the state of the art, **the workshop participants recommended a range of magnitude scaling factors** for performing liquefaction assessments as follows:

- "For magnitudes less than 7.5, the lower bound for the recommended range is the revised set of magnitude scaling factors proposed by Idriss (Column 3, Table 1). The upper bound for the suggested range is the MSF proposed by Andrus and Stokoe (Column 7, Table 1).

  For magnitudes greater than 7.5, the factors recommended by Idriss (Column 3, Table 1) should be used for engineering practice."



(a) FC = 10%



(b) FC = 20%

Figure 6 Comparison between Robinson IPEEE Fines Content (FC) corrected blow counts and those using Seed and NCEER correction models

Table 1 Magnitude Scaling Factors Defined by Various Investigators

Mw (1)	Seed and Idriss (1982) (2)	Idriss revised (3)	Ambraseys (1988) (4)	Arango (1996) (5)	 (6)	Andrus and Stokoe (this report) (7)	Youd and Noble (herein) P_L<20% (8)	 P_L<32% (9)	 P_L<50% (10)
5.5	1.43	2.20	2.86	3.00		2.8	2.86	3.45	4.44
6.0	1.32	1.76	2.20	2.00	1.65	2.1	1.93	2.35	2.92
6.5	1.19	1.44	1.69			1.6	1.34	1.65	1.99
7.0	1.08	1.19	1.30	1.25		1.25	0.96	1.19	1.39
7.5	1.00	1.00	1.00	1.00	1.00	1.0	0.70[1]	0.88[2]	1.00
8.0	0.94	0.84	0.67	0.75		0.8			0.73[2]
8.5	0.89	0.72	0.44		0.76	0.65			0.56[2]

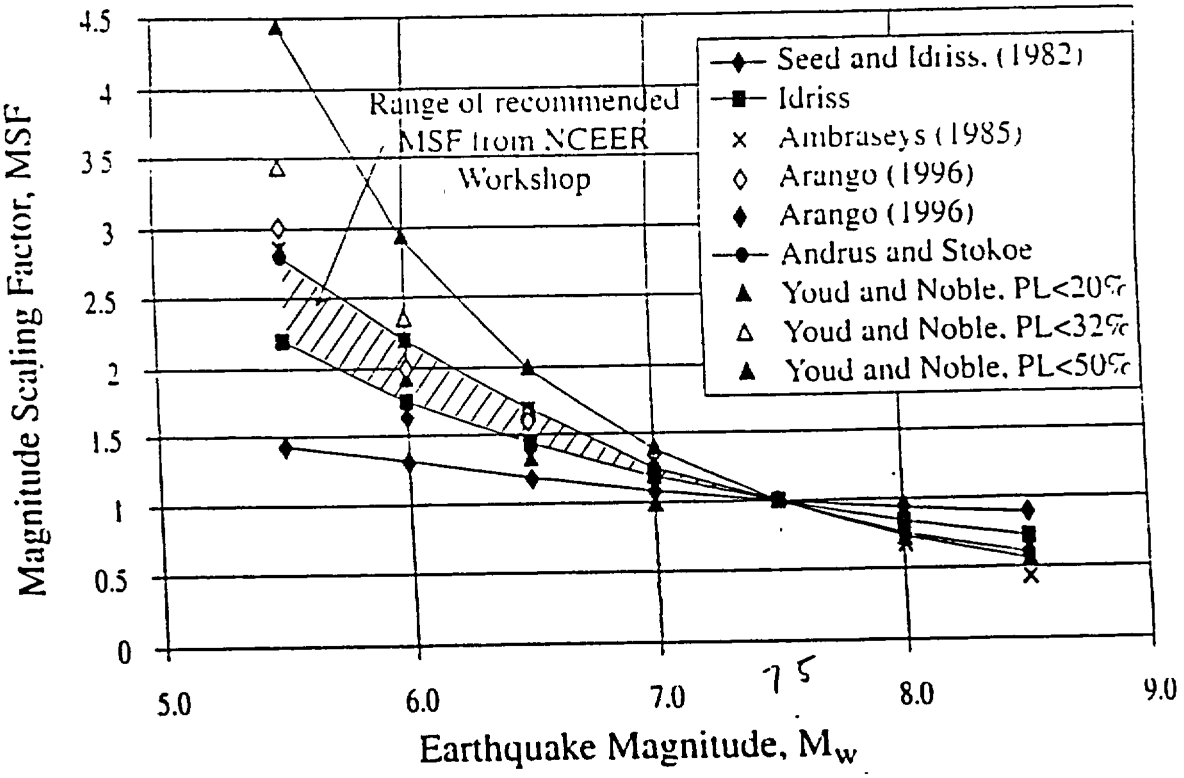


Figure 7 Magnitude Scaling Factors Defined by Various Investigators

This recommended range of values is shown (cross-hatched) on Figure 7 (taken from the1997 NCEER Workshop Proceedings).

In addition to the MSF values mentioned above, Columns 8, 9, and 10 of Table 1 give the magnitude scaling factors derived using statistical regression by Youd and Noble, as presented in the article "Liquefaction Criteria Based on Statistical and Probabilistic Analyses", by Leslie Youd and Steven Noble, ppg. 201 of the same 1997 NCEER Workshop Proceedings. As described this article, the Youd and Noble MSF values are a re-evaluation of the factors derived by Loertscher and Youd (1994), Reference 24. As stated in this article,

> "Youd and Noble (herein) corrected some inconsistencies in the 1994 Loertscher and Youd data set, and re-analyzed the data to verify the results of Loertscher and Youd".

A comparison between the 1994 Loertscher and Youd MSF values and the (revised) 1997 Youd and Noble MSF values (at PL < 32%) is shown below:

Magnitude	1997 Youd and Noble MSF	1994 Loertscher and Youd MSF
5.5	3.45	4.46
6.0	2.35	2.79
6 5	1.65	1.88
7.0	1.19	1.34

It it can be seen that the revised (1997) Youd and Noble values are significantly reduced from the original (1994) Loertscher and Youd values.

The magnitude scaling factor used for the Robinson IPEEE soil liquefaction analyses at 5.5 M_w was (evidently) 4.46. (According to the CP&L RAI responses, their liquefaction analyses used the value recommended in the 1994 Loertscher and Youd paper. Although the 4.46 value was not explicitly listed in the IPEEE submittal, it is the "recommended" value at 5.5 M_w in the Loertscher and Youd (1994) paper and, in addition, this value can be "backed out" of the IPEEE liquefaction results that were presented.)

However, continuing to quote from the same Youd and Noble article (page 202) in the 1997 NCEER Workshop Proceedings,

> "The analyses and results reported herein were developed after the formal workshop event, and hence were not discussed or approved during the workshop discussions".

Further, as stated on page 207 (same article),

> "In general, however, application of probabilistic analysis is beyond the normal practice of most technical engineers. *Hence the workshop participants did not approve recommendations (nb. of Youd and Noble) for engineering practice, but did encourage continued development of these concepts"*.

The significance is that the 1994 Loertscher and Youd magnitude scaling factors used in the H. B. Robinson IPEEE evaluation of liquefaction are primarily responsible for the conclusion that liquefaction will not occur at the H. B. Robinson site for a 0.3g RLE. These values were *not* recommended by the participants in the 1997 NCEER Workshop, and are significantly *non-conservative* when compared to the EPRI 6041 values or the values recommended in the 1997 NCEER Workshop Proceedings.

The Youd and Noble (1997) magnitude scaling factors given in Table 1 are also plotted on Figure 7. It can be seen that all the (probabilistically-based) MSF values of Youd and Noble fall in a range significantly above the recommended range of magnitude scaling factors. *Use of ~~the~~ either the Loertscher and Youd (1994) or the Youd and Noble(1997) values will significantly lower the liquefaction demand parameter CSR (since the correction for different magnitudes is obtained by dividing the computed CSR value by the magnitude scaling factor) and thus predict significantly less liquefaction at the H. B. Robinson site.*

As noted on Table 1, the various magnitude scaling factors in the 1994 Loertscher and Youd paper and the Youd and Noble article in the 1997 NCEER Workshop Proceedings were derived on a probabilistic basis, and the probability that liquefaction might occur is listed as $P_L < 32\%$ or $P_L < 50\%$, etc. for the different MSF values presented. The 1994 Loertscher and Youd value of MSF = 4.46 evidently used in the IPEEEE soil analyses corresponds to a 32% probability that liquefaction would occur using the liquefaction/no liquefaction boundary resulting from that MSF value. In keeping with the intent and spirit of an "IPEEE Margins Assessment", wherein it is desired to obtain a "High Confidence of Low Probability of Failure", it would be much more appropriate to have used a bound corresponding to $P_L < 5\%$ or less, which would be nearly a lower bound.

?

For comparison with the IPEEE submittal results (for a magnitude 5.5 earthquake), the EPRI 6041 magnitude scaling factor value [original Seed and Idriss (1982)] is 1.43 and the 1997 NCEER Workshop Proceedings recommended lower bound (Idriss revised) value is 2.2. Use of either of these MSF values would result in prediction of some liquefaction at many points in the soil columns at the Robinson site.

Check that

For a magnitude of 6.0 M_w earthquake (see Section 4.1.1 above), the corresponding magnitude scaling factor from EPRI 6041 is 1.32, and 1997 NCEER Workshop Proceedings [Idriss (revised)] recommended value is 1.76. Using these factors, even more regions of liquefaction in the site soil columns would be predicted, as shown below

Columns 1 through 7 of Table 2 list the measured (and corrected) SPT blow counts versus depth for borings 101, 110, 113, RL4, and RL5 as provided by CP&L in the February 10, 1999 RAI response. Column 8 lists the critical blow count required for no liquefaction also provided by CP&L (for 5.5 M_w using the 1994 Loertscher and Youd magnitude scaling factor). Column 9 ~~lists the~~ lists the critical blow count required for no liquefaction derived in this review for 5.5 M_w using the EPRI-6041 recommended magnitude scaling factors (i.e., those of Seed and Idriss, 1982, herein shown on Table 1). Columns 10 and 11 list the critical blow count required for no liquefaction derived using the lower and upper 1997 NCEER Workshop Proceedings recommended magnitude scaling factors, respectively. Similarly, columns 12 through 14 give the critical blow counts derived from the EPRI-6041 and 1997 NCEER Workshop Proceedings recommended magnitude scaling factors for a 6.0 M_w earthquake (again, as derived in this review). The data in this table is plotted on Figure 8.

Figure 8 is a copy of the CP&L liquefaction results presented in support of the H. B. Robinson IPEEE based on the site borings made during plan design and construction. The lowest curve on this figure denotes the liquefaction/no liquefaction boundary obtained by CP&L based on a magnitude of 5.5 M_w and (evidently) assumed uniform values of soil density and water table depth (values not provided). The data points plotted vs. depth are the corrected SPT blow count values for clean sands normalized to 1 tsf. In this plot, data points lying below the boundary line correspond to points of potential liquefaction.

For comparison, I have added four lines to this CP&L plot. The top two curves give the liquefaction/no liquefaction boundaries obtained using the Seed and Idriss (1982) magnitude scaling factors for earthquake magnitudes 5.5 and 6.0 M_w. (These are the boundaries obtained if the EPRI 6041 approach is followed.). The middle pair of curves give the liquefaction/no liquefaction boundaries based on the Idriss (revised) magnitude scaling factors for earthquake magnitudes 5.5 and 6.0 M_w. (These are the boundaries obtained using the lower bound magnitude scaling factors recommended by the participants in the 1997 NCEER Workshop Proceedings.) As can be seen, for either the boundaries based on the EPRI 6041 MSF values or the boundaries based on the 1997 NCEER Workshop Proceedings recommended MSF values, many of the CP&L measured SPT blow count points would fall below the boundaries, and thus would be points of predicted liquefaction. This is true at either magnitude 5.5 or 6.0 M_w, although, of course, more points of liquefaction are predicted with the magnitude 6.0 M_w curves.

In regard to Table 2 and Figure 8, it should be noted that, regardless of the measured blow counts, liquefaction will not occur in a soil layer that is inherently not susceptible to liquefaction. That is, some of the points showing a measured blow count less than the blow count required for no liquefaction are not, in fact, indicative of liquefaction. This is because the soil layer in which the blow count was taken consists of clay or gravel (or a sufficient mix of sand, clay and/or gravel) so as not to be liquefiable.

?

not clear

Table 2

Comparison Between Robinson-Unit 2 Site Measured (Corrected) Blow Counts and Blow Counts for No Liquefaction Using CP&L IPEEE, EPRI and 97NCEER Magnitude Scaling Factors

	MEASURED (CORRECTED) BLOWCOUNTS						BLOWCOUNTS NEEDED FOR NO LIQUEFACTION						
	BORING NUMBER						***** MAGNITUDE 5 5 Mw*****				***** MAGNITUD. 6 0 Mw******		
Depth (ft)	101	110	113	114	RL5	RL4	CP&L	EPRI 6041	NCEER97	NCEER97	EPRI 6041	NCEER97	NCEER97
									Lower Bound	Upper Bound		Lower Bound	Upper Bound
(Col 1)	(Col 2)	(Col 3)	(Col 4)	(Col 5)	(Col 6)	(Col 7)	(Col 8)	(Col 9)	(Col 10)	(Col 11)	(Col 12)	(Col 13)	(Col 14)
2			7 8	9 9			4 0	12 5	7 9	5 6	13 6	10 2	8 4
3	9 5	6.1					4.7	15 2	9 8	7 5	16 5	12 3	10 3
4						4 2	5 2	17 0	11 0	8 5	18 4	13 8	11 5
5			6 0		6 2		5 6	18 3	11 8	9 2	19 8	14 8	12 4
6				3 7			5.9	19 2	12 4	9 7	20 7	15 6	13 0
7		3.7	4 6				6.1	19 9	12 9	10 1	21 4	16 2	13 5
8		7.3					6 2	20 4	13 3	10 4	22 0	16 6	13 9
9				16 0	6 0		6.3	20 8	13 6	10 7	22 4	17 0	14 2
10						7 4	6.4	21 2	13 8	10 8	22 7	17 3	14 5
11	13 0		7.9				6 5	21 5	14 0	11 0	23 0	17 6	14 7
12		6 5					6 6	21 7	14 2	11 1	23 2	17 8	14 9
13							6 6	21 9	14 3	11 2	23 4	18 0	15 0
14			16 0	8 3		11 0	6.7	22 0	14 4	11 3	23 5	18 1	15 1
15					6 9		6.7	22 2	14 5	11 4	23 6	18 2	15 2
16	13 0						6 7	22 3	14 6	11 5	23 8	18 3	15 3
17		9.2	8.1	14 0			6 8	22 3	14 7	11 5	23 8	18 4	15 4
18							6.8	22 4	14 8	11 6	23 9	18 5	15 5
19							6 8	22 5	14 8	11 6	24 0	18 6	15 5
20					12 0	14 0	6 8	22 5	14 9	11 7	24 0	18 6	15 6
21	17.0	9 8	32 0				6.8	22 6	14 9	11 7	24 1	18 7	15 6
22				20 0			6 8	22 6	14 9	11 7	24 1	18 7	15 6
23							6 8	22 6	14 9	11 7	24 1	18 7	15 7
24							6 8	22 7	15 0	11 7	24 2	18 7	15 7
25	25 0				14 0	17 0	6 8	22.7	15 0	11.7	24 2	18 8	15 7
26							6 8	22 7	15 0	11 8	24 2	18 8	15 7
27		18.0	12.0	27 0			6 8	22 7	15 0	11 8	24 2	18 8	15 7
28							6.8	22 7	15 0	11 8	24 2	18 8	15 7
29							6 8	22 7	15 0	11 8	24 2	18 8	15 7
30					12 0	19 0	6.8	22 7	15 0	11 8	24 2	18 8	15 7
31	9.7	11.0	21 0				6 8	22 6	14 9	11 7	24 1	18 7	15 6
32							6 8	22 5	14 8	11 6	24 0	18 5	15 5
33				13 0			0 8	22 3	14 7	11 5	23 8	18 4	15 1

Table 2 (Cont.'d)
Comparison Between Robinson-Unit 2 Site Measured (Corrected) Blow Counts and Blow Counts for No Liquefaction Using CP&L IPEEE, EPRI and 97NCEER Magnitude Scaling Factors

	BORING NUMBER						***** MAGNITUDE 5.5 Mw*****				***** MAGNITUDE 6.0 Mw******		
Depth (ft)	101	110	113	114	RL5	RL4	CP&L	EPRI 6041	NCEER97 Lower Bound	NCEER97 Upper Bound	EPRI 6041	NCEER97 Lower Bound	NCEER97 Upper Bound
(Col 1)	(Col 2)	(Col 3)	(Col 4)	(Col 5)	(Col 6)	(Col 7)	(Col 8)	(Col 9)	(Col 10)	(Col 11)	(Col 12)	(Col 13)	(Col 14)
34	12.0						6.8	22.2	14.6	11.4	23.7	18.3	15.3
35	4.2				29.0	18.0	6.8	22.1	14.5	11.4	23.5	18.1	15.2
36							6.7	21.9	14.4	11.3	23.4	18.0	15.0
37		15.0	18.0	22.0			6.7	21.8	14.2	11.2	23.3	17.9	14.9
38							6.7	21.6	14.1	11.1	23.1	17.7	14.8
39	37.0						6.6	21.5	14.0	11.0	23.0	17.6	14.7
40					15.0	9.6	6.6	21.3	13.9	10.9	22.8	17.4	14.6
41	25.0	8.3	48.0				6.6	21.1	13.8	10.8	22.7	17.3	14.5
42							6.5	21.0	13.7	10.7	22.5	17.1	14.3
43							6.5	20.8	13.6	10.6	22.3	17.0	14.2
44				40.0			6.4	20.6	13.4	10.5	22.2	16.8	14.1
45					30.0	15.0	6.4	20.5	13.3	10.4	22.0	16.7	13.9
46	31.0	25.0					6.3	20.3	13.2	10.3	21.8	16.5	13.8
47			44.0				6.3	20.1	13.1	10.2	21.6	16.4	13.7
48	49.0						6.2	19.9	12.9	10.1	21.5	16.2	13.6
49							6.2	19.8	12.8	10.0	21.3	16.1	13.4
50					53.0	20.0	6.1	19.6	12.7	9.9	21.1	15.9	13.3

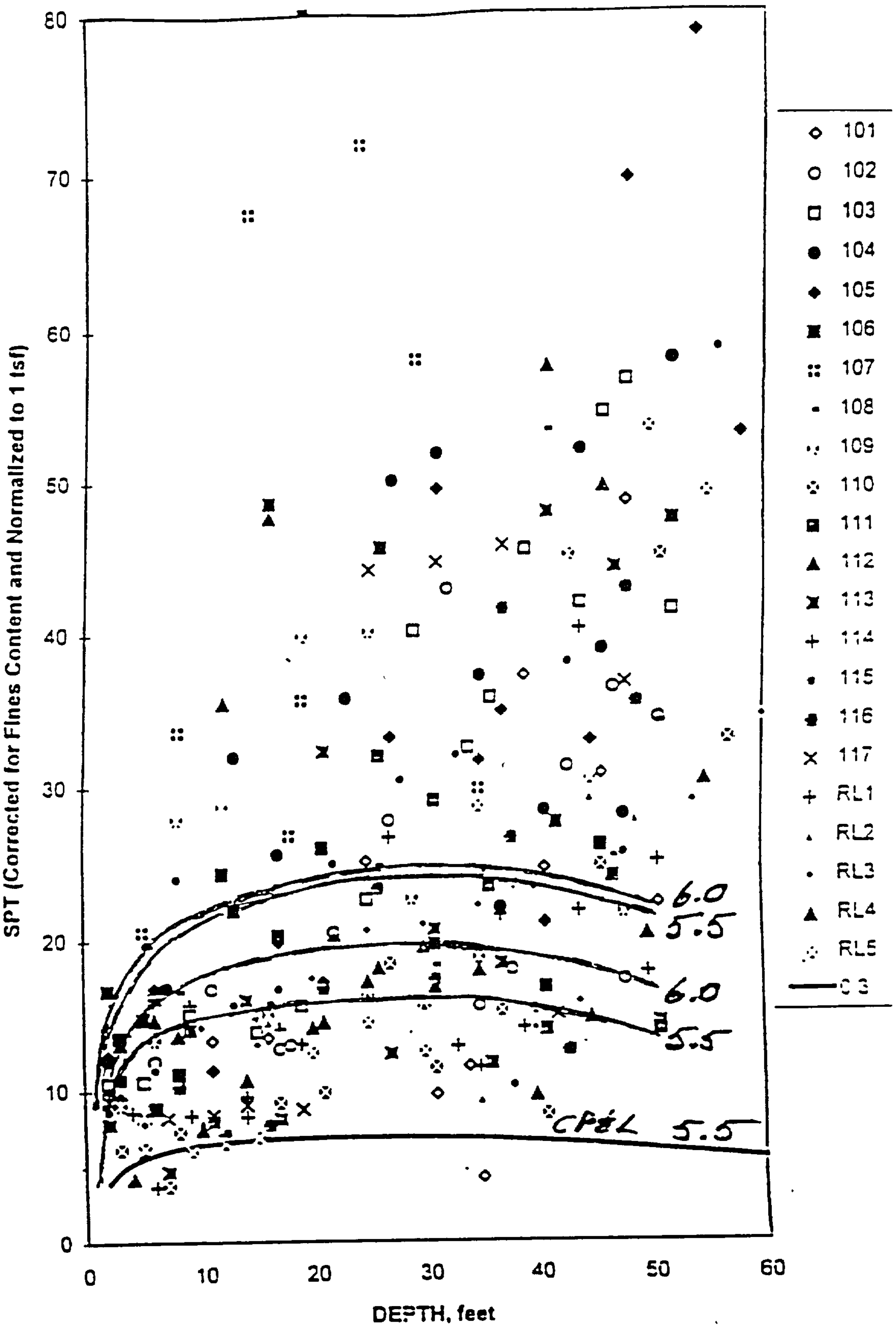


Figure 8 Plot of Robinson-Unit 2 Site Measured (Corrected) Blow Count data and Blow Count Boundaries for No Liquefaction Based on CP&L IPEEE, EPRI and 97NCEER Magnitude Scaling Factors

Such non-liquefiable soil layers can be identified from the actual boring logs. The CP&L boring records for borings 101, 110, 113, RL4, and RL5 (on which the measured blow count data in Figure 8 were based) are reproduced in Appendix A. In these, it can be seen that some of the soil layers are listed as clay, and thus not susceptible to liquefaction. However, most of the soil layers (to a depth of 50 feet) are designated as sands of varying degrees of purity and mixture, as shown:

Designator	Boring Log Description
SW	Fine to coarse sand (in some layers with silt and occasional gravel)
SC	Clayey fine to medium sand
SM	Mottled fine to medium sand (in one layer including some layers of multicolored silty clay)
SP	Fine sand

This is the only description (provided in the RAI responses) of these soil layers. In the RAI response, as part of their argument that liquefaction could not occur, CP&L explicitly mentioned that "fines content" corrections were applied to the SW and SP soil types. Thus these two soil types were clearly considered as being potentially liquefiable. No statement was made (in the RAI) as to whether the SC or SM soil types were considered liquefiable or not. Without knowing the typical constituents of the SC or SM soil layers, it is not possible to exclude them from consideration as potentially liquefiable. **However, from their general description, all four of these soil types would normally be considered as potentially liquefiable.** Thus, in this review, I have considered all four soil layer types noted above as being potentially liquefiable and contributing to potential displacements resulting from liquefaction.

Of course, the impact of any regions of liquefaction on the plant's response to an earthquake will depend on the physical extent of the potentially liquefying soils, the resulting soil displacements, and the ability of the design of the structural foundations and piping support systems to accommodate such displacements.

The calculation of soil displacements due to liquefaction-induced volumetric strains in a soil column seems to be less well established than the identification of incipient liquefaction. However, EPRI-6041 includes two figures that can be used to calculate such displacements. Figure 9 (which is Figure C-17 in EPRI-6041) provides volumetric strains as a function of the CSR and equivalent SPT blow count $N_{1,60}$. Quoting from the original source (Ref. 25),

> "It should be noted that the resulting volumetric strains after liquefaction may be as high as 2 – 3% for loose to medium sands and even higher for very loose sands",

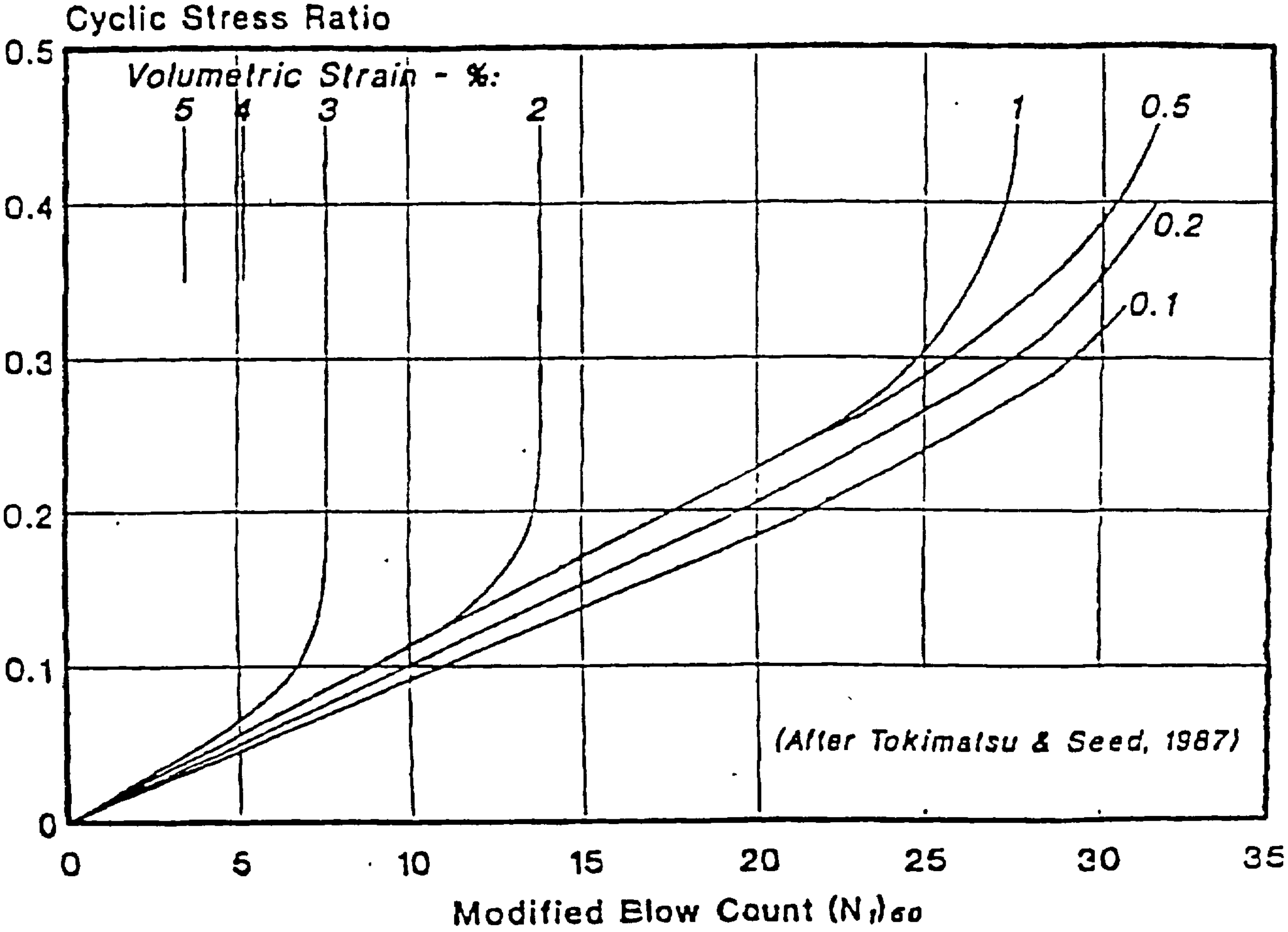


Figure 9 Volumetric Strain versus Cyclic Stress Ratio and Modified Blow Count (from Ref. 10 after Ref. 25)

and

> "It should be recognized that, even under static loading conditions, the error associated with the estimation of settlements in sands is on the order of 25 – 50%. It is therefore reasonable to expect less accuracy in predicting settlements for the more complicated conditions associated with earthquake loading "

However, using the data in Table 2 (assuming a water table depth of 2 feet and a soil density of 118.5 pcf) and a particular choice of magnitude scaling factor, one can compute the soil settlement using the volumetric strains from Figure 9. The details of these calculations are given in the tables in Appendix B, and the results [based on the magnitude scaling factors given by the lower bound recommendations of the 1997 NCEER Workshop Proceedings (ie., MSF = 2.2 for 5.5 M_w and 1.76 for 6.0 M_w)] show the following displacement estimates:

Boring No.	Settlement at 5.5 M_w	Settlement at 6.0 M_w
101	4.8 in.	6.9 in.
110	11.7 in.	12.5 in.
R4	6.9 in.	8.4 in.
R5	10.0 in.	11.3 in.

Most of this displacement comes from soil layers 30 feet deep or shallower. And these values may be conservative due to the presence of non-liquefiable (silt, clay) constituents in the sand layers. However, such displacements (if imposed on a piping system as a relative support displacement) would have to be examined relative to the piping design capacity.

Conclusions

(a) Identification of regions of potential liquefaction is strongly dependent on the (deterministic) earthquake magnitude used to characterize the site and upon the methods used to correct the measured blow counts and to correct for various different earthquake magnitudes (magnitude scaling factors).

(b) The NRC evidently does not currently have a recommended procedure for selecting an appropriately conservative earthquake magnitude or an approved set of magnitude scaling factors for use in soil liquefaction analyses in support of a margin-type assessment.

(c) Use of the magnitude scaling factors in EPRI 6041 or the more recent values recommended in the 1997 NCEER Workshop Proceedings would have resulted in a significant number of points of potential liquefaction being identified at the H. B. Robinson site.

(d) The conclusion (drawn in the H. B. Robinson IPEEE submittal) that there are no areas of liquefaction for a 0.3g RLE at the H. B. Robinson-2 site does not seem to be valid.

(e) Uncertainty in the earthquake magnitude chosen to characterize the site, in the liquefaction/no liquefaction boundaries, in the correlations used in correction for different magnitudes, fines contents, type of equipment used for the SPT, etc., could significantly affect the identification of the regions of potential liquefaction.

4.1.3 Buried Piping Failure due to Soil Liquefaction

As identified in the RAI, the two buried piping systems important to the success paths identified for H. B. Robinson are the diesel fuel system and the primary water system. As stated in the CP&L RAI response, " For the diesel fuel and primary water systems, the finding that permanent ground displacements from soil failure are not expected at Robinson-2 is sufficient to consider these lines as having a HCLPF capacity of at least 0.3 g". Hence, in the RAI response, these lines were initially reviewed only for strains induced by wave propagation. It was estimated that axial strains no greater than 0.013% would result from the RLE earthquake. These strains were identified by CP&L as being within acceptable limits.

However, as discussed during the site visit, if soil liquefaction occurred with associated soil displacements under the piping supports, somewhat larger relative displacements and strains might be expected. In response to a NRC request made during the site visit, CP&L also included (in their RAI response of Feb. 9, 1999) a "bounding" calculation of hypothetical soil displacements in the primary water system piping assuming liquefaction did occur. Referring to the boring logs reproduced in Appendix C, they assumed a 2% volumetric strain in all soil layers for which a blow count less than 10 was measured. This gave a (worst case) total soil column thickness of approximately 10 feet, and a resulting hypothetical soil displacement of 4.3 inches. Adding to this a calculated long-term static displacement of the adjacent Rad Waste building of 0.6 inches gave a total bounding estimate of relative pipe settlement of 4.9 inches. The design limit (for a 102 inch leg) was stated to be 5.7 inches, and this was stated to be a conservative limit for loss of pressure boundary. Thus they demonstrated a 5.7/4.9 = 1.16 factor of safety against hypothetical liquefaction-induced soil displacements.

However, this argument is not convincing, for two reasons:

1. First, they evidently used the raw blow counts directly off the boring logs to determine which layers had blow counts less than 10. They should have used corrected blow counts in estimating the regions of liquefaction. As mentioned earlier, CP&L did not provide the details of the blow count correction factors that they used. However, comparing the corrected blow counts provided by CP&L in Table 2 (this report) with the corresponding raw blow counts shown on the boring

logs in Appendix A (this report) shows that, in the first 10 or 15 feet, the corrected blow counts are less than the raw blow counts Thus, by using raw blow count data in estimating regions of potential liquefaction, they have likely underestimated the extent of potential liquefying layers.

2. Second, and more importantly, the CP&L assumption that liquefaction will not occur has contaminated their argument. As they state, the 2% volumetric strain value was taken from Figure C-18 in EPRI-6041. This figure is reproduced herein as Figure 10 (and is essentially a re-plot of Figure 9). As they state, the 2% volumetric strain value corresponds to an assumed margin of safety against liquefaction of 1.0 and a blow count of 10. **Thus they have assumed that, at worst, only incipient liquefaction might occur (which, of course, is based on their non-conservative choice of magnitude scaling factors as discussed previously).** Referring back to Figure 8 (this report), it can be seen that the margin of safety against liquefaction will be significantly less than 1.0 for many locations in the site soils when either the EPRI-6041 magnitude scaling factors or the 97 NCEER magnitude scaling factors are used in computing the liquefaction potential. Thus, as can be seen from Figure 10, for those soil layers having factors of safety from 0.80 to 0.95 and corrected blow counts between 6 and 10, the actual volumetric strains implied by this figure are 3– 4.5%. Hence the CP&L computed value of maximum soil displacement under the piping system (4.3 inches) is undoubtedly too low by at least 50% and perhaps up to 150%. Such displacements would significantly exceed the allowable support displacement quoted by CP&L.

Support for the above assertions is provided by the soil settlement calculations for Borings 101, 110, RL4, and RL5 which (although not the borings analyzed by CP&L for the primary water system piping) are borings for which corrected blow counts were reported by CP&L (see Table 2). As described in Section 4.1.2, the soil settlements for these borings were re-computed (in this review) using more realistic magnitude scaling factors (see Tables B-1 through B-8). This resulted in computed soil displacements ranging from 4.8 to 11.7 inches at 5.5 M_w and from 6.9 to 12.5 inches at 6.0 M_w. Most of these are significantly greater than the maximum CP&L estimate of 4.3 inches for the primary water piping system.

Finally, it should be noted that the primary service water piping under discussion here consists of 30 inch OD steel pipe with 3/8 inch nominal thickness. Thus the pipe/joint gasket capacity used in estimating the allowable support displacements (quoted by CP&L) may not be very conservative.

Conclusion: The performance of the buried piping associated with the diesel fuel and primary water systems should be reexamined (using nearby SPT bore hole test data) basing the prediction of liquefaction-induced displacements on a magnitude range of 6.0-6.5 M_w and the more realistic magnitude scaling factors recommended by the 1997 NCEER Workshop Proceedings as discussed above.

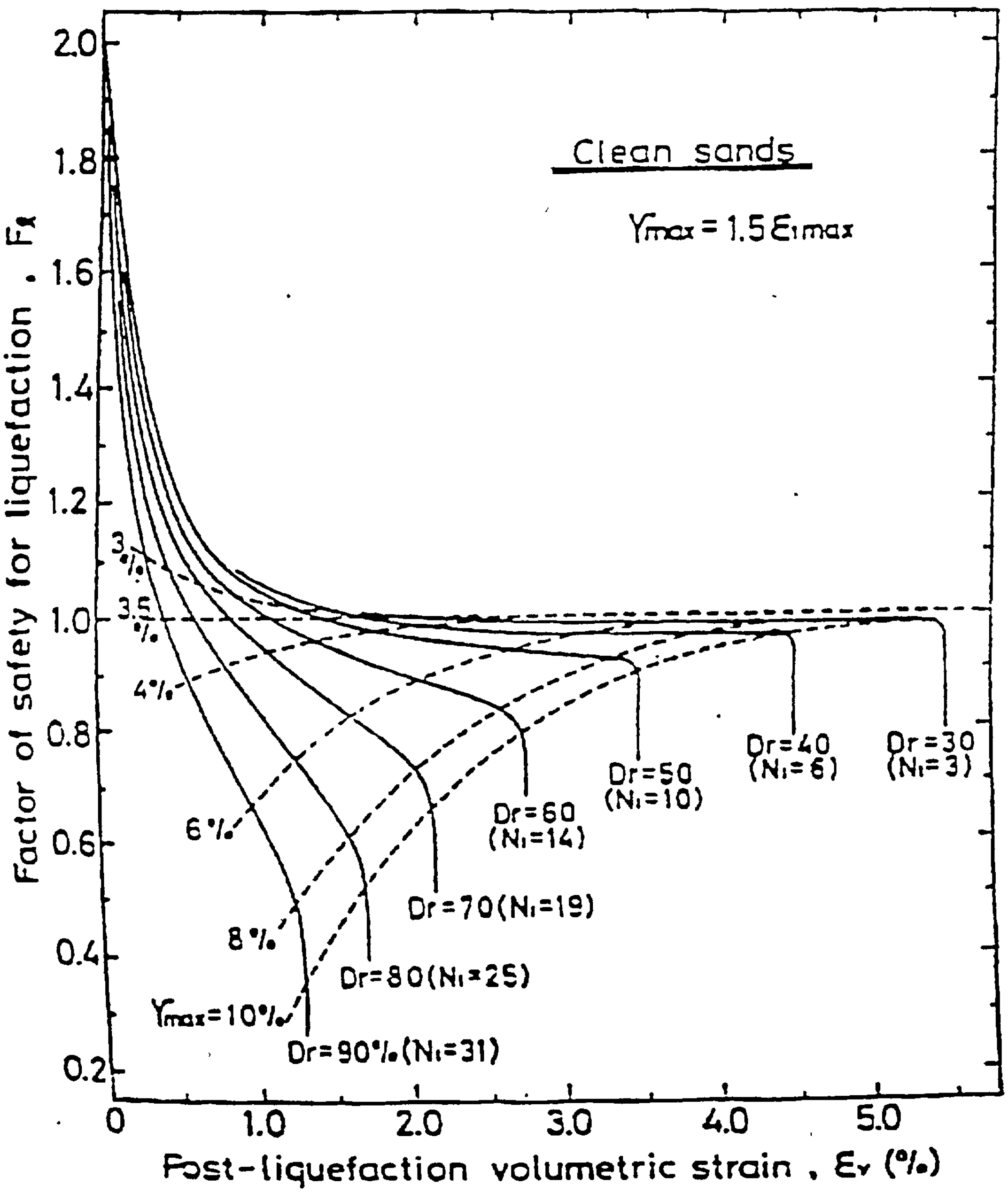


Figure 10 Estimation of Volumetric Strain Based on Factor of Safety Against Liquefaction and Relative Density (from Ref. 10 after Ref. 26)

4.1.4 Seismic Dam Failure

A simplified method due to Newmark (Reference 27) based on block sliding of soil masses was used to assess the Robinson dam (a clay core dam surrounded by cohesionless material). As stated in the RAI response, this procedure is identified as being an acceptable screening tool in EPRI 6041. As further noted, the method is applicable for cases were significant soil degradation of soil strength does not occur.

Using this method, it was found that only minute deformations in the dam would result from the 0.3g RLE, and hence it was inferred that dam failure could be screened from further consideration.

However, if soil liquefaction occurred with associated degradation of soil strength underlying the dam, the results of applying the simplified method may not be appropriate.

Conclusion: The performance of the Robinson dam should be reexamined (using nearby SPT bore hole test data) basing the prediction of liquefaction-induced displacements on a magnitude range of 6.0-6.5 M_w and the more realistic magnitude scaling factors recommended by the 1997 NCEER Workshop Proceedings as discussed above.

4.2 Capacity (HCLPF) Calculations

In the additional information provided by CP&L as part of the side audit (Reference 10), the requested SEWS and engineering calculation sheets were provided for the:

- Motor Operated Valves RHR-750 and RHR-751
- Diesel Fuel Oil Storage Tank, RWST, CST
- Service Water Pumps
- 125 VDC MCCs A & B.

In addition, photographs were provided for all items except the valves. Review of these items verified that appropriate evaluation techniques were employed, and that these items had adequate seismic capacity at the RLE demand level.

Conclusion: Response is adequate

4.3 Concern With Combined Failures Of Two Motor-Operated Valves (With Cast- Iron Yokes)

After the initial IPEEE submitttal, CP&L performed a HCLPF calculation for the valves in question and determined that the valves had HCLPF values of 0.38g, and thus had adequate capacity at the 0.3g RLE demand level.

Conclusion: Response is adequate

4.4 Containment Walkdown Results

In response to RAI No. 4, CP&L provided a copy of a letter (Reference 9) which verified that a containment walkdown had been performed during refueling outage RFO17. The letter also stated that:

a) the walkdown was conducted by Seismic Capability Engineers that met the requirements of the Generic Implementation Procedures, and

b) the scope of the effort complied with the guidance of NUREG 1407.

The letter briefly summarized the results of that walkdown, stating that:

- Containment heat removal system components were determined to have adequate seismic capacity for the 0.3g RLE,
- Component anchorage was determined to be adequate,
- No seismic interactions were noted, and
- The containment fan cooler fan-motors are supported on vibration isolators of sufficient capacity for the RLE seismic demand.

Finally, the letter noted that valves were not included in the walkdown because of their known high capacity (as allowed by NUREG 1407), and that support systems and relays were not specifically addressed in this walkdown because support systems were included in previous walkdowns, and the relays for the containment spray system and containment fan coolers were addressed as part of USI A-46.

Conclusion: Response is adequate

5.0 CONCLUSIONS

As noted above, the supplementary RAI issues associated with the Capacity (HCLPF) Calculations, Motor-Operated Valves With Cast-Iron Yokes and Containment Walkdown Results were satisfactory resolved by the CP&L responses.

The RAI issue related to Soil Failure, Soil Liquefaction and Slope Instability Analyses was not resolved by the information contained in the CP&L responses.

At fundamental issue is the licensee's assertion that no liquefaction is possible for a 0.3g RLE at the Robinson-2 site. This assertion was based on a commonly-used liquefaction identification approach (documented in EPRI-6041) augmented by a different model for choosing magnitude scaling factors. Given the approach the licensee presented, this review has (of necessity) focused on the methods, data, and the numerical results presented.

As documented above, the numerical models and criteria indicate that liquefaction in certain soil layers at the Robinson-2 site is possible for an earthquake magnitude of 5.5 M_w and very probable for earthquake magnitudes above 5.5 M_w. Furthermore, these numerical models show that the resulting liquefaction-induced soil displacements are non-negligible. In the specific case of the buried primary water system piping running outside the Rad Waste building, the best estimate computed soil displacements will exceed those allowable for the piping system. Thus, the liquefaction issue for the IPEEE review level earthquake at Robinson-2 was not resolved by the data and arguments submitted by the licensee.

However, it is recognized that the database from which the models (for predicting liquefaction and resulting displacements) derive comes from observances of liquefaction in clean sands and silty sands. None of the models available today explicitly take into account mixtures of sands (or silty sands) with other constituents (e.g., clay).

At Robinson-2, (from the brief descriptions presented in the boring logs), there are layers of "clean" sands or silty sands as well as layers of sand mixtures. To resolve the liquefaction issue at Robinson-2, it may be necessary to:

(a) examine the constituents of the various soil layers and document a finding that certain layers are not liquefiable, and

(b) demonstrate that any remaining liquefaction-induced soil displacements are accommodated by the plant design and/or residual soil strength.

From an IPEEE viewpoint, the liquefaction issue is very important because it is a phenomenon that is either present or not, and the results of the IPEEE seismic evaluation may be drastically different depending on the answer to the liquefaction question.

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3. Initial RAI ltr (Reference ?)

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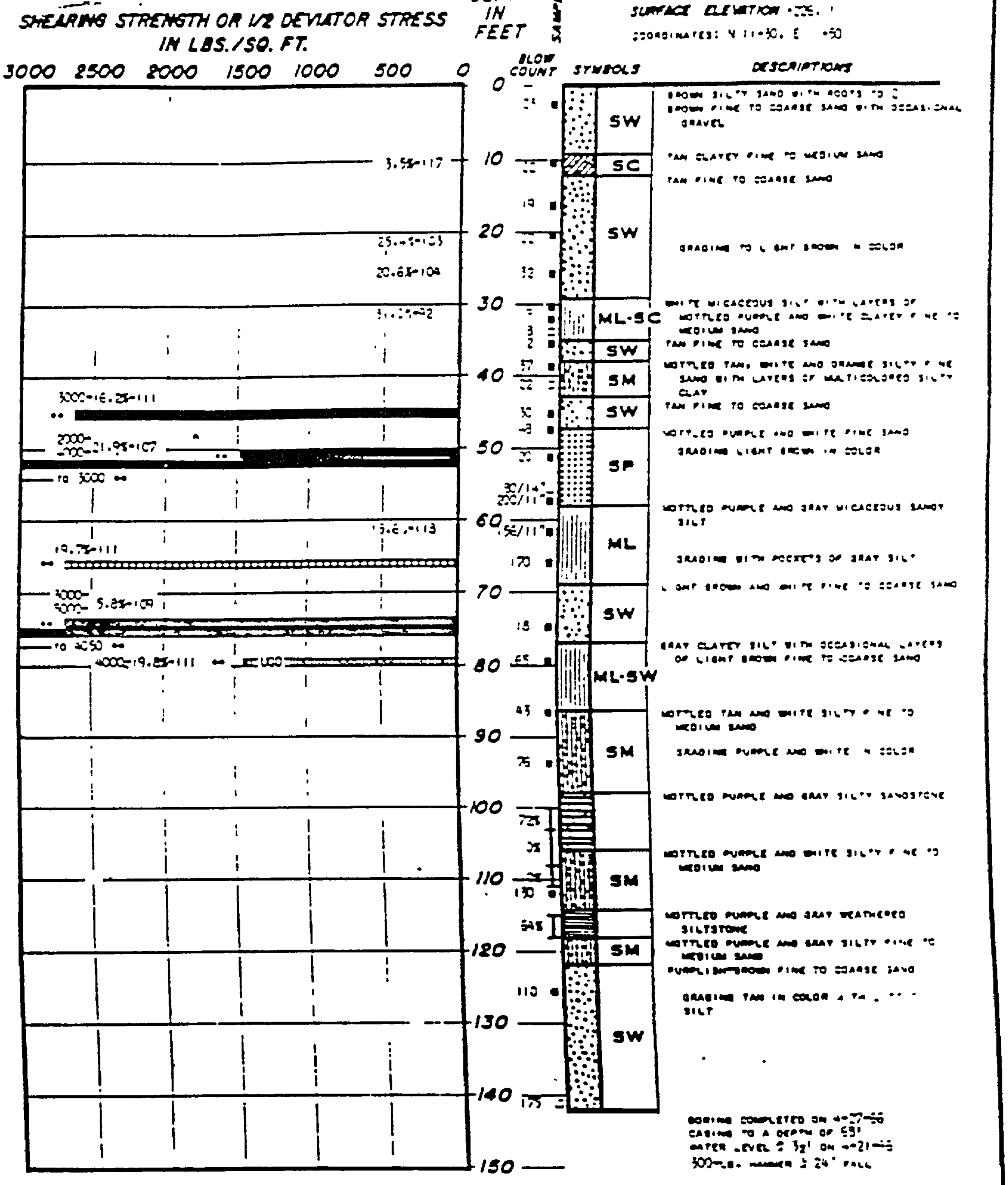
Appendix A

Site Borings Records 101, 110, 113, 114, RL4, RL5

BORING 101

SURFACE ELEVATION +[illegible]

COORDINATES: N 11+30, E [illegible]+50



BORING COMPLETED ON 4-27-66
CASING TO A DEPTH OF 53'
WATER LEVEL @ 3½' ON 4-21-66
300-LB. HAMMER @ 24" FALL

LOG OF BORING

NOTES:

THE FIGURES UNDER THE COLUMN LABELED "BLOW COUNT" INDICATE THE NUMBER OF BLOWS REQUIRED TO ADVANCE THE DAMES & MOORE SAMPLER A DISTANCE OF 1' INTO THE UNDISTURBED SOIL. THE ENERGY USED TO DRIVE THE SAMPLER IS INDICATED AT THE BOTTOM OF EACH BORING LOG.

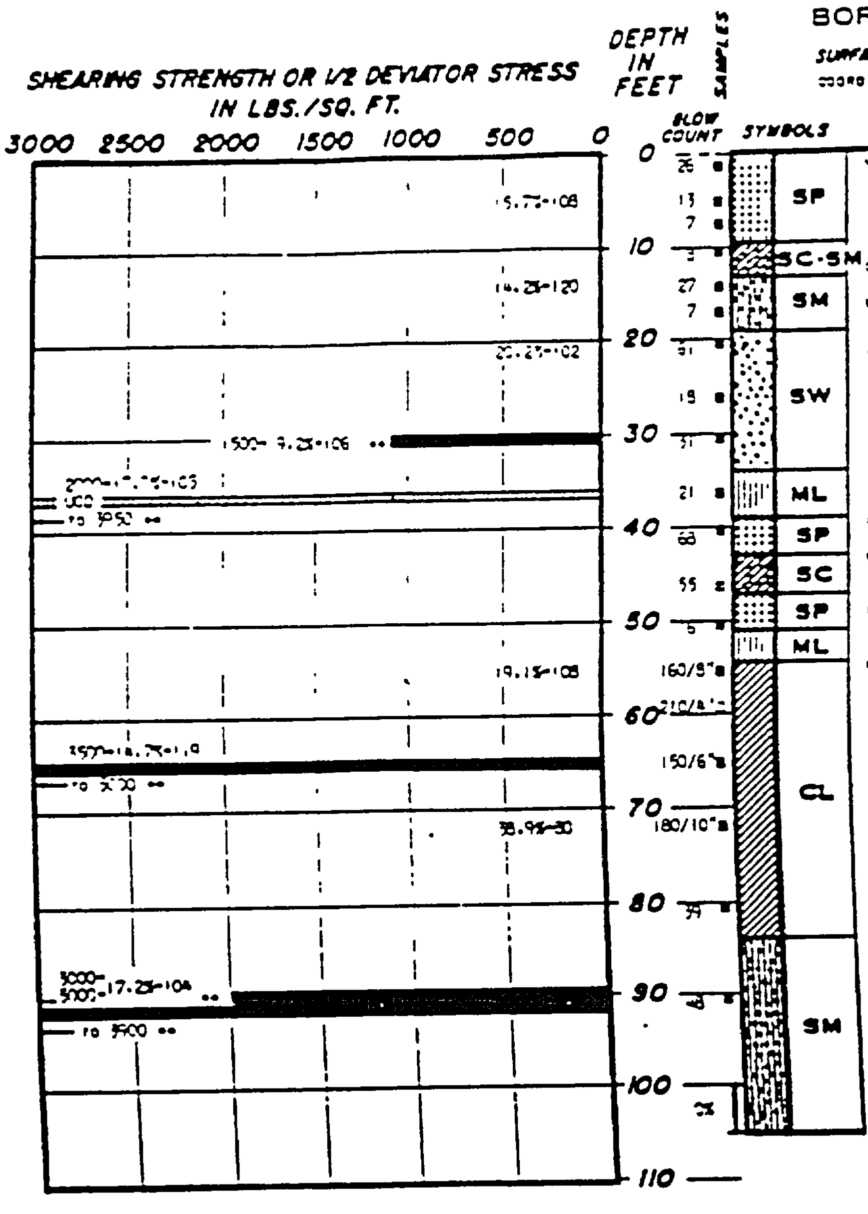
THE DAMES & MOORE SAMPLER IS 3¼" O.D. AND APPROXIMATELY [illegible] I.D.

ELEVATIONS REFER TO PLANT DATUM.

DAMES & MOORE

BORING 113

SURFACE ELEVATION +224.7'
COORDINATES: N 12+00; E 9+00

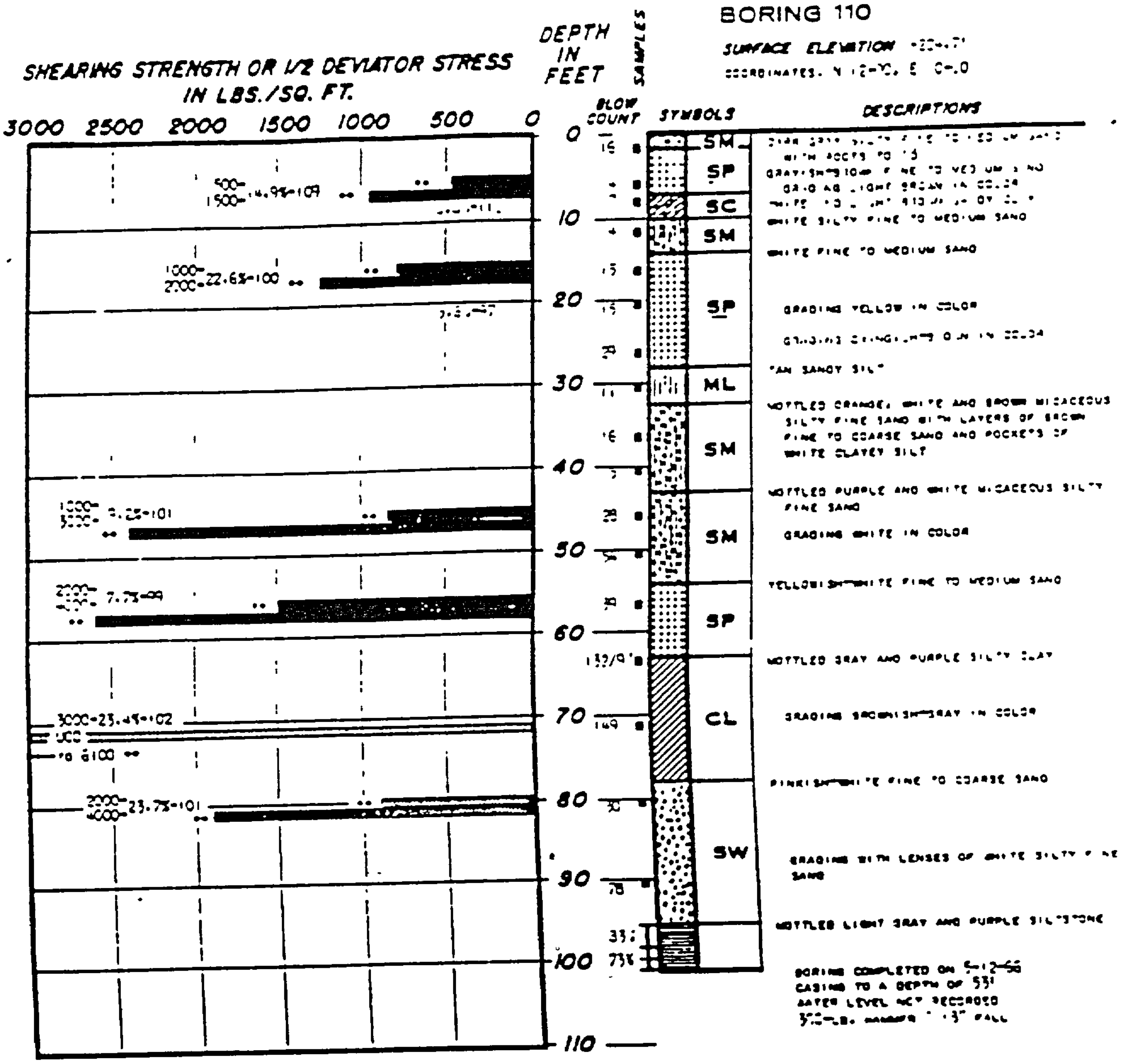


DESCRIPTIONS

LIGHT BROWN AND GRAY FINE TO MEDIUM SAND WITH ROOTS TO 2 INCHES" FILL
GRADING TO LIGHT BROWN IN COLOR, WITH SAND
GRADING TO GRAY FINE TO MEDIUM SAND
END OF FILL
GRAY TO LIGHT GRAY CLAYEY AND SILTY FINE SAND
LIGHT BROWN SILTY FINE SAND
GRADING MOTTLED PURPLE AND WHITE IN COLOR WITH LENSES OF WHITE S.
YELLOWISH-BROWN FINE TO COARSE SAND WITH WHITE CLAY NODULES
GRADING MOTTLED PURPLE AND WHITE IN COLOR
GRADING MOTTLED PURPLE AND ORANGE IN COLOR
TAN MICACEOUS SANDY SILT WITH POCKETS OF WHITE SILT AND FINE TO MEDIUM SAND
PINKISH-WHITE FINE TO MEDIUM SAND
MOTTLED PURPLE, WHITE AND BROWN CLAYEY FINE SAND WITH LENSES OF MEDIUM TO COARSE SAND
BROWN FINE TO COARSE SAND
TAN MICACEOUS SILT
MOTTLED PURPLE AND GRAY SILTY CLAY
GRADING DARK PURPLISH-GRAY IN COLOR
GRADING LIGHT GRAY IN COLOR AND WITH A TRACE OF FINE SAND
MOTTLED WHITE, YELLOW AND PURPLE MICACEOUS SILTY FINE SAND

BORING COMPLETED ON 5-11-66
CASING TO A DEPTH OF 57'
WATER LEVEL @ GROUND SURFACE ON 5-20-66
350-LB. HAMMER @ 18" FALL

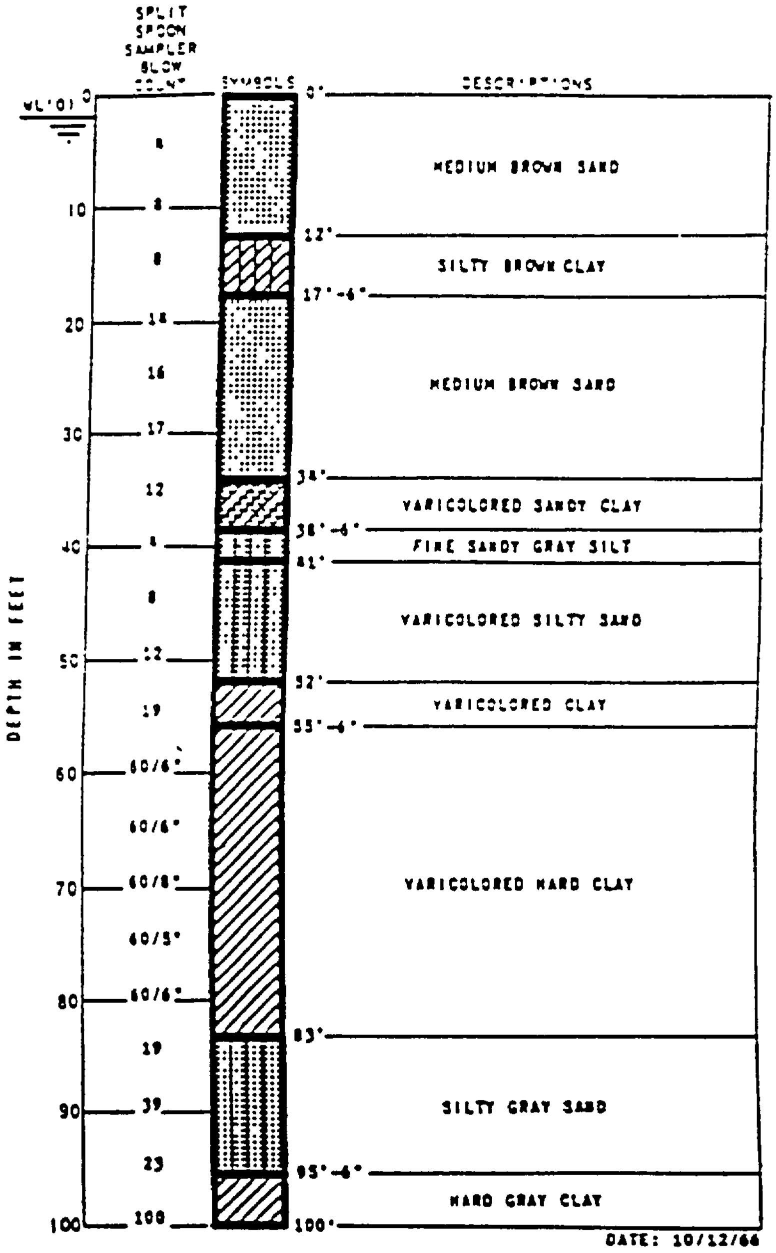
LOG OF BORING



LOG OF BORING

DAMES & MOORE

RAYMOND LOG
OF
BORING NO. 4



USED 10 FT. OF 4" CASING

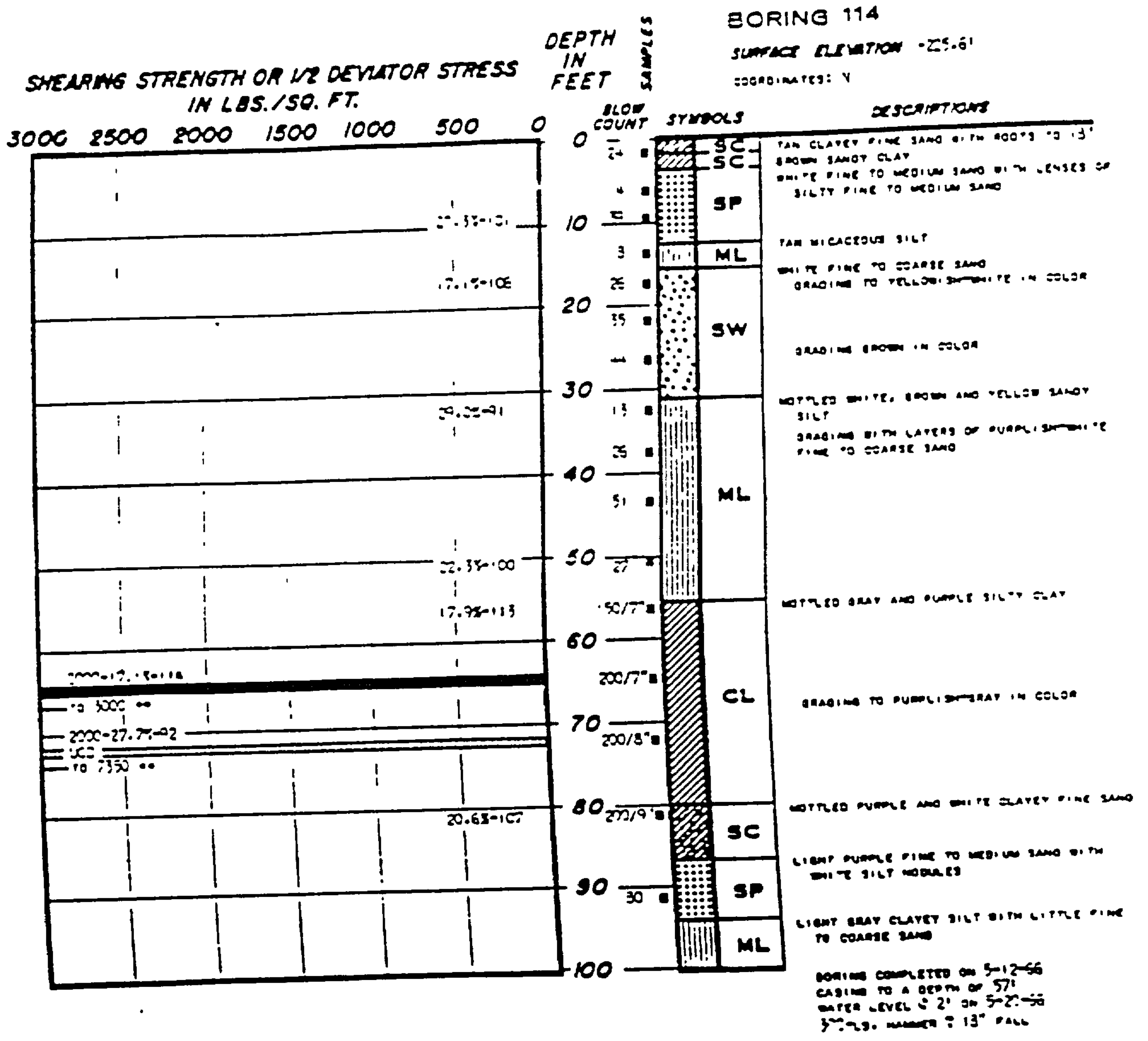
BORING ADVANCED WITH ROTARY
BIT AND DRILLING MUD

NOTES:

Classifications are made by visual inspection.

Water levels (WL). Figure indicates time of reading (hours) after completion of boring. Water levels indicated are those observed when borings were made, or as noted. Porosity of the soil strata, variations of rainfall, site topography, etc., may cause changes in these levels.

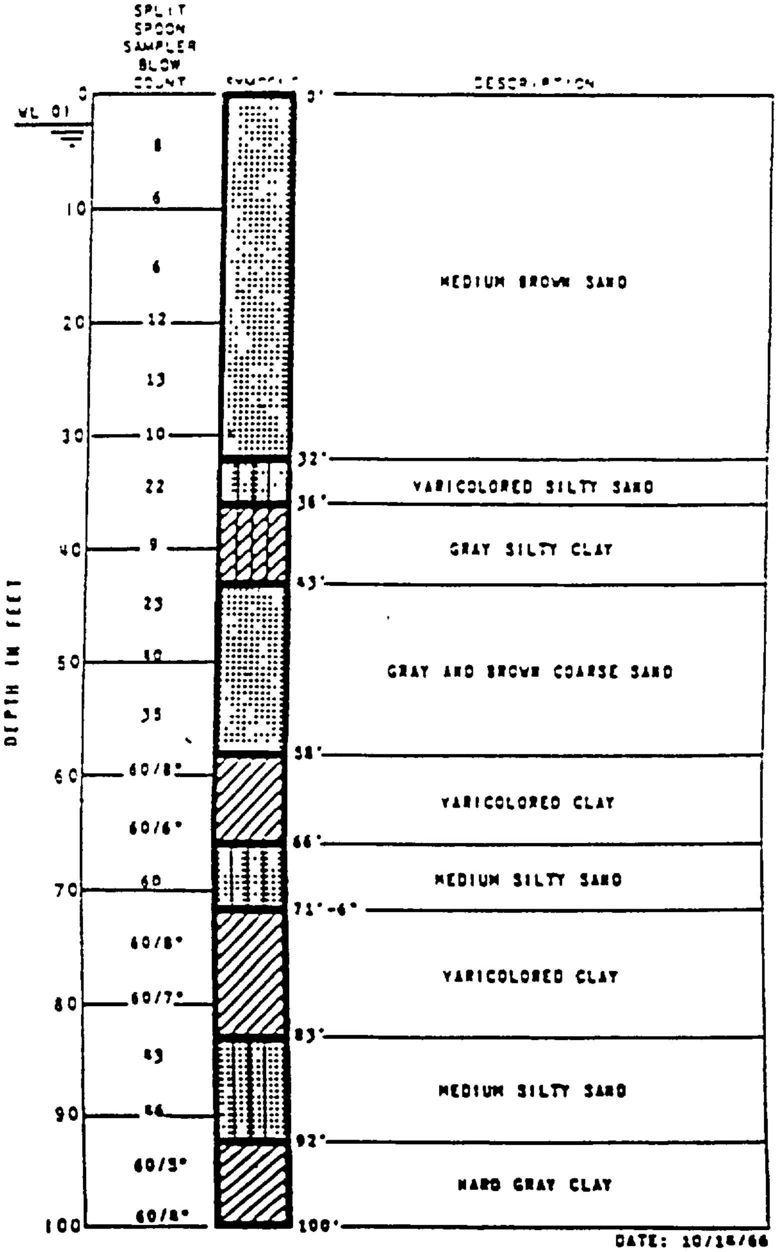
Figures in left-hand column indicate number of blows required to drive 2" O.D.



LOG OF BORING

DAMES & MOORE

RAYMOND LOG
OF
BORING NO. 5



DATE: 10/14/66

USED 10 FT. OF 4" CASING

BORING ADVANCED WITH ROTARY
BIT AND DRILLING MUD

NOTES:

Classifications are made by visual inspection.

Water levels (WL). Figure indicates time of reading (hours) after completion of boring. Water levels indicated are those observed when borings were made, or as noted. Porosity of the soil strata, variations of rainfall, site topography, etc., may cause changes in these levels.

Figures in left-hand column indicate number of blows required to drive 2" O.D. sampling pipe one foot

Appendix B

Computation of Soil Displacements
for
Borings 101, 110, RL4, RL5

Table B-1 Boring 101 Calculation of Soil Displacement at 5.5 M_w

Layer	Type	Depth (ft)	Thk. (ft)	CSR	$N_{1,60}$	ε_{vol} (%)	Δv (in)
1	SW	0.0 – 3.5	3.5	Above	Water	Table	
2	SW	3.5 – 6.5	3.0	0.120	9.5	2.0	0.72
3	SW	6.5 – 10.0	3.5	0.140	9.5	2.0	0.84
4	SC	10.0 – 12.5	2.5	0.142	13.0	0.2	0.06
5	SW	12.5 – 15.0	2.5	0.156	13.0	1.5	0.45
6	SW	15.0 – 17.5	2.5	0.158	13.0	1.5	0.45
7	SW	17.5 – 20.0	2.5	0.160	13.0	1.7	0.51
8	SW	20.0 – 22.5	2.5	0.161	17.0	0.2	0.06
9	SW	22.5 – 25.0	2.5	0.162	17.0	0.2	0.06
10	SW	25.0 – 27.5	2.5	0.162	25.0	0.0	0.0
11	SW	27.5 – 30.0	2.5	0.162	25.0	0.0	0.0
12	ML-SC	30.0 – 35.0	6.0	No	Liquef.		
13	SW	35.0 – 38.0	3.0	0.155	4.2	4.5	1.62
14	SM	38.0 – 42.5	4.5	0.150	37.0	0.0	0.0
15	SW	42.5 – 45.0	2.5	0.145	25.0	0.0	0.0
16	SW	45.0 – 47.5	2.5	0.143	31.0	0.0	0.0
17	SP	47 5 – 50.0	2.5	0.139	49.0	0.0	0.0
						Total	**4.8 in.**

Table B-2 Boring 110 Calculation of Soil Displacement at 5.5 M_w

Layer	Type	Depth (ft)	Thk. (ft)	CSR	$N_{1,60}$	ε_{vol} (%)	Δv (in)
1	SM	0.0 – 2.0	2.0	Above	Water	Table	
2	SP	2.0 – 5.0	3.0	0.119	6.1	3.75	1.35
3	SP	5.0 – 7.0	2.0	0.135	3.7	5.0	1.20
4	SC	7.0 – 10.0	3.0	0.142	7.3	3.0	1.08
5	SM	10.0 – 13.0	3.0	0.153	7.3	3.0	1.08
6	SP	13.0 – 16.0	3.0	0.157	6.5	3.3	1.19
7	SP	16.0 – 19.0	3.0	0.159	9.2	2.7	0.97
8	SP	10.0 – 22.0	3.0	0.161	9.8	2.7	0.97
9	SP	22.0 – 25.0	3.0	0.162	9.8	2.7	0.97
10	SP	25.0 – 28.0	3.0	0.162	18.0	0.1	0.04
11	ML	28.0 – 32.0	4.0		No	Liquef.	
12	SM	32.0 – 35.0	3.0	0.158	11.0	2.5	0.9
13	SM	35.0 – 38.0	3.0	0.154	11.0	2.5	0.9
14	SM	38.0 – 41.0	3.0	0.151	15.0	0.2	0.07
15	SM	41.0 – 44.0	3.0	0.148	8.3	2.8	1.01
16	SM	44.0 – 47.0	3.0	0.143	25.0	0.0	0.0
17	SM	47.0 – 50.0	3.0	0.140	25.0	0.0	0.0
						Total	**11.7 in.**

Table B-3 Boring RL4 Calculation of Soil Displacement at 5.5 M_w

Layer	Type	Depth (ft)	Thk. (ft)	CSR	$N_{1,60}$	ε_{vol} (%)	Δv (in)
1	Brown Sand	0.0 – 2.0	2.0	Above	Water	Table	
2	Brown Sand	2.0 – 5.0	3.0	0.113	4.2	4.6	1.66
3	Brown Sand	5.0 – 8.0	3.0	0.137	4.2	4.6	1.66
4	Brown Sand	8.0 – 12.0	4.0	0.150	7.4	3.0	1.44
5	Clay	12.0 – 17.5			No	Liquef.	
6	Brown Sand	17.5 – 21.0	3.5	0.160	11.0	2.4	1.01
7	Brown Sand	21.0 – 25.0	4.0	0.162	14.0	0.4	0.19
8	Brown Sand	25.0 – 28.0	3.0	0.162	17.0	0.1	0.04
9	Brown Sand	28.0 – 31.0	3.0	0.162	19.0	0.0	0.0
10	Brown Sand	31.0 – 34.0	3.0	0.159	19.0	0.0	0.0
11	Sandy Clay	34.0 – 38.5			No	Liquef.	
12	Silt	38.5 – 41.0			No	Liquef.	
13	Silty Sand	41.0 – 44.0	3.0	0.147	9.8	2.4	0.86
14	Silty Sand	44.0 – 47.0	3.0	0.143	15.0	0.1	0.04
15	Silty Sand	47.0 – 51.0	4.0	0.139	20.0	0.0	0.0
						Total	**6.9 in.**

Table B-4 Boring RL5 Calculation of Soil Displacement at 5.5 M_w

Layer	Type	Depth (ft)	Thk. (ft)	CSR	$N_{1,60}$	ε_{vol} (%)	Δv (in)
1	Brown Sand	0.0 – 2.0	2.0	Above	Water	Table	
2	Brown Sand	2.0 – 6.0	4.0	0.119	6.2	3.5	1.68
3	Brown Sand	6.0 – 10.0	4.0	0.144	6.0	3.75	1.80
4	Brown Sand	10.0 – 14.0	4.0	0.154	6.0	3.75	1.80
5	Brown Sand	14.0 – 18.0	4.0	0.158	6.9	3.50	1.68
6	Brown Sand	18.0 – 22.0	4.0	0.161	12.0	2.33	1.12
7	Brown Sand	22.0 – 26.0	4.0	0.162	12.0	2.33	1.12
8	Brown Sand	26.0 – 30.0	4.0	0.162	14.0	0.35	0.17
9	Brown Sand	30.0 – 32.0	2 0	0.162	12.0	2.33	0.56
10	Silty Sand	32.0 – 36 0	4.0	0.158	29.0	0.0	0.0
11	Sandy Clay	36.0 – 43.0	7.0		No	Liquef.	
12	Coarse Sand	43.0 – 47.0	4 0	0.144	15.0	0.1	0.05
13	Coarse Sand	47.0 – 51.0	4.0	0.139	30.0	0.0	0.0
						Total	**10.0 in.**

Table B-5 Boring 101 Calculation of Soil Displacement at 6.0 M_w

Layer	Type	Depth (ft)	Thk. (ft)	CSR	$N_{1,60}$	ε_{vol} (%)	Δv (in)
1	SW	0.0 – 3.5	3.5	Above	Water	Table	
2	SW	3.5 – 6.5	3.0	0.150	9.5	2.6	0.94
3	SW	6.5 – 10 0	3.5	0.175	9.5	2.7	1.13
4	SC	10.0 – 12.5	2.5	0.178	13 0	2.0	0.60
5	SW	12.5 – 15.0	2.5	0.195	13.0	2.1	0.63
6	SW	15.0 – 17.5	2.5	0 198	13.0	2.1	0.63
7	SW	17.5 – 20.0	2.5	0.200	13.0	2.1	0.63
8	SW	20.0 – 22.5	2.5	0.201		1.2	0.36
9	SW	22.5 – 25.0	2.5	0.203	17.0	1.2	0.36
10	SW	25.0 – 27.5	2.5	0.203	25.0	0.0	0.0
11	SW	27.5 – 30.0	2.5	0.203	25.0	0.0	0.0
12	ML-SC	30.0 – 35.0	6.0	No	Liquef.		
13	SW	35.0 – 38.0	3.0	0.194	4.2	4.5	1.62
14	SM	38.0 – 42.5	4.5	0.188	37.0	0.0	0.0
15	SW	42.5 – 45.0	2.5	0.181	25.0	0.0	0.0
16	SW	45 0 – 47 5	2.5	0.179	31.0	0 0	0.0
17	SP	47 5 – 50 0	2 5	0.174	49.0	0.0	0.0
						Total	**6.9 in.**

Table B-6 Boring 110 Calculation of Soil Displacement at 6.0 M_w

Layer	Type	Depth (ft)	Thk. (ft)	CSR	$N_{1,60}$	ε_{vol} (%)	Δv (in)
1	SM	0.0 – 2.0	2.0	Above	Water	Table	
2	SP	2.0 – 5.0	3 0	0.149	6.1	3.75	1.35
3	SP	5.0 – 7.0	2.0	0.169	3 7	5.0	1.20
4	SC	7.0 – 10.0	3.0	0.178	7.3	3.0	1.08
5	SM	10.0 – 13.0	3.0	0.191	7.3	3.0	1.08
6	SP	13.0 – 16.0	3.0	0.196	6.5	3.6	1.30
7	SP	16.0 – 19.0	3.0	0.199	9.2	2.7	0.97
8	SP	10.0 – 22.0	3.0	0.201	9.8	2.7	0.97
9	SP	22.0 – 25.0	3.0	0.203	9.8	2.7	0.97
10	SP	25.0 – 28.0	3.0	0.203	18.0	0.4	0.14
11	ML	28.0 – 32.0	3.0		No	Liquef.	
12	SM	32.0 – 35.0	4.0	0.198	11.0	2.5	0 9
13	SM	35.0 – 38.0	3.0	0.193	11.0	2.5	0.9
14	SM	38.0 – 41.0	3.0	0.189	15.0	1.6	0.58
15	SM	41.0 – 44.0	3.0	0.185	8.3	2.8	1.01
16	SM	44 0 – 47 0	3 0	0 179	25.0	0.0	0.0
17	SM	47.0 – 50.0	3.0	0.175	25 0	0.0	0.0
						Total	**12.5 in.**

Table B-7 Boring RL4 Calculation of Soil Displacement at 6.0 M_w

Layer	Type	Depth (ft)	Thk. (ft)	CSR	$N_{1,60}$	ε_{vol} (%)	Δv (in)
1	Brown Sand	0.0 – 2.0	2.0	Above	Water	Table	
2	Brown Sand	2.0 – 5.0	3.0	0.141	4 2	4.6	1.66
3	Brown Sand	5.0 – 8.0	3.0	0.171	4.2	4.6	1.66
4	Brown Sand	8.0 – 12.0	4.0	0.188	7.4	3.0	1.44
5	Clay	12.0 – 17.5			No	Liquef.	
6	Brown Sand	17.5 – 21.0	3.5	0.200	11.0	2.5	1.05
7	Brown Sand	21.0 – 25.0	4.0	0.203	14.0	1.8	0.86
8	Brown Sand	25.0 – 28.0	3.0	0.203	17.0	1.2	0.43
9	Brown Sand	28.0 – 31.0	3.0	0.203	19.0	0.2	0.07
10	Brown Sand	31.0 – 34.0	3.0	0.199	19.0	0.2	0.07
11	Sandy Clay	34.0 – 38.5			No	Liquef.	
12	Silt	38.5 – 41.0			No	Liquef.	
13	Silty Sand	41.0 – 44.0	3.0	0.184	9.8	2.6	0.94
14	Silty Sand	44.0 – 47.0	3.0	0.179	15.0	0.5	0.18
15	Silty Sand	47.0 – 51.0	4.0	0.174	20.0	0.0	0.0
						Total	**8.4 in.**

Table B-8 Boring RL5 Calculation of Soil Displacement at 6.0 M_w

Layer	Type	Depth (ft)	Thk. (ft)	CSR	$N_{1,60}$	ε_{vol} (%)	Δv (in)
1	Brown Sand	0.0 – 2.0	2.0	Above	Water	Table	
2	Brown Sand	2.0 – 6.0	4.0	0.149	6.2	3.6	1.73
3	Brown Sand	6.0 – 10.0	4.0	0.180	6.0	3.75	1.80
4	Brown Sand	10.0 – 14.0	4.0	0.193	6.0	3.75	1.80
5	Brown Sand	14.0 – 18.0	4.0	0.198	6.9	3.50	1.68
6	Brown Sand	18.0 – 22.0	4.0	0.201	12.0	2.33	1.12
7	Brown Sand	22.0 – 26.0	4.0	0.203	12.0	2.33	1.12
8	Brown Sand	26.0 – 30.0	4.0	0.203	14.0	1.75	0.84
9	Brown Sand	30.0 – 32.0	2.0	0.203	12.0	2.33	0.56
10	Silty Sand	32.0 – 36.0	4.0	0.198	29.0	0.0	0.0
11	Sandy Clay	36.0 – 43.0	7.0		No	Liquef.	
12	Coarse Sand	43.0 – 47.0	4.0	0.180	15.0	1.4	0.67
13	Coarse Sand	47.0 – 51.0	4.0	0.174	30.0	0.0	0.0
						Total	**11.3 in.**

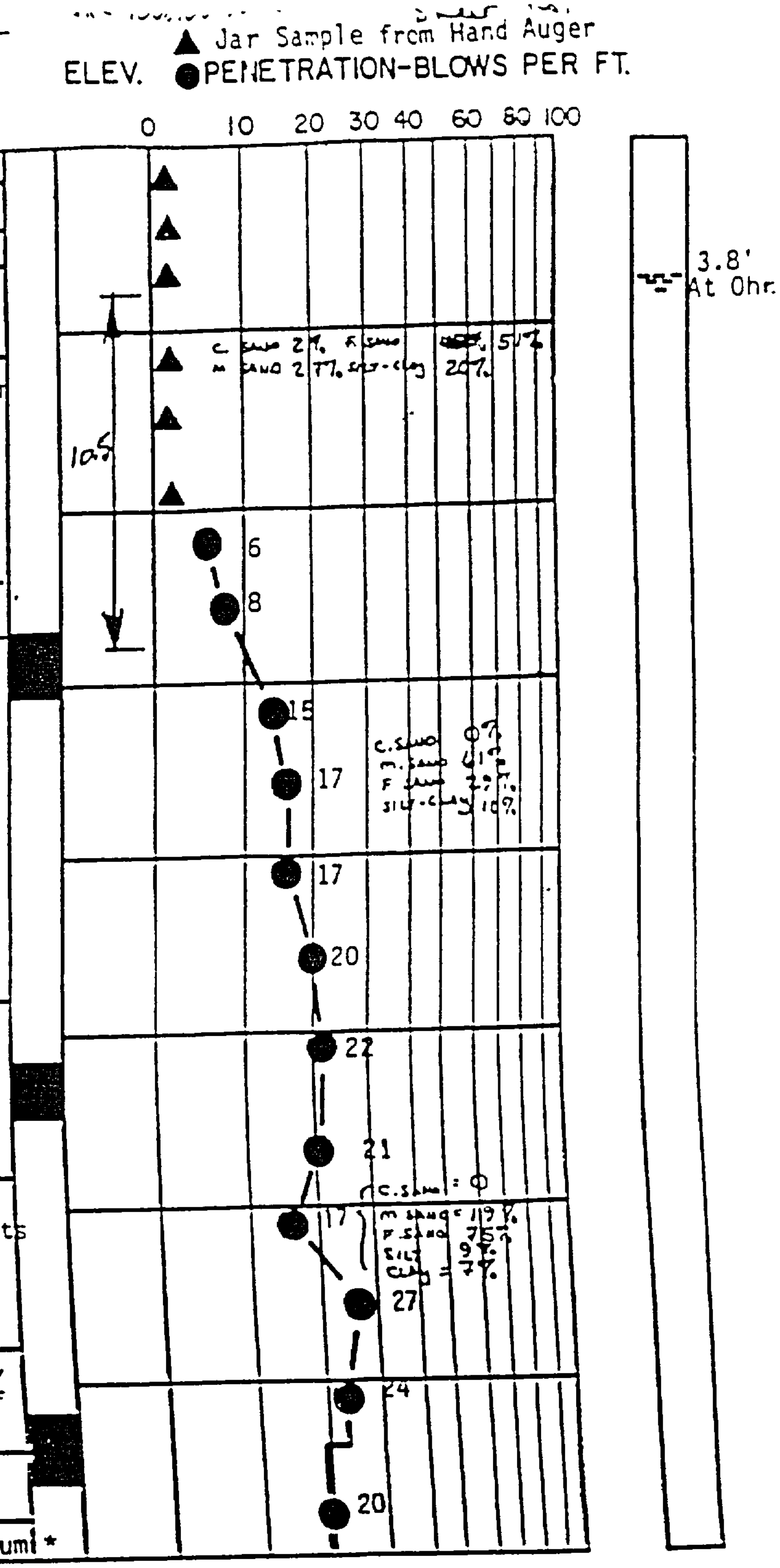
Appendix C

Site Borings Records R1, R2, R3, R4

CP&L H.B Robinson Unit 2
VR/SMS Foundation Analysis
By: B. Shields March '82

DEPTH FT.	DESCRIPTION
0.0	
0.	CONCRETE SLAB C-
.0	Tan Slightly Silty Fine SAND (Fill)
	White with Orange Fine Sandy SILT
3.0	Tan to Orange Silty Fine SAND
5.5	Loose Tan ~~Slightly~~ Silty Fine to Medium SAND
12.0	Loose to Firm Tan to Orange Slightly Silty Fine SAND, Occasional Small Silt-Clay Pockets (TO MEDIUM)
24.0	Very Firm Orange Slightly Silty Fine to Medium SAND, occasional Small Silt-Clay Pockets
29.0	Firm to Very Firm Tan Slightly Silty Fine SAND, occasional Small Silt Pockets
34.0	Very Firm Red to Maroon Slightly Silty Medium SAND with Pockets and Lenses of Silt
37.0	Firm Yellow Tan SILT and Fine SAND
39.0	
40.0	Firm Pink Slightly Silty Fine to Medium*

*SAND



TEST BORING RECORD

BORING NO. R-1
DATE DRILLED 12-23-81
JOB NO. RS-1823

SOIL & MATERIAL ENGINEERS, INC.

BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113

PENETRATION IS THE NUMBER OF BLOWS OF 140 LB HAMMER FALLING 30 IN REQUIRED TO DRIVE 1.4 IN I.D. SAMPLER 1 FT

3W-14

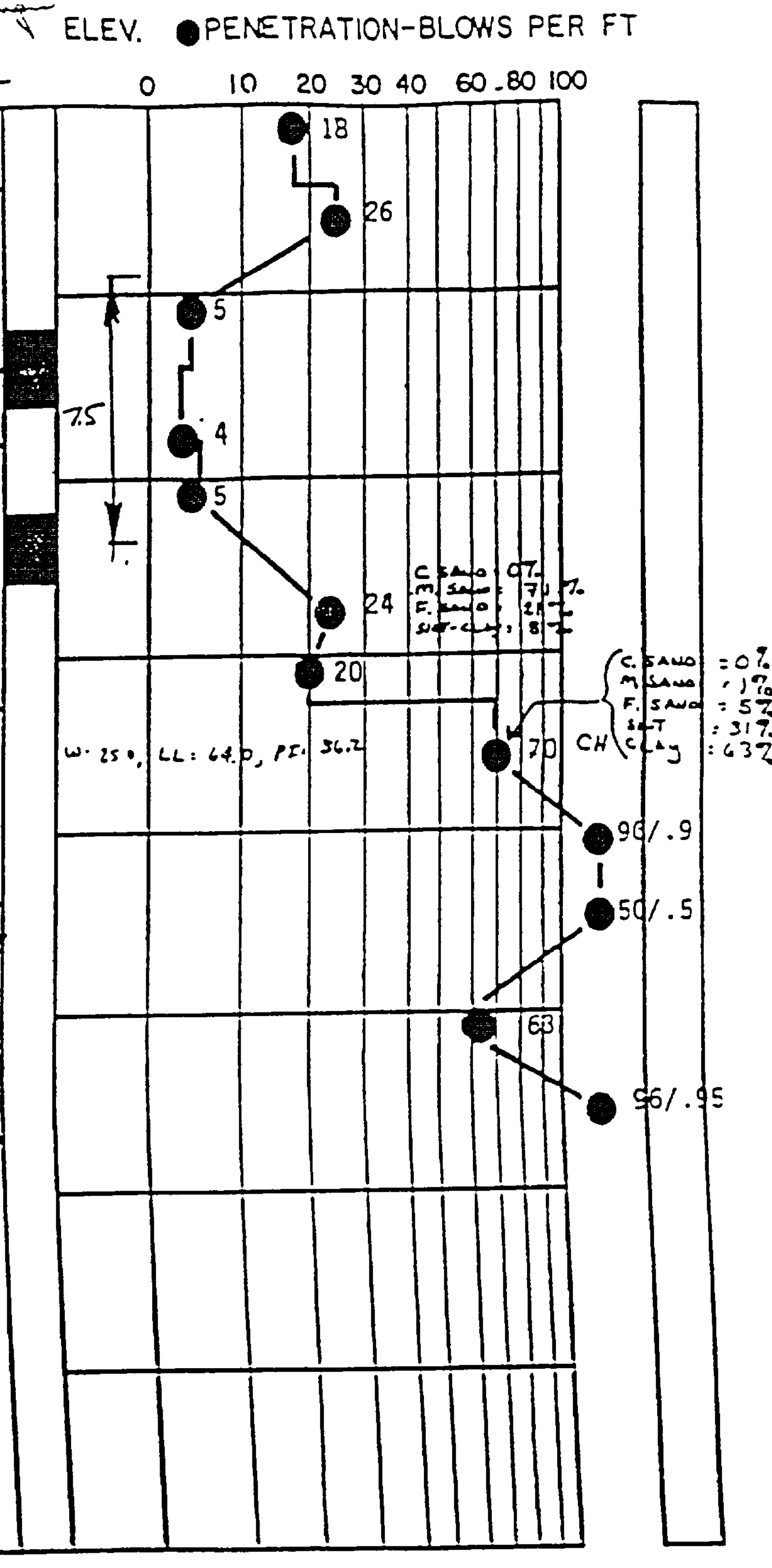
▇ UNDISTURBED SAMPLE
|50|% ROCK CORE RECOVERY
◀ LOSS OF DRILLING WATER
WATER TABLE-24 HR
WATER TABLE-1 HR

VRISHS [illegible]

By: B. Shield, March '82

DEPTH FT.	DESCRIPTION
40.0	(See previous page for description)
	Loose to Very Firm Brown, Orange and Maroon Slightly Silty Fine SAND and Thin SILT Layers
47.0	Loose Tan Fine to Coarse SAND
49.0	Loose to Firm Tan to Orange Slightly Silty Fine to Medium SAND
56.5	Very Hard Light Gray to Maroon Slightly ~~Clayey SILT~~ FINE SANDY SILTY CLAY
62.0	Very Hard Light Gray and Maroon Slightly Sandy an~~d Clayey SILT~~ SILTY CLAY
66.0	Very Hard Light Gray and Maroon ~~Slightly Clayey SILT~~ SILTY CLAY
68.5	Boring Terminated at 68.5

18' WITH BLOW COUNTS <10
@ 1.5% ⇛ 3.2"
@ 2% ⇛ 4.3"



SM-14

BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB HAMMER FALLING 30 IN REQUIRED TO DRIVE 1.4 IN I D SAMPLER 1 FT

▇ UNDISTURBED SAMPLE
50% ROCK CORE RECOVERY
◀ LOSS OF DRILLING WATER
═ WATER TABLE-24 HR
═ WATER TABLE-1 HR

TEST BORING RECORD

BORING NO. R-1
DATE DRILLED 12-23-81
JOB NO. RS-1823

SOIL & MATERIAL ENGINEERS, INC.

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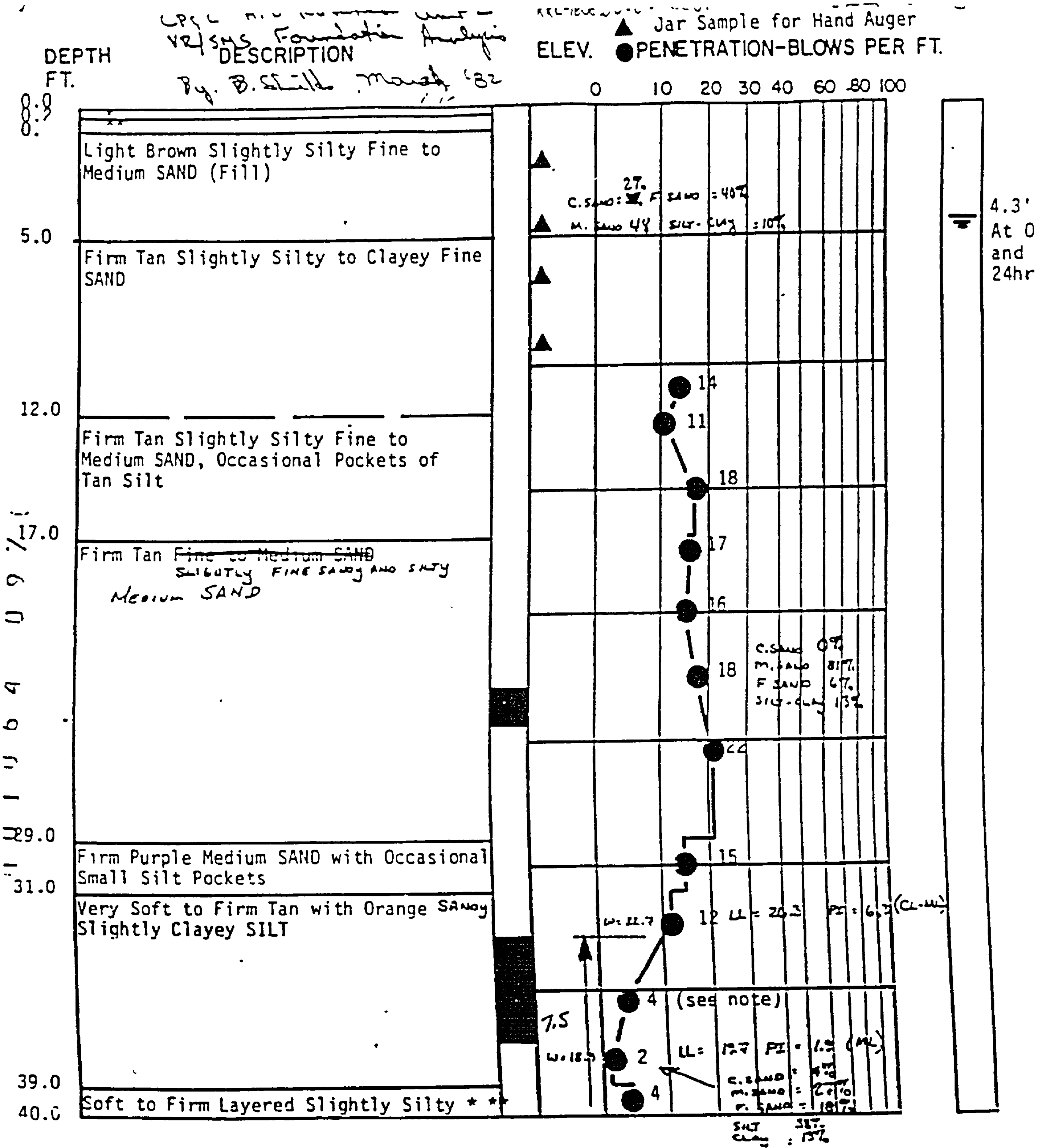
VR/SMS Foundation Analysis
By. B. Shill March '82



Light Brown Slightly Silty Fine to Medium SAND (Fill)

Firm Tan Slightly Silty to Clayey Fine SAND

Firm Tan Slightly Silty Fine to Medium SAND, Occasional Pockets of Tan Silt

Firm Tan ~~Fine to Medium SAND~~ Slightly fine sandy and silty Medium SAND

Firm Purple Medium SAND with Occasional Small Silt Pockets

Very Soft to Firm Tan with Orange Sandy Slightly Clayey SILT

Soft to Firm Layered Slightly Silty * *

TEST BORING RECORD

BORING NO. R-2
DATE DRILLED 12-21-81
JOB NO. RS-1823

SOIL & MATERIAL ENGINEERS, INC.

BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
~~PE~~NETRATION IS THE NUMBER OF BLOWS OF 140 LB HAMMER ~~FA~~LLING 30 IN REQUIRED TO DRIVE 1.4 IN I D SAMPLER 1 FT.

▮ UNDISTURBED SAMPLE
|50|% ROCK CORE RECOVERY
◀ LOSS OF DRILLING WATER
═ WATER TABLE-24HR
╪ WATER TABLE-1HR

SM-14

RNP-RA/99-0013 Page 15 of 24

VR/SMS [illegible] [illegible]

By. D. Shiell March '82

DEPTH FT.	DESCRIPTION
40.0	(See previous page for description)
49.0	Firm Tan Slightly Silty Fine to Medium SAND
51.0	Very Firm Light Maroon Medium SAND Slightly Fine Sandy and Clayey
54.5	Very Firm Light Gray Slightly Fine Sandy ~~SILT~~ Silty CLAY
60.0	Boring Terminated at 60'

* ASPHALT PAVEMENT

** RED Slightly Silty and Gravelly SAND (Fill)

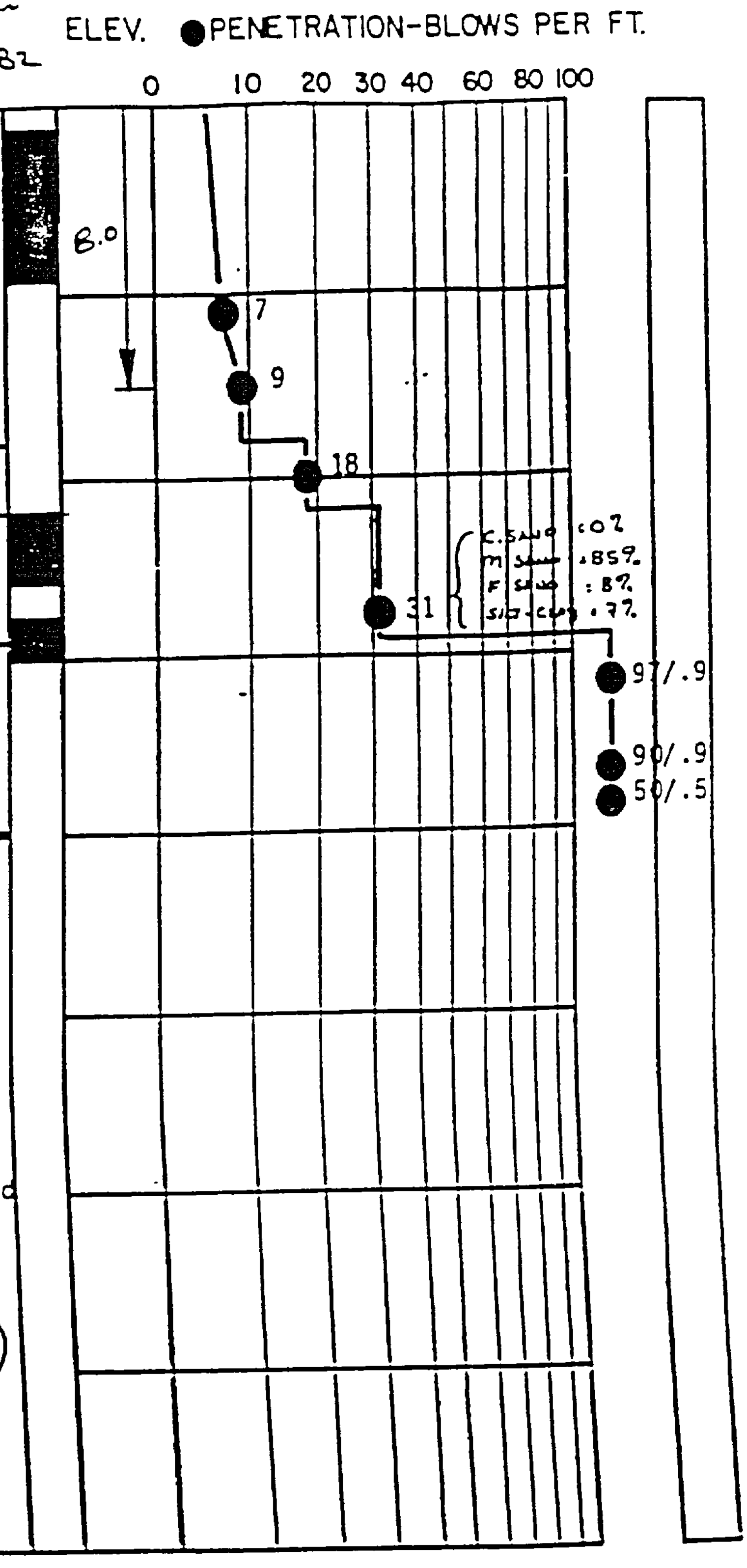
*** Fine SAND and Fine Sandy SILT

Note: Soil from 35.5' to 37.0' Standard Penetration Sample previously disturbed by attempted Shelby Tube Sampling

15.5' with blow counts <10

@ 1.5% ⇒ 2.8"

@ 2% ⇒ 3.7"



TEST BORING RECORD

BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113

ETRATION IS THE NUMBER OF BLOWS OF 140 LB HAMMER
LING 30 IN REQUIRED TO DRIVE 14 IN I D SAMPLER 1 FT

▨ UNDISTURBED SAMPLE
50% ROCK CORE RECOVERY
◀ LOSS OF DRILLING WATER

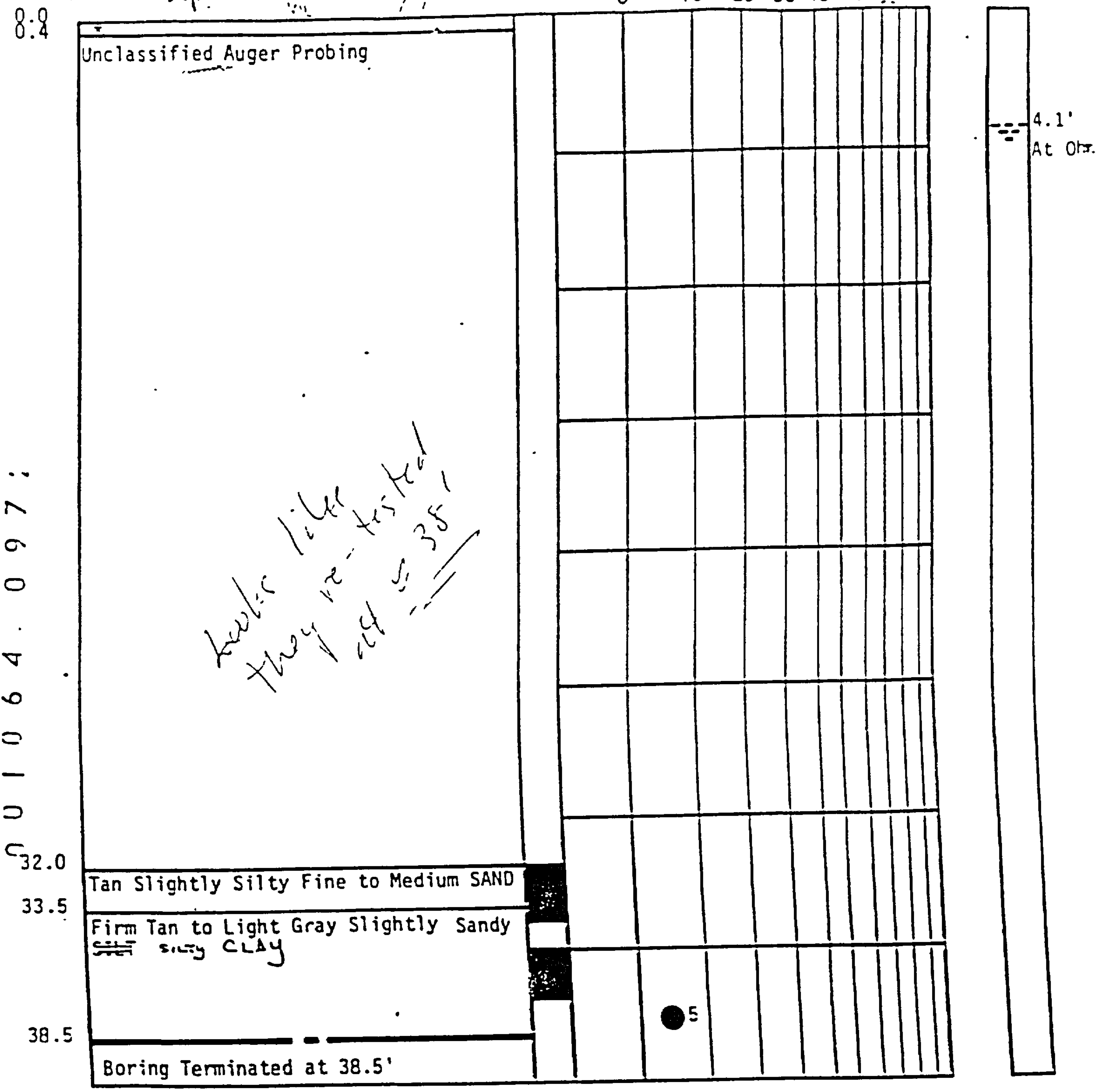
WATER TABLE-24HR
WATER TABLE-1HR

BORING NO. R-2
DATE DRILLED 12-21-81
JOB NO. RS-1822

SOIL & MATERIAL ENGINEERS, INC.

VR/SNS ~~foundation~~ Analysis
By: B. Smith March '82

Sheet no. 2



Unclassified Auger Probing

looks like they re-tested at ≅ 38'

DEPTH FT.	DESCRIPTION
0.0	
0.4	Unclassified Auger Probing
32.0	Tan Slightly Silty Fine to Medium SAND
33.5	Firm Tan to Light Gray Slightly Sandy ~~SILT~~ silty CLAY
38.5	Boring Terminated at 38.5'

TEST BORING RECORD

BORING NO. RS-2A
DATE DRILLED 1-5-82
JOB NO. RS-1823

SOIL & MATERIAL ENGINEERS, INC.

BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB HAMMER FALLING 30 IN REQUIRED TO DRIVE 1.4 IN I D SAMPLER 1 FT

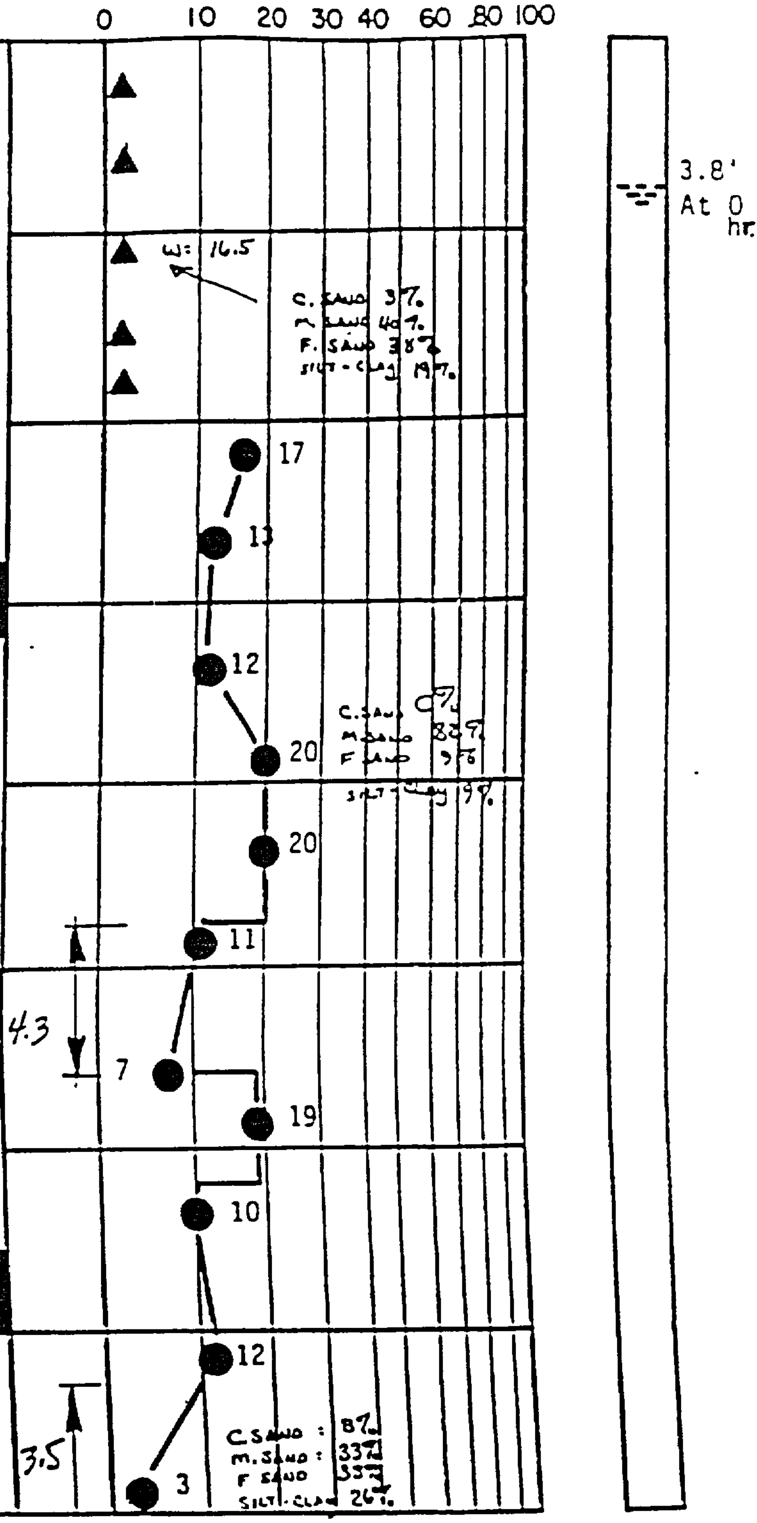
- ▆ UNDISTURBED SAMPLE
- |50|% ROCK CORE RECOVERY
- ◀ LOSS OF DRILLING WATER
- WATER TABLE-24HR
- WATER TABLE-1HR

9.VP-RA/66-0013 Page 17 of 24

DEPTH FT. | DESCRIPTION | ELEV. | ● PENETRATION-BLOWS PER FT.

By: B Shield March '82

DEPTH FT.	DESCRIPTION
0.0	
0.8	CONCRETE SLAB
	Brown Slightly Silty Fine SAND (Fill)
3.0	Light Orange Silty Fine to Medium SAND
8.5	Firm White Silty to Clayey Fine to Medium SAND
15.0	Firm Light Orange Slightly Silty ~~Fine~~ SAND (Fine Sand and Medium)
	Loose Light Gray with Maroon Slightly Clayey Fine to Coarse SAND
28.0	Stiff Light Gray to White Slightly Clayey SILT with Thin Sand Layers
31.0	Loose to Firm Brown to Tan Slightly Silty to Silty Fine to Medium SAND with Occasional Silty Layer and Cemented zones
40.0	



BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113

PENETRATION IS THE NUMBER OF BLOWS OF 140 LB HAMMER FALLING 30 IN REQUIRED TO DRIVE 1.4 IN I D SAMPLER 1 FT

▨ UNDISTURBED SAMPLE
▭ % ROCK CORE RECOVERY
◀ LOSS OF DRILLING WATER
═ WATER TABLE-24HR
═ WATER TABLE-1HR

TEST BORING RECORD

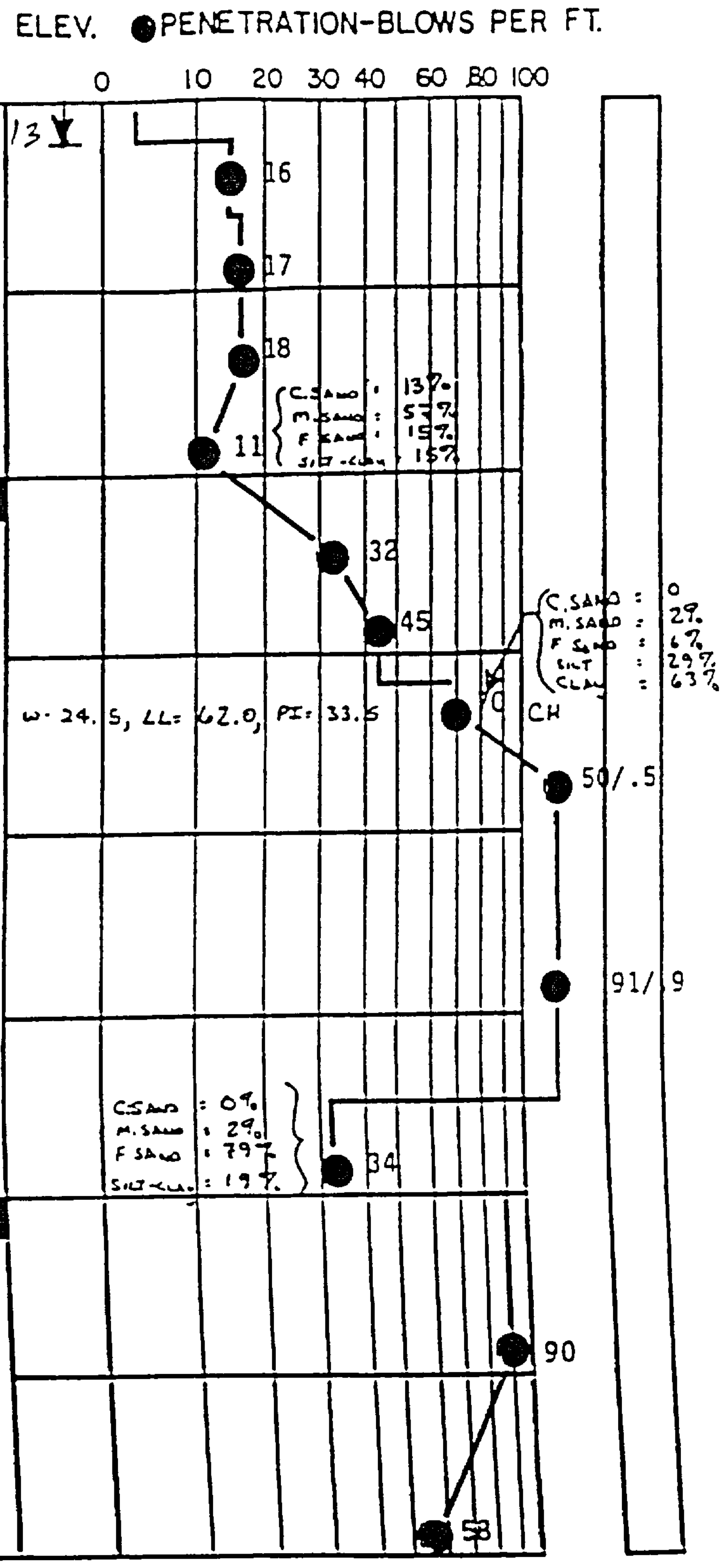
BORING NO. R-3
DATE DRILLED 12-16-81
JOB NO. RS-1823

SOIL & MATERIAL ENGINEERS, INC.

VR/SHS Foundation Analysis
By: B Shields March '82

Sheet no. 22

DEPTH FT.	DESCRIPTION
40.0	
41	(See previous page for description)
	Firm Tan and Maroon Thinnly Layered Fine to Medium SAND and SILT
43.0	Firm Brown and Dark Red Silty Fine SAND with some cemented zones
45.0	Firm Tan Slightly Silty Fine to medium SAND
48.0	Firm Brown Silty Fine to Coarse SAND
51.0	Dense Pink to Tan Slightly Silty Fine SAND
56.0	Very Hard Light Gray with Maroon Slightly ~~Clayey SILT~~ FINE SANDY SILTY CLAY
67.5	Dense Tan Silty Very Fine SAND
70.0	Very Hard Dark Purple to Brown Slightly Clayey SILT
80.0	



TEST BORING RECORD

BORING NO. R-3
DATE DRILLED 12-16-81
JOB NO. RS-1823

SOIL & MATERIAL ENGINEERS, INC.

BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113

PENETRATION IS THE NUMBER OF BLOWS OF 140 LB HAMMER FALLING 30 IN REQUIRED TO DRIVE 1.4 IN I.D. SAMPLER 1 FT

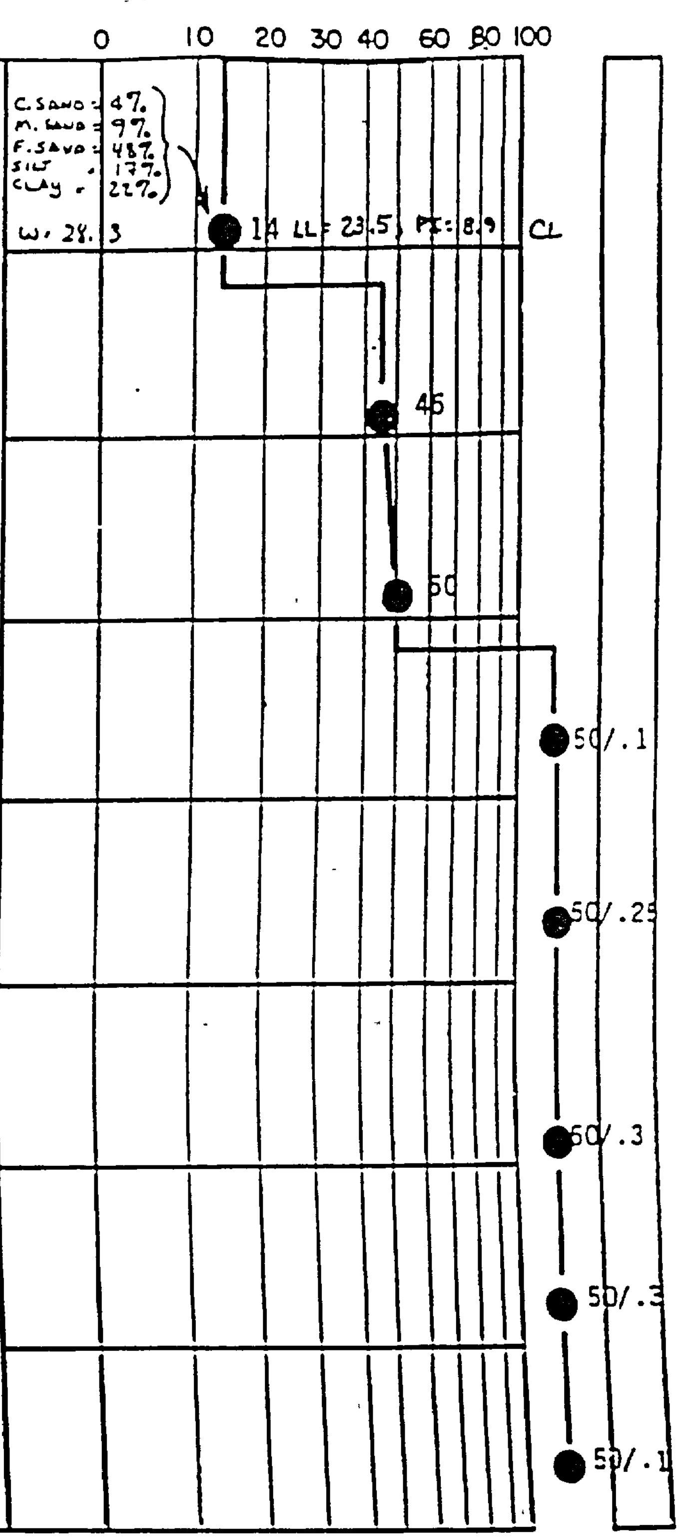
■ UNDISTURBED SAMPLE
50% ROCK CORE RECOVERY
◀ LOSS OF DRILLING WATER
WATER TABLE-24HR
WATER TABLE-1HR

SNP-RF/49-0013 Page 19 of 34

~~Site~~ VR/SGS ~~Foundation~~ Analysis
By: B. Shift March '82

DEPTH FT.	DESCRIPTION
80.0	Stiff Brown S~~lightly Fine Sandy and Clayey SILT~~ SILTY CLAYEY FINE SAND
86.0	Dense White and Maroon Silty to Slightly Silty Fine to Medium SAND
96.0	Gray Highly Fractured SILTSTONE
99.0	Very Hard Light Gray with Maroon Fine Sandy SILT
105.0	Very Dense Light Pink Silty Fine SAND
112.0	Very Hard Light Gray Fine Sandy SILT 9.1' WITH BLOW COUNTS < 10 @ 1.5% ⇒ 1.6" @ 2% ⇒ 2.2"
120	

Boring Terminated at 120 feet



BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113

PENETRATION IS THE NUMBER OF BLOWS OF 140 LB HAMMER
ING 30 IN REQUIRED TO DRIVE 1 4 IN I D SAMPLER 1 FT.

▬ UNDISTURBED SAMPLE	☰ WATER TABLE-24HR
50% ROCK CORE RECOVERY	☰ WATER TABLE-1HR
◀ LOSS OF DRILLING WATER	

TEST BORING RECORD

BORING NO. R-3
DATE DRILLED 12-16-81
JOB NO. RS-1823

SOIL & MATERIAL ENGINEERS, INC.

QEC H.D. [illegible]

VR/SMS Foundation Analysis 2-

By P. Shelly March '82

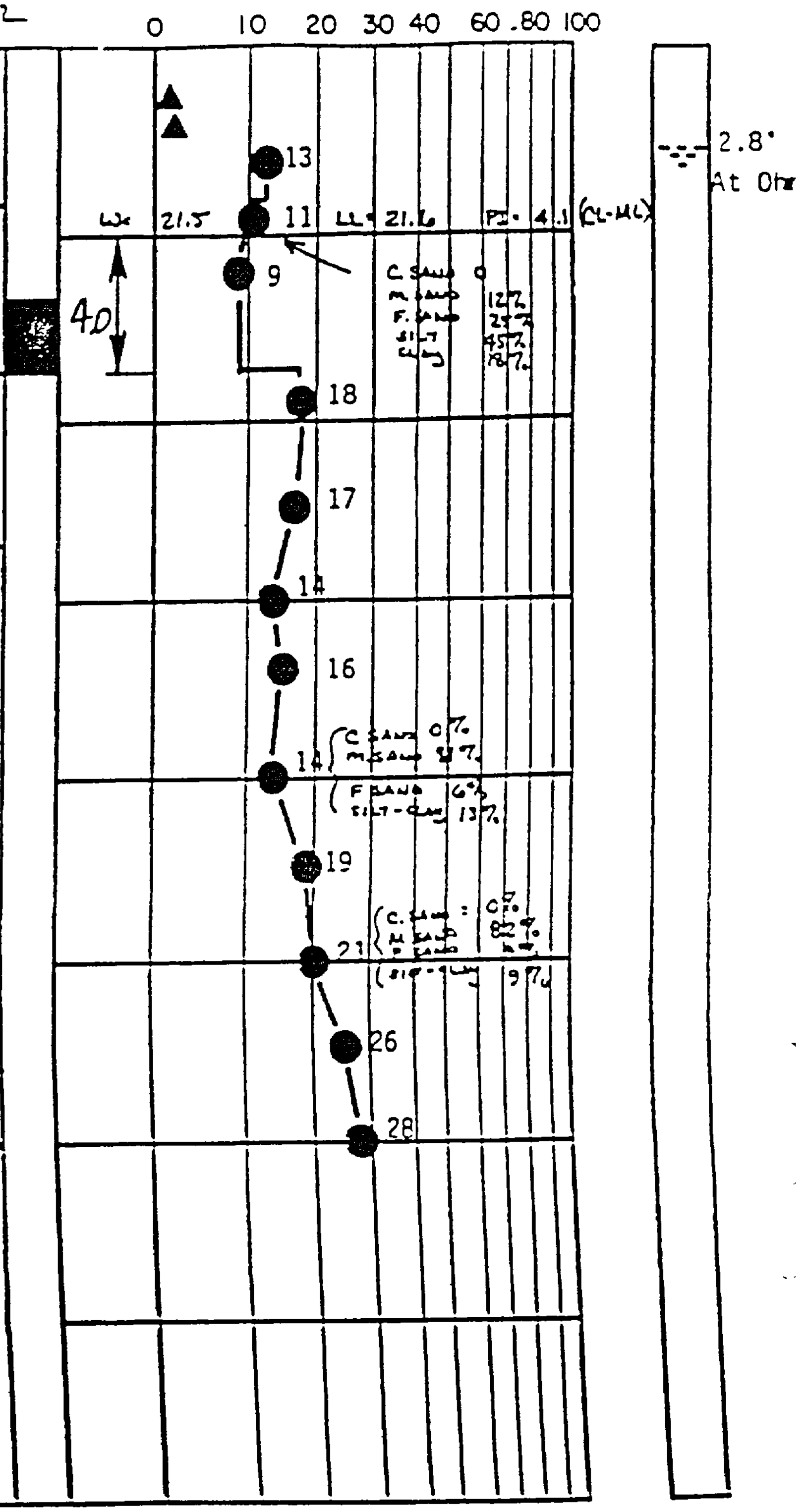
▲ Jar Sample from Hand Auger

● PENETRATION-BLOWS PER FT.

DEPTH FT.	DESCRIPTION
0.0	Firm Brown to Orange Silty Fine SAND
4.0	Stiff Orange to Pink Thinnly Bedded Slightly Clayey and Sandy SILT
8.5	Firm White to Orange Slightly Silty Fine to Medium SAND
13.5	Firm Tan To Orange Slightly Silty AND Fine ~~SAND~~ SANDY MEDIUM SAND
21.0	Firm Orange Slightly Silty Fine to Medium SAND
24.0	Very Firm Tan Slightly Silty AND Fine SANDY ~~SAND~~ MEDIUM SAND
26.0	Very Firm Tan Slightly Silty Fine to Medium SAND
28.0	Very Firm Tan Slightly Silty AND Fine SANDY ~~SAND~~ MEDIUM SAND
30	Boring Terminated at 30 feet

Note: After completion of Soil Boring, 2". PVC piezometer installed, screen depth 10 ft. to 20ft.

4' WITH BLOW COUNTS < 10
@ 1.5% ⇒ 0.7"
@ 2% ⇒ 1.0"



BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113

PENETRATION IS THE NUMBER OF BLOWS OF 140 LB HAMMER FALLING 30 IN REQUIRED TO DRIVE 14 IN I D SAMPLER I FT

▆ UNDISTURBED SAMPLE
|50|% ROCK CORE RECOVERY
◀ LOSS OF DRILLING WATER
WATER TABLE-24HR
WATER TABLE-1HR

TEST BORING RECORD

BORING NO. B-4
DATE DRILLED 12-23-81
JOB NO. BS-1823

SOIL & MATERIAL ENGINEERS, INC.

[illegible]/14-2013 Page 21 of 24