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MEMORANDUM

To: Chief, Dam Safety Office
Attention: 84-44000

From: Katie Bartojay
Materials Engineering and Research Laboratory Group

Subject: Dam Safety Technology Development Report DSO-07-09 - Evaluation of In Situ Methods for Liquefaction Investigation of Dams

Attached for your use is the *Evaluation of In Situ Methods for Liquefaction Investigation of Dams, DSO-07-09* the Dam Safety Technology Development Program that has been prepared by the Technical Service Center at the request of the Dam Safety Office.

This transmittal concludes the work on the Technology Development Report. Additional copies of reports in Adobe Acrobat Format are available upon request and will also be loaded into DSDAMS. If you have any questions, please contact me at 303-445-2374 or at kbartojay@do.usbr.gov.

Attachments

cc: 94-46000 (Brynda) (w/o att)
84-44000 (Becker, Snorteland, Krosley) (w/att)
84-21300 (Archives) (originals)
86-62000 (Hensley) (w/att)
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RECLAMATION
Managing Water in the West

Report DSO-07-09

Evaluation of *In Situ* Methods for Liquefaction Investigation of Dams

Dam Safety Technology Development Program

U.S. Department of the Interior
Bureau of Reclamation
Technical Service Center
Denver, Colorado

October 2007

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14. ABSTRACT This study uses the literature on liquefaction in dams to compare and contrast the various *in situ* methods of determining embankment dams' susceptibility to liquefaction. The study concludes that three soil penetration tests usually used to measure soil density, the principle factor in liquefaction, predict liquefaction fairly reliably when used in combination with other tests and with good data and sound engineering judgement. The study lists advantages and disadvantages of each method and points out conditions where a method may be better (or less) suited to particular site conditions.						
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Technical Service Center, Denver, Colorado
Geotechnical Engineering Group 3, 86-68313

Report DSO-07-09

Evaluation of *In Situ* Methods for Liquefaction Investigation of Dams

Dam Safety Technology Development Program
Denver, Colorado

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Mission Statements

The mission of the Department of the Interior is to protect and provide access to our Nation's natural and cultural heritage and honor our trust responsibilities to Indian Tribes and our commitments to island communities.

The mission of the Bureau of Reclamation is to manage, develop, and protect water and related resources in an environmentally and economically sound manner in the interest of the American public.

Acknowledgments

This research was sponsored by the Dam Safety Technology Development Program of the Bureau of Reclamation.

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Executive Summary

It has been nearly 35 years since the near-failure of the Lower San Fernando Dam in California, an event which certainly brought widespread attention to the potential for liquefaction-induced failure of embankment dams. Since the work in the early 1970s on predicting liquefaction susceptibility in soils by H. Bolton Seed and colleagues, continuing research and evolving applications have led to the use of a number of *in situ* techniques and analytical tools. For embankment dams, these are used to evaluate whether embankment or foundation soils have the potential to liquefy or undergo severe strength loss during earthquake loading. This Bureau of Reclamation (Reclamation) study included a review of numerous key papers or reports that:

- Outline the evolution and current state-of-the-practice in the evaluation of soil resistance to liquefaction by means of *in situ* tests

- Present actual data from *in situ* investigative procedures such as the standard penetration test (SPT), the cone penetration test (CPT), the Becker penetration test (BPT), measurement of shear-wave velocity (V_s), as well as the determination of in-place density.

The second of these tasks is to compare the various *in situ* techniques, not just to each other, but to measured densities as well. It is generally accepted that soil density (described in terms of relative density or related parameters) is the principal factor that determines the liquefaction resistance of a saturated soil. It is not, however, the only potentially significant factor. Four additional factors recognized as having influence in liquefaction resistance are: (1) soil fabric or structure (related to mode of deposition), (2) aging (time under sustained loading) and/or cementation effects, (3) degree of overconsolidation, and (4) previous cyclic loading history. Thus, correlations strive to correlate *in situ* measurements with the occurrence of liquefaction, as opposed to correlating solely to density. Nevertheless, it is believed that a good correlation with density (which is the most important factor in determining a soil's liquefaction resistance) does lead to increased confidence in an *in situ* technique's ability to predict liquefaction potential.

This study compares and contrasts the various *in situ* methods, listing advantages and disadvantages of each as well as pointing out conditions where a method may be better (or less) suited to particular site conditions.

Key Findings

A complete list of findings is included *Summary of Findings* on page 116 of this report. For the purposes of an executive summary, the following key observations are made:

- *In situ* investigation techniques such as the SPT, CPT, BPT, and measurement of V_s are found to be industry-accepted procedures for evaluation of the liquefaction resistance of soils. These methodologies are based on an extensive database of sites where liquefaction or no liquefaction occurred under earthquake loading and are thus considered to be a realistic representation of field behavior (although they are not without uncertainties). Reclamation generally follows state-of-the-practice procedures for these methods.

- There are a number of reasons why the measurement of in-place densities and relative densities are problematic and subject to uncertainties. Furthermore, density does not singularly determine either the penetration resistance or shear-wave velocity, or the soil's liquefaction potential. The secondary, but important, factors of soil fabric, gradation, degree of overconsolidation, aging, and prior cyclic loading history also have significant influence (as discussed later in the report). Therefore, *in situ* methodologies do not explicitly correlate measurements to relative density. However, correlations do exist, and a limited number of field performance case histories actually have in-place density data available. Based on the case history data, it can be concluded with reasonable confidence that the *in situ* techniques correlate reasonably well with measured relative densities.

- In addition, a study of case histories indicates that the predictions of liquefaction resistance by various *in situ* techniques correlate fairly well with each other. The three penetration techniques (SPT, CPT, and BPT) generally show good correlation to each other when reasonably good data are used. Shear-wave velocity measurements do not correlate quite as well to the penetrometers, generally showing a fair or fair to good correlation. Because shear-wave velocity testing causes only small strains and minimum disturbance in soils, it is strongly affected by soil "aging" effects such as even small amounts of cementation among soil particles (as discussed in more detail later in the report). This appears to be a major reason why the shear-wave measurements do not correlate as well.

- Without question, there is significant uncertainty in predictions from any *in situ* technique for liquefaction potential, as there is with almost any geotechnical engineering evaluation. Because of the uncertainty, analysts and decision makers should always base conclusions and decisions on the engineering analyses tempered by good judgment and a full understanding of the mechanics of the procedures.

Key Recommendations

A complete summary of recommendations is listed near the end of this report under *Recommendations*. A few key recommendations for future Reclamation liquefaction evaluations are listed below:

- At most sites, it is prudent to utilize more than one *in situ* technique to evaluate liquefaction potential. In light of uncertainties with all methods, multiple techniques can add some confidence to the conclusions of liquefaction susceptibility, or at least highlight the uncertainties therein (which must be accounted for in analysis and decision making).

- At least one of the exploration techniques should include a means to provide samples of the soils being evaluated. Samples allow for visual classification and laboratory testing of physical properties without having to infer these properties from other indirect *in situ* measurements. Engineers tasked with liquefaction evaluations are strongly encouraged to view, as well as test (particularly for fines content), any available samples from exploration programs.

- SPT and CPT are the most appropriate techniques for soils with minor gravel content. Measurement of V_s can be used for verification as well as determining material properties for input into dynamic response analyses.

- At sites containing gravels, BPT and V_s, and possibly SPT, are generally the most appropriate techniques. Careful use of the SPT with a technique such as short-interval sampling, or otherwise selective testing and correction for gravel content, can provide a means of verifying that BPT data (which might be gathered in much greater volume) are valid at a given site. As the soils being tested become progressively coarser and approach cobble size, use of any technique becomes problematic.

- In general, it is preferred to limit the use of these investigative procedures to the depths and conditions under which the correlations were developed. (Most techniques have limitations such as depth and age of soils, which are discussed in more detail in later sections of this report.) When dealing with massive structures such as earth dams, however, it is often necessary to extrapolate these methods and correlations to greater depths and higher overburden stresses than were present in the generally shallower soils involved the field case histories used to develop the correlations. Caution, and engineering judgment based on the mechanics of the test are needed when extrapolating these methods to higher overburden stresses or significantly older geologic deposits.

- It is critical to conduct a sufficient number of tests or borings that a reasonably representative sampling of the soils is obtained. This becomes even more important when a critical layer is thin (limiting the testing intervals from a single boring) or when the soils are particularly interfingered, lensed, or otherwise heterogeneous.

- Because Reclamation relies heavily on risk analyses for evaluating whether liquefaction potential could lead to dam failure, the various correlations for estimating probability of liquefaction from *in situ* data are of particular interest. It is often prudent to compare two or more of the available probabilistic models to determine the effect of model choice on conclusions about liquefaction potential.

- Although the focus of this study is *in situ* investigation techniques, it is important to remember that a thorough review and study of all existing data are important in any investigative process. Such data often include:

    - Photographs and maps of foundation materials

    - Laboratory or field testing of important materials

    - Logs of drill holes or test trenches

    - Visual descriptions of foundation materials exposed during construction

    - Geology and site history

Background and Scope of Work

Since the Reclamation Safety of Dams Act of 1978 was passed, Reclamation has been involved in a large number of seismic evaluations of embankment dams, some of which have led to structural modifications. During the last 30 years, other federal and State agencies and consultant firms have performed similar evaluations. In addition, universities and government agencies have conducted research into various techniques for determining the nature of embankment and foundation soils as they pertain to seismic stability. Consequently, numerous geotechnical investigations have been undertaken on existing or proposed civil engineering structures such as embankment dams, buildings, roads, and bridges.

Reclamation, like other entities, has attempted to approach new investigations by incorporating lessons learned from previous exploration programs, both those by Reclamation and those by others published or discussed in technical seminars and conferences. However, no single comprehensive look at Reclamation seismic investigation programs had previously been undertaken. Thus, in February of 2004, Reclamation's Dam Safety Office requested the Technical Service Center to examine field exploration data from Reclamation and other entities to determine the overall experience with, applicability of, and consistency among various seismic investigation techniques such as standard penetration tests, cone penetration tests, Becker penetration tests, shear-wave velocity measurements, and in-place density tests. Specifically, the scope of the work of this research effort was to:

- Gather information and data from Reclamation designers and field staff involved in dam modifications that featured more than one seismic investigation technique, and ideally included in-place density testing

- Gather the same type of information, as well as general feedback and experience, from outside entities including other government agencies, private consultants and architect-engineer (A/E) firms, and academic institutions

- Compile the gathered information into a report that summarizes both Reclamation and general industry experiences with the various investigation methods, and discusses the ability of these methods to assess embankment and foundation conditions, specifically liquefaction potential

- Offer recommendations for any new approaches in data collection, as well as providing advice on which methods may be best suited for given site conditions

- Have the draft report reviewed within Reclamation and by an outside board of independent consultants with significant experience in seismic investigations

Throughout this document, the term "*in situ* techniques" is used to describe testing methods such as the SPT, CPT, BPT, and various means of measuring shear-wave velocity that are used to test the ground in its natural state. These four methods are, at present, the principal *empirical* means of evaluating liquefaction, as opposed to laboratory testing. In-place density testing can be considered an *in situ* technique, but it is not specifically covered in the current empirical evaluation procedures. Relative density of soils is discussed in this report, particularly as it relates to correlating liquefaction potential to the *in situ* techniques.

It is important to note a few limitations to the scope of this study:

- The investigation methods being evaluated are limited to *in situ* techniques (and primarily North American techniques), as discussed above. Evaluation of sampling and laboratory testing of undisturbed (and disturbed) soils for the determination of seismic properties and liquefaction potential are beyond the scope of this study.

- This study concentrates on liquefaction "triggering," and does not deal with the response of structures to liquefaction, including the determination of postliquefaction strengths and potential for slope failures.

- Additionally, this study focuses on predicting the resistance of soils to liquefaction under earthquake loading, and not the details of the earthquake loading portion of the liquefaction analysis. Earthquake magnitudes, peak ground accelerations, durations, and other properties are very important to liquefaction evaluation but are largely outside of the scope of this study.

- This study does not deal with the topic of liquefaction resistance of "moderately cohesive" soils. This is a rapidly evolving area of seismic dam engineering, where methods for assessing liquefaction potential have not yet gained wide acceptance.

- The focus of the study is for embankment dams and their foundations, as opposed to other civil/geotechnical engineering structures. Although quality seismic investigation data are available for other types of projects, only limited amounts were obtained during the relatively short period of data collection for this study.

- This study is not a highly comprehensive evaluation of data and techniques; it is instead a general overview intended to verify that Reclamation is (or is not!) appropriately applying industry-accepted techniques and lessons learned from previous projects.

Data Gathering

In order to get broad-based input, a number of different offices and individuals throughout North America were contacted. Initially, contact was made by phone, explaining the nature of this study, and our interest in relating properties from *in situ* techniques (SPT, CPT, etc.) to measurements of relative density or relative compaction (as measured by in-place densities and laboratory tests). After this introduction, an email was sent listing the types of data being sought. Specifically, the request asked for:

- Direct comparisons between densities measured in excavations against penetration resistance or shear-wave velocity

- Any sites where liquefaction potential was assessed with *in situ* techniques and subsequently an earthquake occurred (Class A prediction)

- Side-by-side comparisons of *in situ* techniques at the same site to see if various methods give similar predictions

In general, if individuals had data, they transmitted them to Reclamation in the form of a technical report or published paper.

A complete listing of the data, reports, and papers received or viewed during this study is included in *References*. The vast majority of these references came from the data collection effort, while the remainder were from various Reclamation contacts or government libraries. Sources of data external to Reclamation included other government agencies, including federal, State, and other countries. Universities and professors, particularly those who have researched and published in this area, were also contacted, along with private consultants and A/E firms involved in liquefaction evaluation.

Description of *In Situ* Investigation Methods

Recent literature describing the state of practice in liquefaction evaluation indicates general agreement in noting that four methods are most widely used and accepted. These four are SPT, CPT, BPT, and shear-wave velocity (SWV or V_s) testing. These four mainstays provide the majority of the available data on liquefaction resistance of soils and are the focus of most of the technical papers on the subject. Each of these four methods is described in some detail below, including the current generally accepted procedure or methodology (the state of practice for each method). The section on *Application of In Situ Methods* discusses some variations in applications and provides some specific applications, limits, and cautions in the use of these techniques.

Brief Discussion of Cyclic Stress Ratio

The focus of this document is predicting the resistance of soils to liquefaction under earthquake loading from *in situ* measurement, not detailed discussion of the loading an earthquake would impose on the soils. However, a brief discussion is provided here to provide the reader with a general sense of the key factors involved, as well as the potential uncertainties involved in this step. This discussion is provided before any discussion of the four main *in situ* methods, as this loading is used in all four.

The severity of earthquake loading is generally represented in liquefaction assessments by the cyclic stress ratio (CSR). In essence, CSR is defined as the ratio of the "average" cyclic shear stress on a horizontal plane (τ_{avg}) to the preearthquake vertical effective stress (σ_v'). Since most of the earthquake loading cycles would be less than the peak value, the average shear stress is defined as 65 percent of the peak shear stress, or $\tau_{avg} = (0.65)(\tau_{max})$. The peak shear stress is estimated from the earthquake's peak horizontal ground acceleration (a_{max}), the total and effective overburden stresses (σ_v and σ_v'), and an empirical reduction factor that adjusts for the dynamic response of the soil profile (r_d). The CSR is calculated as:

$$CSR = [(0.65)(a_{max})(\sigma_v)(r_d)] / \sigma'_v$$

Further adjustments are made to the CSR to account for earthquake magnitude, the effect of high normal stress, and the effect of sloping ground. Since larger earthquakes tend to provide more cycles of strong loading, a correction factor K_M is used to adjust the CSR to that expected from a magnitude 7.5 earthquake. A correction factor K_σ is applied to account for the tendency of soils to behave more contractively under higher confining stresses (for a given relative density). In addition, the correction factor K_α is applied to account for the effect of nonzero horizontal stresses on liquefaction resistance (The effect may be harmful or beneficial depending on the situation). These adjustments to the CSR are shown as:

$$CSR_{7.5} = CSR \div [(K_M)(K_\sigma)(K_\alpha)]$$

The mechanics behind these adjustments are not precisely understood, and there is some controversy about each of them. Uncertainties in their values produce uncertainty in the results of liquefaction evaluation, which must be recognized and accounted for. For more details on CSR development and the application of correction factors, the reader may wish to review the 1997 publication by the National Center for Earthquake Engineering Research (NCEER) or Reclamation's design standard on *Seismic Design and Analysis* (Gillette, 2001).

The ability of a soil to resist liquefaction is frequently referred to as the cyclic resistance ratio (CRR); it is the maximum CSR to which the soil could be subjected without occurrence of liquefaction.

Standard Penetration Test

General Description of Test and Equipment

The SPT consists of using a 140-pound hammer to drive a split-barrel sampler in a drill hole and recording the number of 30-inch drops of the hammer required to drive the sampler 1 foot. Drill rigs used and types of drilling procedures to advance a hole during SPTs vary considerably. However, open-hole rotary drilling with drill mud is the *de facto* standard, in order to minimize disturbance. This can be done with drag or fishtail bits, or preferably, roller cone bits with the fluid discharge deflected to avoid disturbance. Hollow stem continuous flight augers with hole advancement by dry-coring methods have been used with success, but caution is required. Procedures that create more soil disturbance are not acceptable; these would include wash boring, cable tools, and casing advancement with down-hole hammer. With any method, great care must be taken to minimize soil disturbance, whether from the drilling method, from heaving or flowing sand at the bottom of the hole caused by failure of fluid pressure to balance the pore-water pressure in the soil immediately below the bottom, or from suction during extraction of the sampler or tools for advancement of the hole between test intervals. All aspects of the drilling operation must be conducted with careful attention to detail and proper procedure.

Drill rod sizes should be selected to be as consistent as is practical with the equipment used for the blow counts in the liquefaction database. For depths of less than 50 feet, flush joint steel AW (1.75-inch outside diameter) or similar rods are recommended. For depths exceeding 50 feet, flush joint BW (2.125-inch outside diameter) or NW (2.625-inch outside diameter) sizes are preferred. Reclamation uses NW rods exclusively. Figure 1 shows a typical split-barrel sampler. The recommended sampler is at least 18 inches long, and has an outside diameter of 2 inches and an inside diameter of either 1.375 or 1.5 inches. The latter provides space for a liner behind the drive shoe (which has an inside diameter of 1.375 inches), although liners are not commonly used. If a 1.5-inch inside diameter sampler is used without a liner, a correction factor of 1.1 to 1.3 (depending on density) is typically applied to the recorded blow count to account for the reduced friction on the inside of the sampler.

Commonly used methods of dropping the hammer include various rope-and-cathead systems (fig. 2) and automatic trip hammers (fig. 3). The preferred configuration for a rope-and-cathead system is a "safety" hammer with two turns of rope on the cathead (plus or minus one quarter turn). It has been found to deliver about 60 percent of the theoretical energy of the hammer (140 lb x 30 in) to the drill rods; this is used as the "standard" energy for liquefaction evaluation. Blow counts are adjusted to the equivalent blow count by simple proportion with the standard 60-percent energy. Regardless of which system is utilized, energy should be measured before or during testing so that the adjustment can be applied with the best available values. Even with the standard configuration, the amount of energy is very sensitive to variations in stroke length, rope condition, sheaves at the top of the mast, etc.

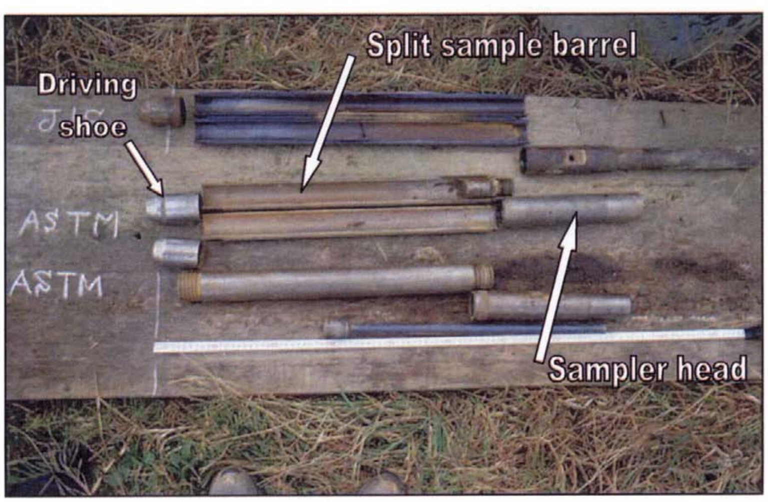


Figure 1.—SPT samplers.

Figure 2.—Rope and cathead system for SPT.

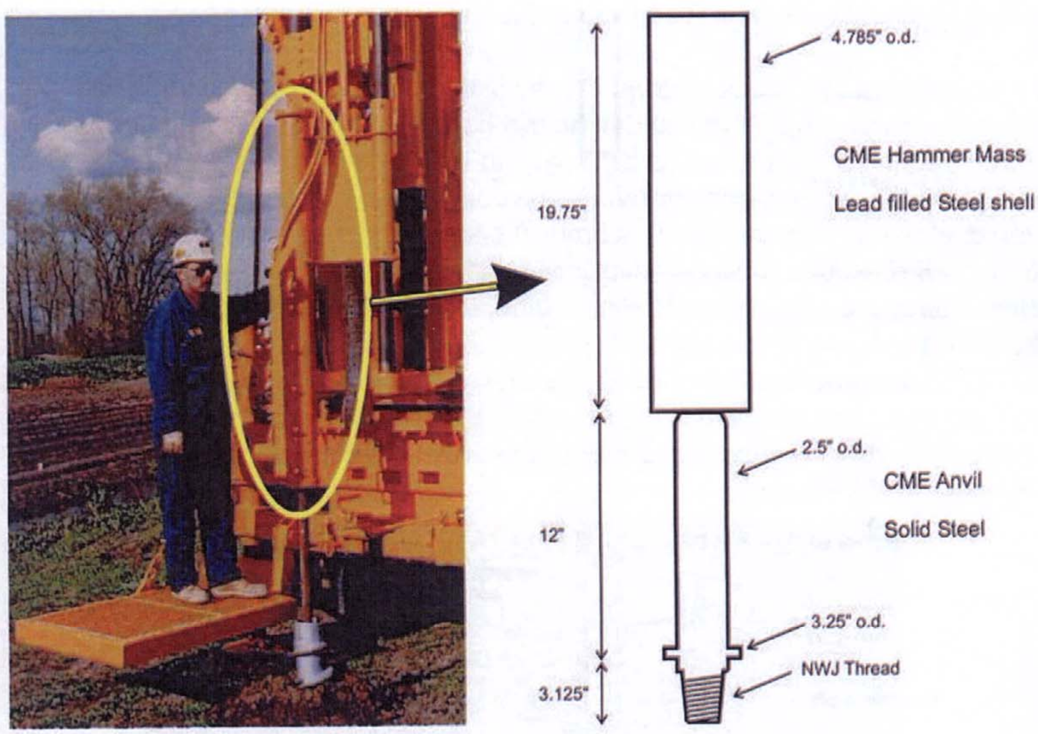


Figure 3.—Automatic hammer for SPT.

Data Collection and Reduction

In an SPT test, the sampler is driven 1.5 feet for each test, and the blow counts are recorded for each 6-inch interval. The first 6-inch interval is referred to as the seating interval. Blow counts are recorded in the seating interval for comparison, but are not actually considered as data for the liquefaction evaluation. The second and third 6-inch intervals constitute the actual penetration test, and the sum of the blow counts recorded in this 1-foot interval is referred to as N. In addition to recording the summary N-value over the interval, standard Reclamation practice is also to record cumulative penetration after each blow, or else blows per 0.1 foot of penetration. This is important in determining whether gravel is interfering with the SPT penetration, and often permits correction of the N-value for gravel interference. For testing and correcting for gravel interference, the cumulative blows are plotted as shown in figure 4. If the curve steepens abruptly, it is likely that the tip of the sampler has encountered a gravel particle or cobble that is interfering with penetration. The slope of the previous portion of the curve can be projected over a 12-inch interval to get an estimate of the N value that would have been measured without the assumed gravel interference. The presence of the gravel can and should be confirmed by inspection of the sample to ensure the change in penetration rate is not due to variations within finer grained (silt or sand) materials. If increasing penetration is due to a dense layer instead of a gravel particle, inspection of the sample may be able to verify that.

Once a gravel-corrected N value is obtained, a number of additional corrections or adjustments may be needed. These include corrections for hammer energy, effective overburden stress, effects of short drill rods at shallow depths (the significance of which is being studied and debated currently), sample liner, and borehole diameter, to obtain a value called $(N_1)_{60}$. This is the equivalent value that would be measured in the same soil with 1 ton/ft^2 of effective overburden stress, and hammer energy equal to 60 percent of the theoretical energy of the hammer (4,200 in-lb). Combined, the adjustments appear as:

$$(N_1)_{60} = N\, C_E\, C_N\, C_R\, C_S\, C_B$$

where

C_E is the actual transmitted energy divided by 60% of the theoretical energy.
C_N adjusts for the effect of overburden stress.
C_R adjusts for the effect on energy of wave reflections in short rods.
C_S adjusts for the lack of a liner in a sampler with space for one.
C_B adjusts for the effect of a borehole diameter greater than 4 inches.

The adjusted blow count $(N_1)_{60}$ in silty sand or silt can then be converted to the equivalent clean-sand blowout $(N_1)_{60\text{-cs}}$, which is the blow count that would be measured in a clean sand having the same liquefaction resistance. Detailed discussion of each of these adjustments or corrections is beyond the scope of this report. Refer to the 1997 NCEER publication (Youd and Idriss, 1997), the summary report of NCEER (Youd *et al.*, 2001), or Reclamation's design standard on *Seismic Design and Analysis* (Gillette, 2001) for details. A good understanding and thoughtful application of these correction factors is critical in evaluating liquefaction potential. Note that the simplified representation of the

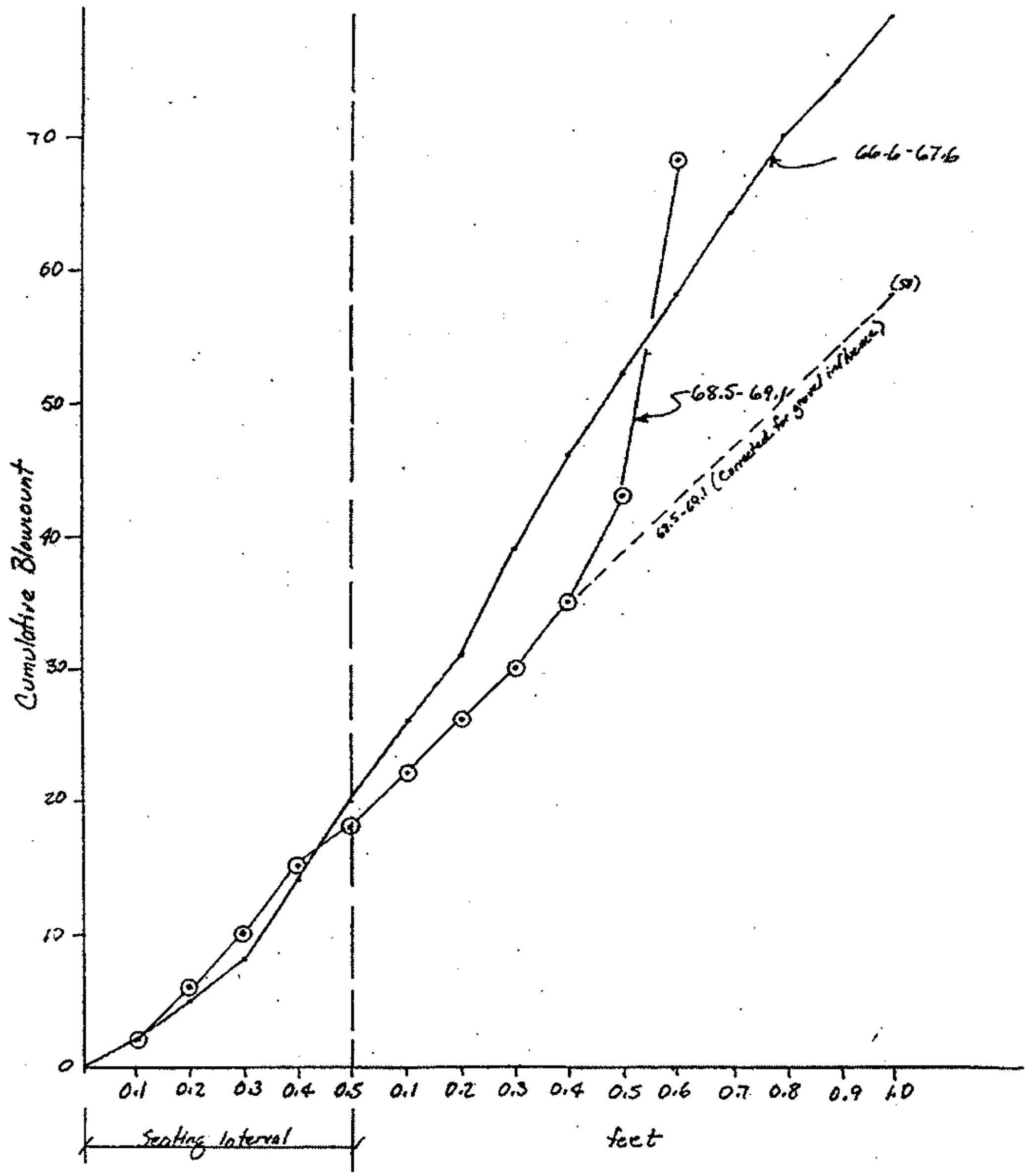


Figure 4.—SPT penetration per blow, indicating potential gravel interference.

earthquake loading by the CSR and the uncertainty in the empirical blow count adjustments suggest that a probabilistic (nondeterministic) framework may be the most appropriate way to use the data (discussed under *Probabilistic Relationships for Use in Risk Analyses*, starting on p. 62).

Evaluation of Liquefaction Potential

The SPT is the most widely used empirical tool to predict liquefaction potential. A correlation first developed by H. Bolton Seed and colleagues has been used for over 30 years with only minor adjustments (This is referred to as the Seed-Idriss or Seed-Lee-Idriss method) (Seed and Idriss, 1970; Seed and Peacock, 1971; Seed and Idriss, 1971). To develop the correlation, the historic earthquake loadings, represented by the CSR, were plotted against the representative corrected SPT blow count $(N_1)_{60}$ of the soil deposits at the sites affected. A family of boundary curves for different fines contents was drawn to separate liquefaction events from nonliquefaction cases, as shown in figure 5. Each curve in the family corresponds to a different fines content (One can also use the fines-adjusted blow count, $(N_1)_{60\text{-}cs}$, with the curve corresponding to 0 to 5 percent fines). For a site under consideration, the expected loading and measured $(N_1)_{60}$ are compared with the applicable boundary curve. A number of steps are required to develop the cyclic stress ratio for the expected earthquake loading, and a number of adjustments made to the raw blow count as described above. However, these details are beyond the scope of this report. The generally industry-accepted version of the triggering boundary curve, developed by the NCEER working group in 1996-97 (Youd and Idriss, 1997), is shown in figure 5; it is modified only slightly from the earlier curves by H. Seed *et al.*

In the 30 years since H. Seed developed his correlation, additional data have been gathered at sites of known earthquake shaking, enlarging the database of cases of liquefaction and nonliquefaction, reportedly to more than 450 case histories. The additional cases were included in a reevaluation and expansion of the database by R. Seed *et al.,* published in 2003. From that work came a new correlation (shown in fig. 6) developed by multivariate regression analysis considering blow count, cyclic stress ratio, earthquake magnitude, fines content, and overburden stress. Refinements in the methodology included rating the quality of the data and weighting them accordingly, improved estimates of peak horizontal ground acceleration for each case history, and a more detailed analysis of the cyclic stress ratio. The latter included a proposed new relationship for estimating the factor r_d, used to estimate CSR from the peak ground-surface acceleration of the earthquake. Other significant differences between the two correlations are discussed below.

The new correlation (R. Seed *et al.*) is not intended to be a deterministic boundary on liquefaction potential. Instead, it represents a liquefaction probability of 20 percent, as estimated by multivariate regression on variables that affect liquefaction potential, such as blow count and fines content.

Another difference (one potentially important for dam analyses) is that the earlier (NCEER-adopted) correlation was based on the "simplified" estimation of *in situ* CSR for each field case history using the r_d curve from Seed and Idriss (1971). This r_d curve has been shown to be biased, relative to actual site-response analyses for calculation of CSR. Use of that correlation should be unbiased when the same procedure is used for CSR, but it is unconservative when site-response analyses are used. (The NCEER-adopted correlation is therefore used most correctly with the simplified CSR estimation using the original r_d relationship from Seed and Idriss [1971].) The newer correlation is based on site-response analyses to find the CSR for some sites; CSRs for the remainder were estimated from the new r_d relationship based on a large number of site-response analyses. It thus provides results that are fully compatible (unbiased) with performance of full

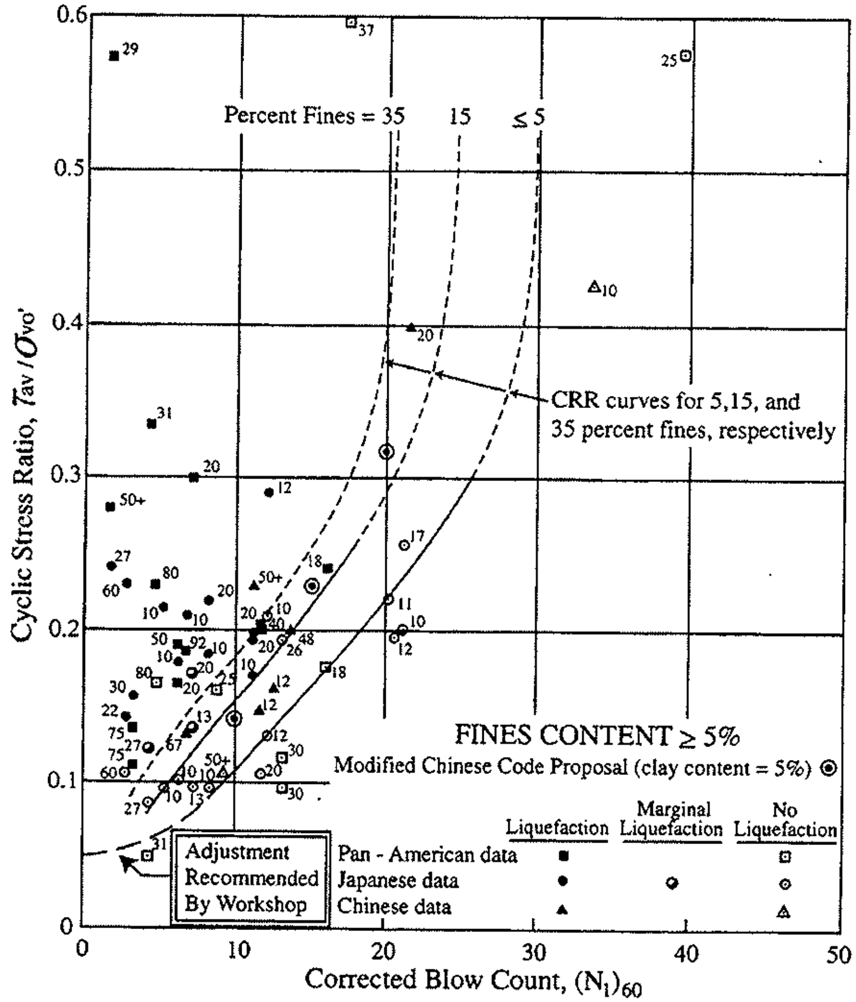


Figure 5.—SPT-based liquefaction triggering relationship from NCEER (1997).

response analyses. Since dams are often not well suited to simple one-dimensional estimates of CSR, and Reclamation seismologists usually provide ground motions tailored to the site and type of faulting involved, this newer approach appears to be a superior approach for embankment dams. Note however, that regardless of bias, there is a wide range of variability within the range of the new r_d curves, which results from wide variation in earthquake ground motions and site properties assumed in the response analyses.

Relatively few data points from sites with high CSR values were available when the original correlation was developed, whereas the new correlation includes many more with CSR values greater than 0.3; thus, it is better constrained at high CSRs. The new correlation also leans to the right at high blow counts (instead of being asymptotic to a vertical line); it is therefore somewhat more conservative than the earlier correlation in this range. (Note that this has only limited influence on dam evaluations. The meaning of "liquefaction" at such high blow counts is likely limited to high excess pore pressure, and the postliquefaction residual

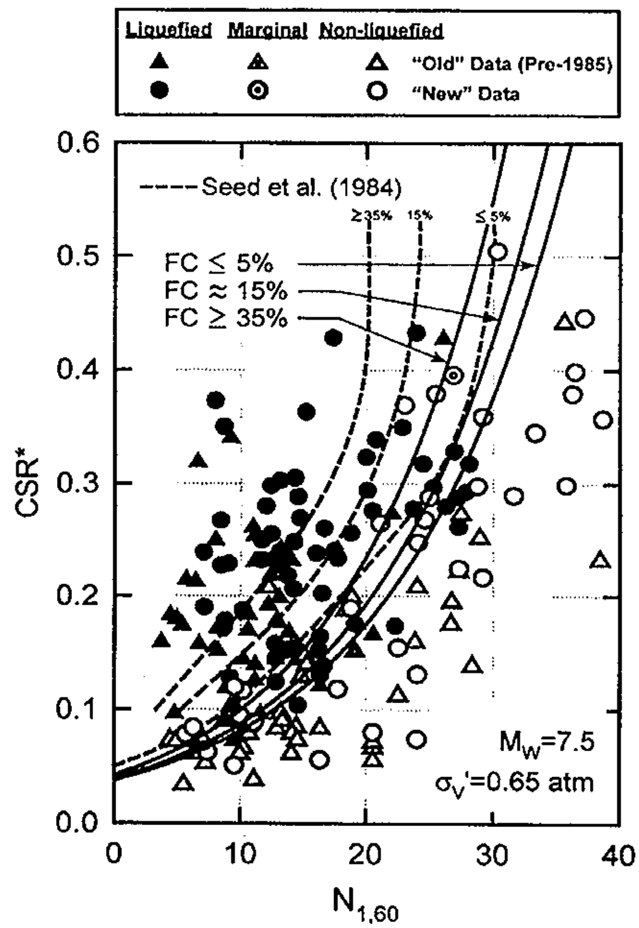


Figure 6.—SPT-based liquefaction triggering relationship from Seed *et al.* (2003).

strength would be very high, limiting the strain that could occur. Furthermore, it may simply be an artifact of the statistical model where there are few data to constrain it.)

The fines correction in the new correlation is somewhat smaller, indicating less "benefit" from silty fines. This change was due in part to the new field case histories since 1984, in addition to the new approach to determining the effect of fines (multivariate regression).

Since this latest correlation is still relatively new, it is appropriate to compare the results to those that would be obtained using NCEER-adopted correlation, with consideration of the differences cited above.

Cone Penetration Test

General Description of Test and Equipment

The cone penetration test, or CPT, is sometimes also referred to as the electric cone penetration test, or ECPT, to distinguish it from the earlier mechanical cone penetrometer or Dutch cone. The CPT is useful in most fine-grained or sand deposits, but often it is not useful because of gravel interference invalidating the measurement, or the difficulty of penetrating coarse or dense soils. (Many Reclamation dams, particularly those in mountainous terrain, have large amounts of gravel in their foundations, so the CPT cannot be used.) The test procedure consists of pushing a conical penetrometer into the ground at a steady rate of 0.8 in/s (2 cm/s), stopping only to add additional rods to the string, while the resistance is measured by load cells attached to the conical tip and the cylindrical sleeve immediately behind the tip. Thus, the CPT produces a nearly continuous record of penetration resistance, unlike the SPT, which provides a single value of penetration resistance at intervals of 2.5 feet or more. Because of the continuous record, as well as the small diameter of the penetrometer, the CPT can assess the liquefaction potential of thin soil strata more accurately than the SPT (although the tip measurement may require adjustment in thinly layered soils).

A typical cone penetrometer is shown in figure 7. The tip of a standard penetrometer is a 60-degree cone, with a projected end area of 1.55 in^2 (10 cm^2), and a diameter of 1.4 inches (3.6 cm). Penetrometers with a 2.33-in^2 (15-cm^2) tip area are sometimes used in softer materials. The tip is connected to the body of the penetrometer by a load cell that measures the force required to push the cone. Following the tip is a cylindrical sleeve with the same diameter as the cone. This sleeve has a length of 5.3 inches (13.4 cm), which results in a surface area of 23.25 in^2 (150 cm^2), and is also attached by a load cell. These load cells are read and recorded by a computer data-acquisition system, typically at intervals of 0.8 to 2 inches (2 to 5 cm). Although the tip resistance is the main parameter in liquefaction analysis, the ratio of the friction measured on the sleeve to the tip resistance can provide an indication of the material type, sensitivity, and other information.

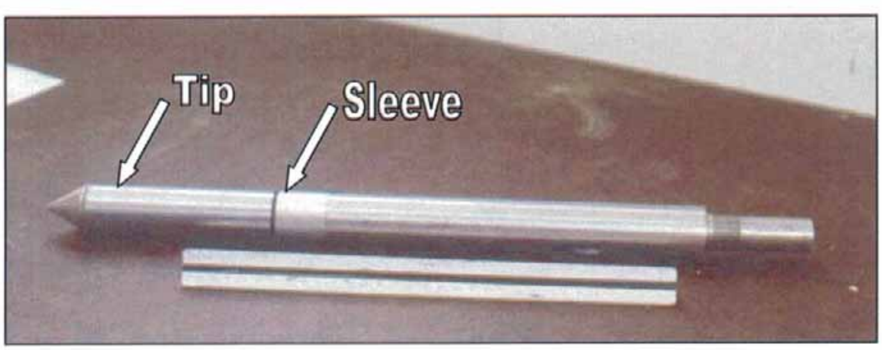


Figure 7.—Typical CPT penetrometer.

CPT penetrometers are frequently equipped with porous elements and a transducer to allow pore pressure measurements; this device is referred to as a piezo-cone. Changes in pore pressure measured during pushing can help in determining stratigraphy. Another variant is the seismic cone, which can be used for downhole measurement of shear-wave velocities, in addition to the usual CPT data. This is discussed in more detail under *Seismic Cone* on page 109.

Most CPT rigs today consist of trucks that contain all pushing equipment and the computer data acquisition system. A typical rig, belonging to Reclamation, is shown in figures 8 and 9.

Because the CPT penetration is a steady push rather than hammering, and data are measured at the tip rather than at the top of the drill rods, there is no concern about wave transmission and energy losses, unlike the SPT and BPT. This is an important advantage of the CPT, as uncertainty in energy transmission for very shallow or very deep intervals complicates interpretation of SPT and BPT. Also, the CPT is generally much faster and less expensive to use than the SPT. Finally, CPT equipment and procedures are much more standardized than the SPT, and less subject to operator variability.

The most obvious disadvantage of the CPT is that it does not retrieve samples of the materials being tested. Therefore, in any liquefaction evaluation, it is necessary to include additional explorations to provide samples. This is not an insurmountable drawback, as good practice is generally to employ multiple techniques in a liquefaction evaluation anyway. As mentioned previously, the CPT is generally not appropriate in gravelly soils.

Data Collection and Reduction

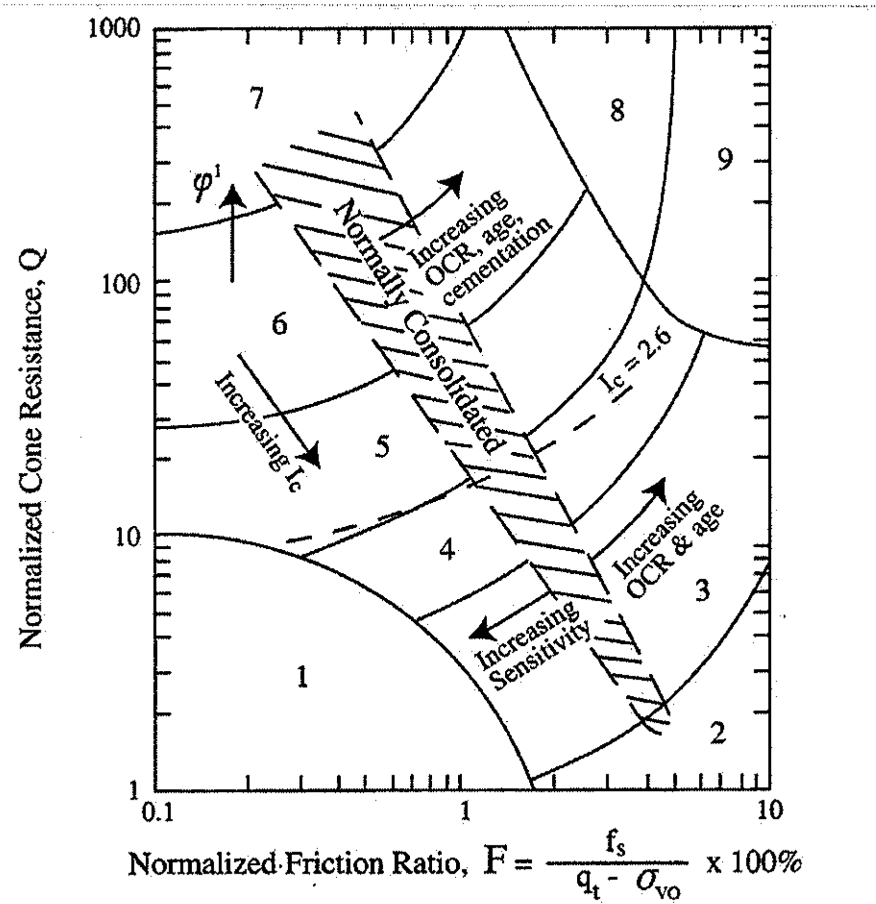
Data collection for the CPT is almost completely computerized and thus relatively simple. Once the data are recorded, they can be processed to predict liquefaction potential using computer spreadsheets with little effort beyond "pasting" in the raw data.

There are generally only two correction factors applied to raw CPT data, although a third is sometimes needed to account for the effects of thin layering. The first adjustment, common to all *in situ* liquefaction procedures, is to normalize the tip resistance for effective overburden pressure, analogous to C_N in the SPT procedure. Based on recent work by R. Seed *et al.* (2003), this overburden adjustment factor depends on soil type. The second adjustment is for the effect of fines content on liquefaction potential (For a given tip resistance, a soil with more fines or a higher friction ratio is generally more resistant to liquefaction). The fines adjustment is the aspect of the CPT that is most debated. In fact, the 1996 NCEER workshop participants could not arrive at a consensus for standardized CPT evaluation procedure, largely because of this issue. Since there is no sample of the material, the CPT procedure uses an empirical correlation with tip resistance and sleeve friction to classify the soil being tested. Figure 10 shows a widely accepted system. Although the CPT is sometimes used to estimate an

Figure 8.—Reclamation's CPT truck.

Figure 9.—Hydraulic CPT pushing mechanism.

"apparent" fines content, none of the more recent methods (Robertson, 2004; Moss and R. Seed, 2004) uses the fines content as an explicit part of the procedure. Instead, they use the friction ratio as a proxy to represent the effect of the fines. (Some earlier procedures *did* use a measured or estimated fines content. Such procedures have mostly been superseded in practice.)



1. Sensitive, fine grained
2. Organic soils - peats
3. Clays - silty clay to clay
4. Silt mixtures - clayey silt to silty clay
5. Sand mixtures - silty sand to sandy silt
6. Sands - clean sand to silty sand
7. Gravelly sand to dense sand
8. Very stiff sand to clayey sand*
9. Very stiff, fine grained*

**Heavily overconsilidated or cemented*

Figure 10.—CPT classification of soil type from Robertson (1990).

Because of the uncertainty in the determination of material properties from CPT data, it is generally prudent to perform some companion SPT tests or borings at selected locations to collect samples for verification or comparison of material classification. Companion SPT data permit confirmation of the indications of the CPT data.

A final correction factor is sometimes needed when thin layers are present within a deposit. The measured tip resistance in a thin layer of a given material can be substantially different from what would be measured if the same material was present in a thicker layer. This is because cone tip resistance is influenced by the condition of the soil several cone diameters below the tip. Thin beds of sand sandwiched between softer silts or clays may give tip resistances indicative of looser material, and they can be misclassified by the CPT procedure. A correction factor can be applied, as described in Youd and Idriss (1997) and elsewhere.

For specifics on all of the CPT correction factors and their use, the reader is referred to the NCEER publication (Youd and Idriss, 1997), the Summary Report of NCEER (Youd *et al.*, 2001), or Reclamation's design standard on *Seismic Design and Analysis* (Gillette, 2001).

Evaluation of Liquefaction Potential

The CPT procedure has evolved significantly and is now viewed as a very sound and reliable method for liquefaction evaluation in appropriate (nongravel) materials. This is due in large part to a large, and growing, database of CPT penetration resistance values in areas that have experienced liquefaction (or no liquefaction) under earthquake loading (reportedly more than 600 case histories). As with the SPT procedure, the NCEER working group looked at the triggering methodology and documented the state of practice (although full consensus was not reached on the preferred method). One relationship between liquefaction potential and adjusted CPT tip resistance published by NCEER (Youd and Idriss, 1997) is based on work by Robertson and others, and as of 2005, it was the most widely used procedure. It is shown in figure 11. The "correction" or adjustment is described in the NCEER publication.

More recent work at Berkeley (R. Seed *et al.*, 2003; Moss and R. Seed, 2004), and University of Alberta (Robertson, 2004), should also be considered for use in liquefaction evaluations. In particular, the correlation reported by R. Seed *et al.* may become the accepted standard in the future, particularly in risk analyses, so it should be considered in addition to the Robertson approach. Key differences between the two methods include:

- The work by R. Seed *et al.* takes advantage of a much larger number of earthquake field case histories, including events postdating the work by Robertson and others. The larger data set permitted them to determine the various functions and adjustments by multivariate regression, rather than estimating them individually, providing greater assurance that they are consistent with each other.

- R. Seed *et al.* reevaluated the adjustment factor for the effect of overburden stress on tip resistance, and their findings are in good agreement with recent work by Boulanger and others for sand and silty soils. Since dam evaluations often involve assessment of liquefaction at large depths, this factor is of particular importance.

- As with the SPT-based correlation of R. Seed *et al.* (2003), the new CPT-based correlation of R. Seed *et al.* employs CSR values that are unbiased with respect to CSRs calculated directly from site response analysis. This is

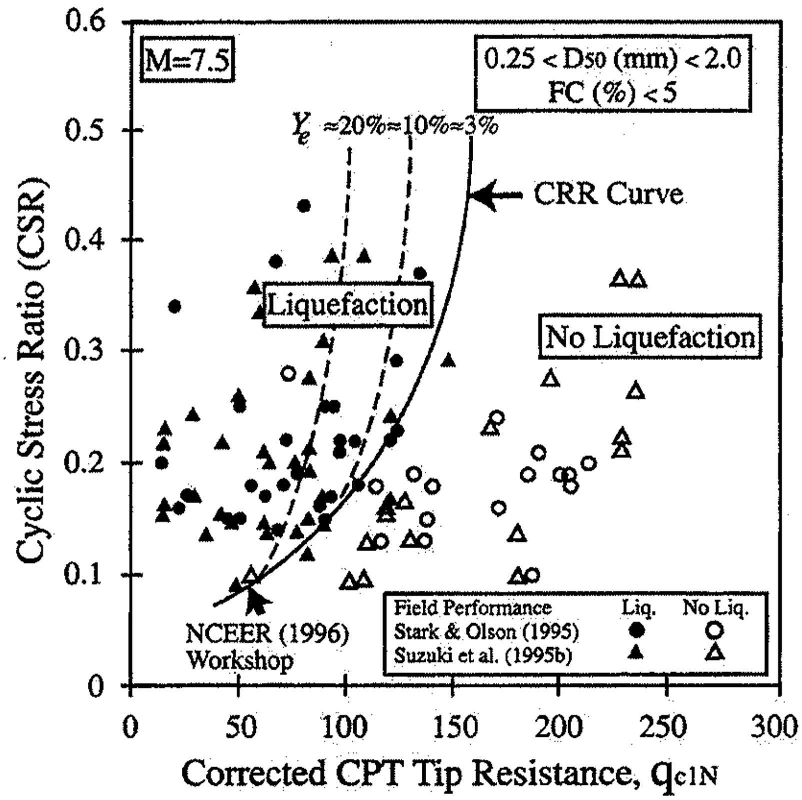


Figure 11.—CPT-based liquefaction triggering relationship from NCEER (1997).

another potentially significant advantage for liquefaction evaluations at dams, where response analyses are often performed.

- The clean sand triggering correlation of R. Seed *et al.* is in fairly good general agreement with the clean sand line of NCEER (Robertson and others). It is the correction for fines that differs significantly between the two correlations. There was discomfort with the fines correction proposed for NCEER (discussed later under *SPT and CPT Applications and Cautions* on p. 50), with the primary concern being that the adjustment for fines was too large, and thus unconservative. The R. Seed *et al.*correlation shows less benefit from fines and is therefore more conservative in silty sands and silts. In this regard, it more closely matches the R. Seed *et al.* SPT correlation in soils with high fines contents. For both of those, the fines adjustment was determined by multivariate regression on a large data set.

Becker Penetration Test

The Becker-hammer penetration test (BPT) is a dynamic penetration test that employs a truck-mounted diesel pile hammer to drive a large-diameter (generally 6.7-inch outside diameter), double-wall threaded casing. The casing is driven into the ground, and the blow counts are recorded for each foot (0.3 m) of penetration. After each 8- to 10-foot length of casing is driven into the ground, another length is threaded onto the end of the previous casing, and driving is resumed. The larger tip diameter, in comparison with the SPT sampler, allows meaningful penetration testing in much coarser deposits, although even the BPT can be adversely affected by grain-size effects from large gravel and cobbles. Unlike the SPT, there are no gaps between test intervals caused by seating intervals and clean-out between tests; Becker testing is essentially continuous. For penetration testing, the casing is driven with a plugged bit, so no samples are recovered. For sampling, the casing can be driven with an open bit with compressed air to lift the cuttings to a cyclone on the surface (This sample is completely disintegrated, and layers of soil may be mixed, so the stratigraphy is very "broad-brush").

The double-wall Becker drive casing comes in 8- or 10-foot (2.4- or 3.0-m) lengths with threaded ends and is available in three sizes: 5.5-inch, 6.7-inch, and 8.7-inch (140-, 170-, and 220-mm) outside diameter; the industry standard for liquefaction investigations is the 6.7-inch casing, and that is the only size for which correlations have been developed.

No direct correlations between BPT blow counts and liquefaction potential have been developed, due primarily to the lack of a sufficient database. Instead, BPT blow counts are generally used to estimate the equivalent SPT blow count that would have been measured at the same location if there were no interference from oversize particles. Harder and H. Seed (1996; H. Seed *et al.*, 1989), and Sy and Campanella (1994; Sy, 1997) have developed two different procedures for estimating equivalent SPT blow counts.

The Becker hammer drill rig (fig. 12) was developed in 1958 in Alberta, Canada, and reportedly was initially used for mineral exploration in gravel. In North America, the drill is now primarily used in geotechnical investigations for drilling, sampling, and penetration testing in deposits containing gravels and cobbles. The penetration testing is commonly used to evaluate soil density and pile drivability. Due to the geology and soil conditions in the western United States (mountainous, glaciated, semi-arid, etc.), coarse-grained and gravelly soils are frequently encountered at Reclamation projects, and use of the BPT is often appropriate.

There are two Becker drill rig models: the older HAV-180 with a telescoping mast, and the newer AP-1000 which has a fixed mast and a more elaborate system connecting the hammer and mast. Both models use the same International Construction Equipment (ICE) Model 180 diesel pile hammer. The energy output of the hammer varies with driving resistance and combustion conditions, such as throttle setting, altitude, and temperature. The throttle allows the operator to control the amount of fuel injected into the combustion chamber. A blower can be used to clear exhaust gases from the combustion chamber between strokes. With the blower switched on, a higher throttle setting can be used, yielding more

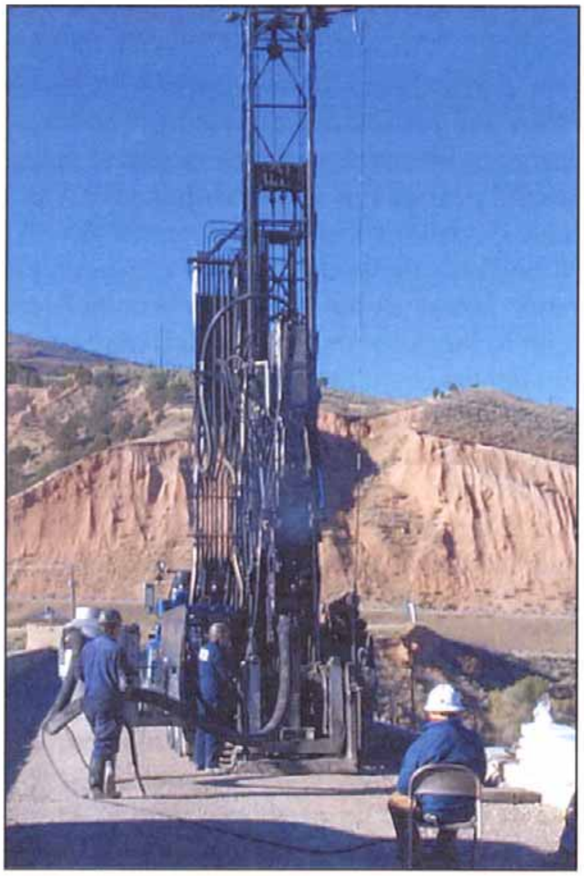


Figure 12.—Model AP-1000 Becker drill rig.

energy per blow. Even if constant combustion conditions could be maintained, the hammer energy output would still depend on soil resistance, which must be accounted for in data reduction. Each of the two methods for estimating equivalent SPT blow counts includes a method for doing so. The ICE 180 hammer used on the Becker drill is not a typical diesel hammer on which the ram rises freely after each impact, to be returned to the anvil by gravity alone for the next impact. Instead, air is compressed in the bounce chamber by the rising ram and acts as a spring to return the ram more quickly, resulting in more blows per minute than with the more typical open-end hammers. In the Harder and Seed method (1996; H. Seed *et al.*, 1989), measuring the pressure in the bounce chamber is used to provide an approximate indication of the energy produced by the hammer, which is greater than the energy that is actually transmitted to the BPT rods. In the Sy and Campanella procedure (1994; Sy, 1997), the energy transmitted to the rods is actually measured by means of an electronic pile-driving analyzer (PDA). These are discussed below; greater detail can be found in Harder and H. Seed (1996), H. Seed *et al.* (1989), and Sy (1997).

Although penetration testing is the primary use of the Becker rig in liquefaction investigations, it can also be used to obtain soil samples, usually in "companion" holes alongside soundings to measure penetration resistance. This is accomplished by driving the double-walled casing with an open bit and air circulation to return cuttings. Air is forced down to the bit through the outside annulus during driving, and soil particles entering the bit are carried up the inner casing by return air flow and collected in a cyclone. The diameter of the inner pipe is 4.3 inches. Large broken pieces of rock or gravel indicate material that is too coarse for meaningful penetration testing with the BPT, in which case the BPT correlations would overestimate the equivalent SPT N. This points out a limitation of the BPT and also shows the value of conducting Becker sampling holes next to penetration holes. Becker samples are completely disturbed and therefore quite difficult to log accurately. Materials may be segregated, or those from slightly different depths could be mixed. The water content will almost certainly be incorrect because of drying by the compressed air used to lift the sample up the casing to the cyclone. For that same reason, one should not attempt to use Becker samples to estimate the depth to the water table. However, due to the ease of Becker sampling, frequent use of sample holes adjacent to BPT holes somewhat helps overcome the disadvantage of the BPT not providing a sample of the tested material.

Early attempts to use open-bit driving to estimate soil density were abandoned when they were found to give inconsistent results. It was concluded that the bit was alternately being plugged by bridging gravel particles, then unplugged. The air circulation to lift the samples apparently does not keep the bit cleaned out consistently, as would be required for consistent driving resistance. Therefore, the standard for penetration testing is use of a plugged bit. Blow counts measured in open-bit sampling holes are generally much lower, and therefore they should provide a lower bound on the closed-bit blow count, which is used in the correlations.

An obvious limitation of the BPT is the need to convert the blow counts to equivalent SPT blow counts in order to evaluate liquefaction potential. Not being able to use BPT blow counts directly creates an additional level of uncertainty in the liquefaction evaluation, in addition to existing uncertainties already inherent in *in situ* testing methods. This is not a fatal flaw, and BPT is still viewed as a useful tool, but engineers need to apply appropriate caution in using the results.

Standardization of Equipment

As of 2005, the BPT is not entirely standardized, but the correlations with SPT N-values developed to date require the use of specific equipment selected from the range of casings and bits available. The NCEER report (Youd and Idriss, 1997) recommended striving for consistency in three areas to help obtain consistent and reliable data: (1) standardization of driving and penetration equipment, (2) determination and interpretation of diesel hammer driving energy, and (3) evaluation/interpretation of the effect of casing friction on penetration resistance. Each of these key factors is discussed below.

The recommended equipment is that used in the Harder and Seed correlation: a plugged, crowd-out bit, 6.7-inch (168-mm) outside diameter casing, driven by an

AP-1000 drill rig without the use of prebored holes or drilling fluids. This is the *de facto* standard for the industry, and it is greatly preferred for Reclamation projects. A distant second choice would be to use the same casing and bit with the older HAV-180 drill rig. If this rig is used, in the Harder-Seed procedure, the BPT blow counts are multiplied by a correction factor of 1.5 to convert to equivalent AP-1000 blow counts. The precise value of that factor is supported by only a few comparisons, and it has, in fact, been shown not to be consistent among all rigs. In the Sy-Campanella method, the rod energy is measured, and the blow counts are adjusted in proportion to the energy. At least in theory, this makes the choice of rig model unimportant, but the AP-1000 should be used on Reclamation projects whenever possible. This will maintain greater consistency and allow both methods to be used; the Harder-Seed method allows the data to be processed much more quickly because it does not require detailed analysis, and it can provide a check on the general reasonableness of the Sy-Campanella results. Even if the preferred equipment is used, and especially if different equipment is used, it is prudent to perform some adjacent SPT testing to verify to the extent possible that the correlation between BPT and SPT blow counts holds for the site under study. This is difficult in coarse soils, but short-interval SPT testing (discussed in *Special Notes on Gravelly Soils*, p. 59) may yield some confirmation. The SPT drilling would also provide a better representation of the stratigraphy than can be obtained from open-bit Becker drilling.

Harder-Seed Method for Estimating Equivalent SPT Blow Counts

In 1986, L.F. Harder, Jr. and H.B. Seed, working at the University of California at Berkeley developed the first widely used procedure for estimating equivalent standard penetration test blow counts from BPT blow counts (Harder and H. Seed, 1996). Recognizing the great sensitivity of the blow count to driving conditions, they developed a rather ingenious method of using the measured bounce-chamber pressure to adjust the blow count to a hypothetical constant combustion condition, meaning consistent throttle setting and performance of the diesel hammer. That adjusted blow count, referred to as N_{bc}, is used in a simple correlation to predict the equivalent SPT blow count N_{60}.

Adjustment for Hammer Energy

As mentioned above, hammer performance and the energy transmitted to the rods are quite sensitive to a number of influences. In the Harder-Seed method, energy is monitored indirectly by measuring the bounce chamber pressure. Harder and Seed found that constant combustion conditions result in a unique relationship between bounce chamber pressure and Becker blow count, plotted in figure 13. To account for deviation from the constant combustion condition, they developed a family of curves for correcting measured blow counts to their equivalents on the constant combustion condition curve. These curves are shown in figure 13. To use them, locate the intersection point of the raw BPT blow count and the bounce chamber pressure. From that point, follow a path parallel to the nearest blow count correction curve down to line A-A (constant combustion condition rating curve). The blow count value at the intersection is referred to the corrected Becker blow count, or N_{BC}. Although the curves are based on sea level

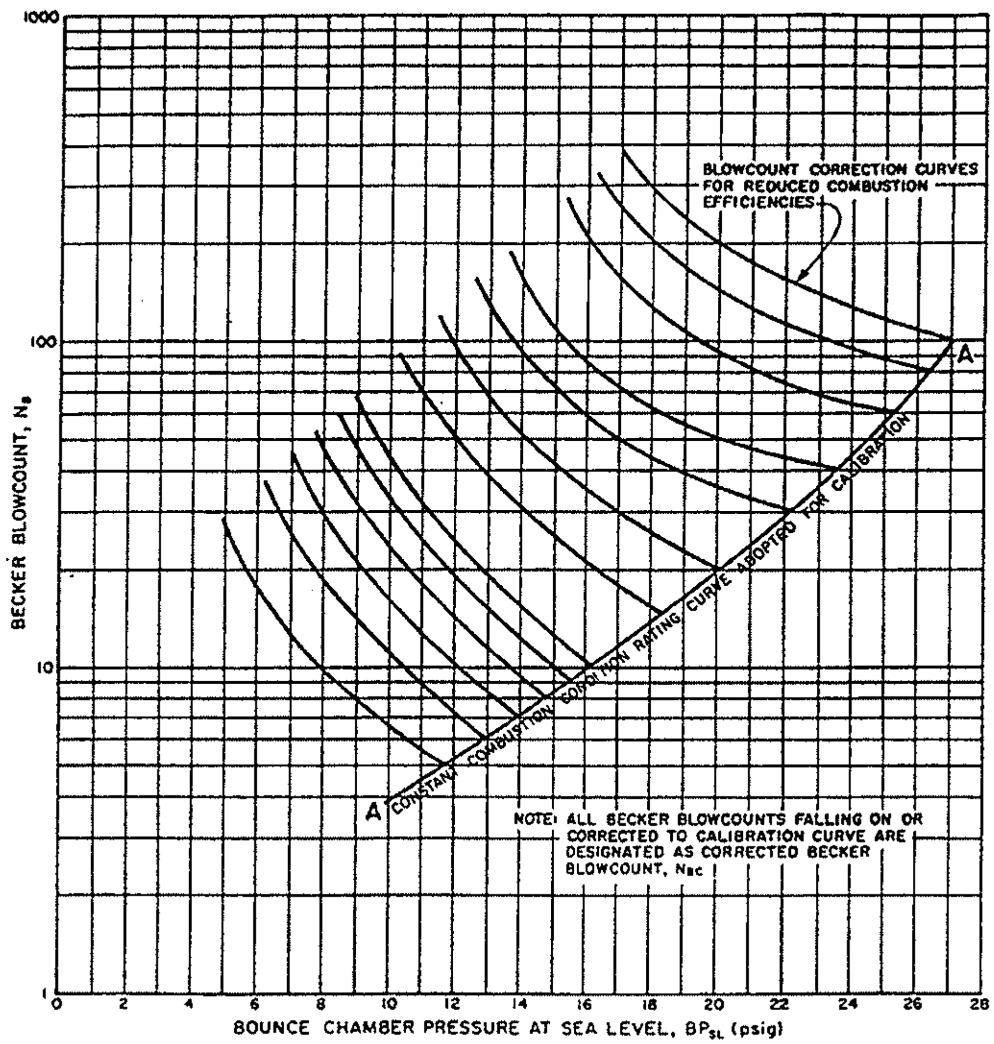


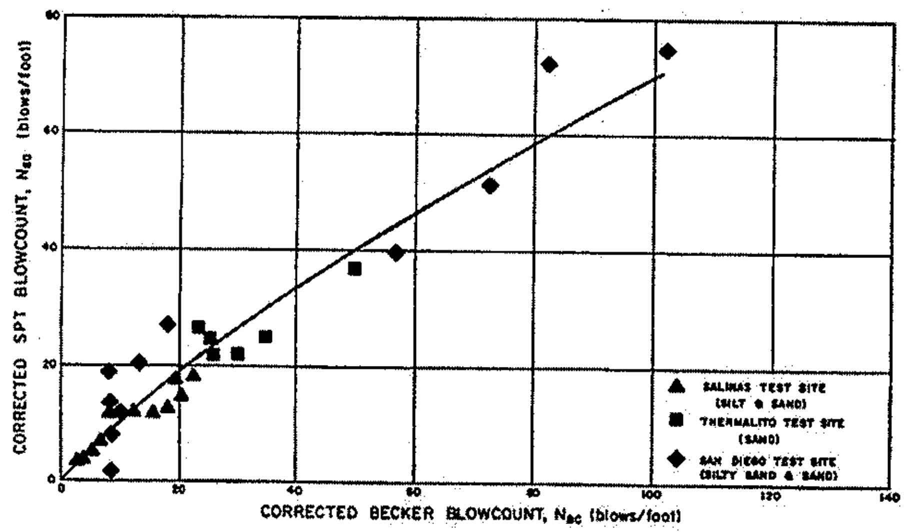
Figure 13.—Harder and Seed correction curves for BPT blow counts.

atmospheric conditions, bounce chamber pressures at different site elevations can be converted to approximate sea level conditions. For details, see Harder and H. Seed (1996).

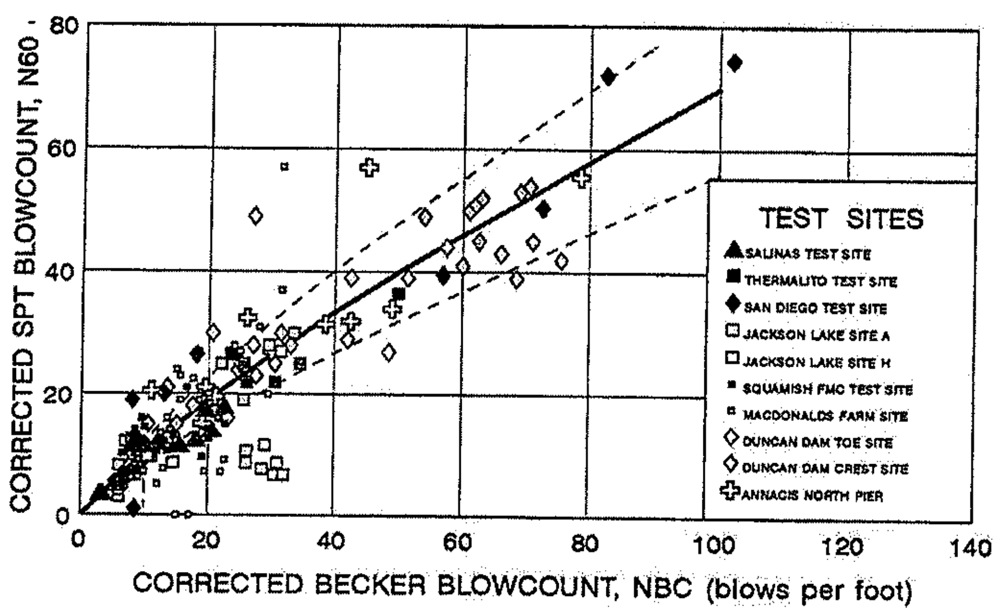
Prediction of Equivalent SPT Blow Count

Once it has been determined using figure 13, the adjusted Becker blow count N_{BC} is used with the simple correlation shown in figure 14a below to estimate the equivalent SPT blow count N_{60}, which is the SPT blow count adjusted for driving energy, but not for overburden stress.

Note that figure 14a shows a fairly small number of data points. It was published as part of the original research project. Figure 14b shows additional correlation data obtained subsequently from other sites. Although there are considerably more scatter in the data, it can be seen that the original correlation curve is still a good approximation of the "best fit" curve.



a. Original Harder and Seed (1986) Correlation



b. Harder and Seed (1986) Correlation Supplemented with Data from Additional Test Sites

Figure 14.—Harder and Seed SPT-BPT blow count correlation.

Impact of Casing Friction

Driving the BPT casing mobilizes resistance both at the advancing tip and along the full embedded casing length. The Harder and Seed correlation implicitly includes casing friction, but does not account for variation from the typical conditions that existed at the original correlation sites. With alternate driving methods, such as predrilled or cased holes or mud injection, use of the Harder and Seed correlation is not appropriate and can result in overly low estimates of the equivalent SPT blow counts. The reverse can occur at large depths or with certain types of overburden above the layer being tested, in which case the side friction can be very large. Subsequent work by Alex Sy and Richard Campanella at the University of British Columbia produced an alternative method that explicitly measures and adjusts for shaft friction. This is discussed in the following section.

Sy-Campanella Method for Estimating Equivalent SPT Blow Counts

The second procedure, developed by Sy and Campanella, includes use of a pile-driving analyzer to record time-series measurements of rod force and acceleration during each blow (Sy and Campanella, 1994; Sy, 1997). These data are used to calculate the energy transferred to the drill string, and to separate, analytically, the components of driving resistance from the tip of the rods, and from the "shaft resistance," R_s, resulting from friction and adhesion along the rods. The measured raw blow count is adjusted to the reference energy of 30 percent of the hammer's rated energy by simple proportion with the measured energy; this adjusted blow count is termed N_{b30}. Subsequently, wave-equation analysis of the recorded force and acceleration measurements and/or measurements of the force required to withdraw the rods from the ground are used to separate the driving resistance into its two components. The equivalent SPT blow count is then predicted from a correlation with blow count and shaft resistance, shown on figure 15. Each curve in the figure corresponds to a different value of shaft resistance. Details of this procedure can be found in Gillette (2001), Sy and Campanella (1994), and Sy (1997)

The procedure can be summarized as follows:

- The PDA is used to determine ENTHRU, the energy transferred to the top of the BPT drill string (casing), expressed as a percentage of the rated energy of the ICE 180 diesel pile hammer.

- The measured field BPT blow count N_b is then adjusted to a reference energy of 30 percent of the rated energy by simple proportion, using the equation $N_{b30} = N_b * (\text{ENTHRU})/30$

- Casing friction, termed R_s, is estimated using the wave equation analysis program CAPWAP, which is routinely used in the pile-driving industry, or by pullback tests that directly measure the force required to extract the casing from the ground.

- The values of BPT N_{b30} and R_s, are used with figure 15 to estimate the equivalent SPT N_{60}. Each curve on the figure corresponds to a given value of R_s.

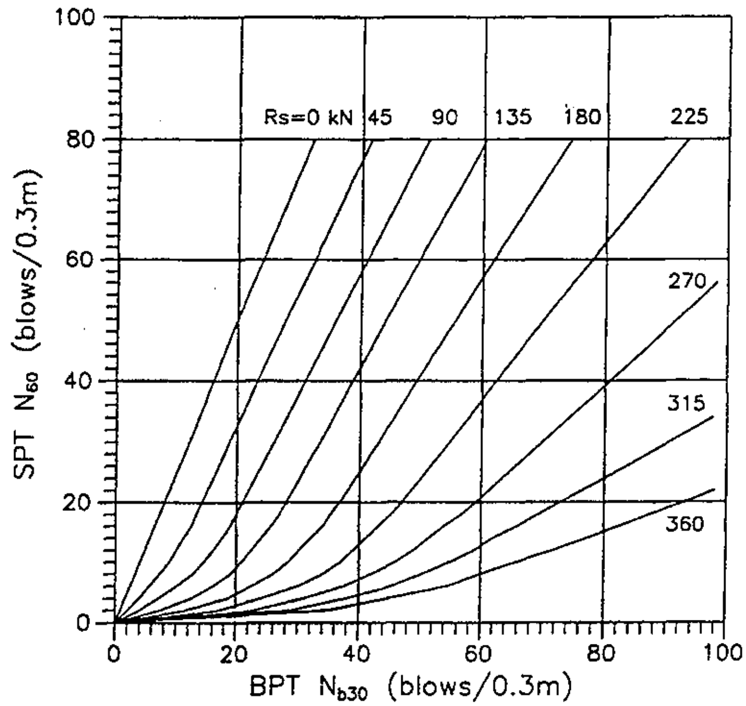


Figure 15.—Sy and Campanella correlation for SPT vs. BPT.

The Sy-Campanella procedure was developed to account explicitly for the effect of variation in rod friction, and to adjust for differences in transferred energy with actual measurements, rather than energy estimated from bounce pressure. The Harder-Seed method does include the friction component, but only implicitly, and it does not account for variation due to different materials above the tip. It often produces results in close agreement with SPTs and the Sy-Campanella method. However, at sites where the shaft resistance is atypically large or small, the results of the two methods can differ substantially. In one recent example from Deer Creek Dam, the Sy-Campanella procedure indicated much higher equivalent SPT blow counts. The difference was due to the atypically low measured shaft resistance that resulted from fairly shallow, moderately dense, gravelly overburden.

Because of the need to measure energy and determine the shaft resistance, this approach is more complicated and costly to implement, which is part of the reason that it is not as widely used in practice as the Harder and Seed method. Other than the greater time and cost for its use, another concern is the sensitivity of the N_{60} estimate to the value of the shaft resistance. The wave-equation analysis (using the computer program CAPWAP), is not unique, in that the equations can be solved with different combinations of soil parameters. The estimate of shaft resistance is sensitive to variation in a number of estimated parameters, including an assumed casing/soil viscous damping constant. For this reason, it is becoming common to interrupt driving periodically to measure the force required to pull the rods back out of the ground a few feet, providing a direct measurement of the shaft resistance. This provides an approximation of the casing friction occurring during driving. Reclamation Becker programs should include both pullback measurements and PDA measurements to provide independent estimates of shaft resistance. (Bounce chamber pressures should also be recorded so the Harder-Seed procedure can be used as well.)

As an alternative method for determining shaft resistance, some practitioners prefer to measure casing friction by redriving using the PDA. One would drive the BPT for 10 feet, conducting measurements as is normally done, but then stop and withdraw the casing 5 feet, then redrive it 5 feet. During redrive, practically all of the resistance occurs along the rods, not at the tip, which should permit CAPWAP to make a more accurate estimate of the friction. Since the first foot of redrive includes some disturbance, and the bottom 2 or 3 feet begin to encounter appreciable tip and casing resistance, only the second foot of redrive is used to measure casing friction. Proponents of this approach argue that there is a viscous component to driving and pull-back friction or adhesion, and therefore, the shaft resistance is best measured by driving the casing with the same (or similar) rates of strain and displacement as in the actual BPT testing. Without the viscous component, pullback tests could underestimate the shaft resistance, which would cause overestimation of the equivalent SPT blow count. Later sections of this report present some case histories where pullback tests and redriving after pullback were used to help account for the effects of friction.

Measurement of Shear-Wave Velocity

The stiffness, and therefore the shear-wave velocity, of soils are governed in large part by the density of the soil. The shear-wave velocity can be used to predict liquefaction potential, which is also a function of density. Measurements of shear-wave velocity are often needed for site-response analysis, so they can serve both purposes. In material with significant amounts of coarse gravel or cobbles "floating" in a matrix of finer material, it may not be possible to obtain valid SPT or BPT data. The SWV can often be used as an indication of liquefaction potential in such materials.

Shear-wave velocity is most commonly measured using crosshole surveys, surface-to-borehole (downhole) surveys, suspension borehole logging, and spectral analysis of surface waves (SASW). The appropriate applications, advantages, and limitations of each of these four methods are discussed below and summarized in table 1. The recommendations presented here are based on experience with the various techniques at Reclamation dam sites and others. The choice of technique is influenced by dam site topography, depth of investigation, and the spatial resolution required. (Other practitioners may have slightly different approaches.)

Table 1.—Comparison of methods used to measure shear-wave velocities for soil liquefaction evaluation

Method	Relative vertical resolution	Applicable depth range	Drilling require-ments	Comments
Crosshole survey	high; constant with depth	Unlimited	Preferred: 3 cased boreholes (possible with 2)	Best vertical resolution. Grout travel in very coarse materials can cause overestimation.
Downhole survey	moderate; decreases with depth	< ~100 feet at most Reclamation dam sites	1 cased borehole	Data quality is very site specific.
Suspension logger	high; constant with depth	Unlimited	1 open borehole	Data at dam sites usually acquired as drill casing is removed. Must be performed by contractor (Reclamation doesn't have equipment).
SASW	low; decreases with depth	< ~50 feet at most Reclamation dam sites	none	Need topographically flat area for survey. Can resolve a velocity layer whose thickness is > ~1/5 of its depth.

Crosshole Shear-Wave Survey

The crosshole shear-wave survey is the technique most commonly used by the Bureau of Reclamation for obtaining shear-wave velocities for soil liquefaction evaluation. The crosshole method consists of propagating shear-wave energy horizontally from a source borehole to one or two receiver boreholes (fig. 16). The S-wave velocity is computed from the arrival times of the shear waves. Measurements are typically taken at depth intervals of 2 feet. Figure 17 shows a typical profile of velocity measurements. The most important advantages of the crosshole method are its relatively high vertical resolution (i.e., its ability to provide reasonably accurate velocities in low-velocity layers as thin as 2 feet under the right conditions) and the fact that the resolution is not diminished with depth. The method may also be applied in any location that a drill rig can access, including on the slope or crest of a dam. The disadvantages include the necessity of drilling two or three boreholes to the desired depth of investigation. Also, the casing must be grouted in place, which can cause problems in coarse-grained gravelly and cobbly materials if the grout invades open-work voids some distance from the borehole. This can bias the measured velocities to be too high because the cured grout is much stiffer than the soil. Reclamation experience indicates that crosshole shear-wave velocity measurements can also be influenced by difficult-to-quantify drilling disturbance. Disturbance would be worse with high-energy air-rotary or percussion drilling like the Odex system, than with mud-rotary drilling or hollow-stem augers. Despite these complications, the crosshole

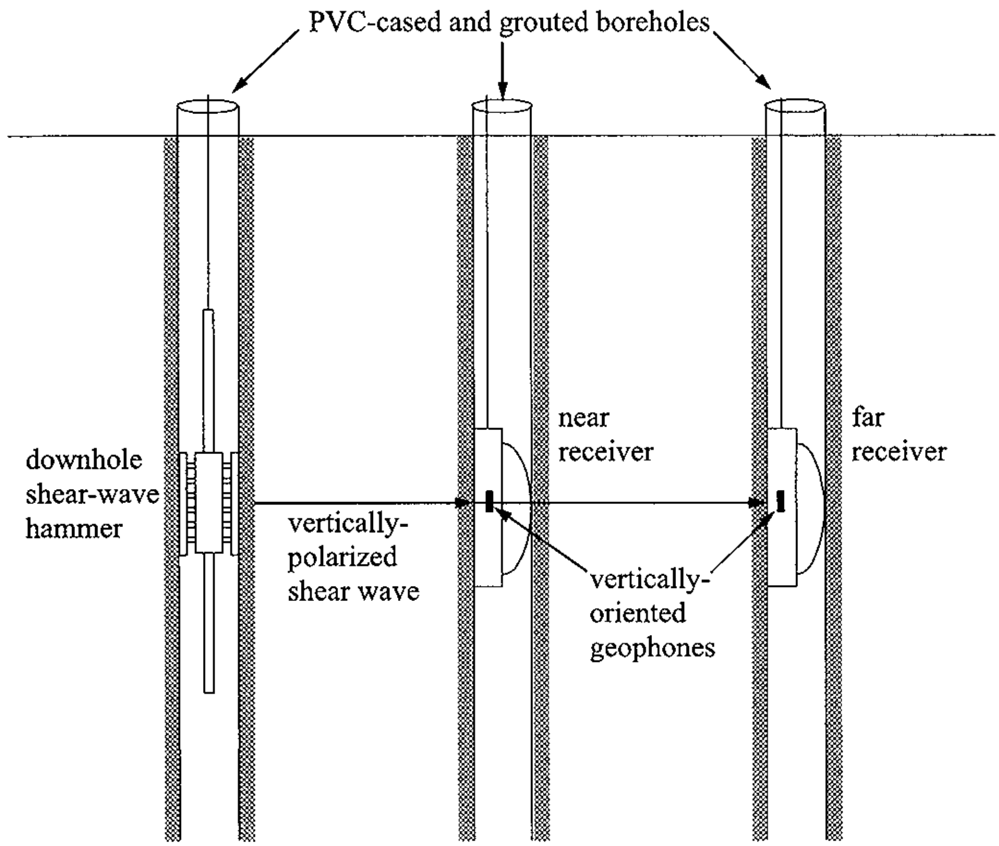


Figure 16.—Crosshole shear-wave-velocity method.

shear-wave survey remains the preferred method of investigation in many situations because of its ability to resolve thin low-velocity layers at depth.

Surface-to-Borehole (Downhole) Shear-Wave Survey

Surface-to-borehole, or downhole, shear-wave surveys can be used when relatively shallow depths (less than 100 feet, preferably less than about 50 feet) are involved and lower resolution is acceptable (generally when low-velocity layers thinner than about 5 feet are not anticipated). For a downhole shear-wave test, a wave is generated by a source on the ground surface and recorded by a geophone in a nearby borehole (fig. 18). The testing arrangement can be reversed, using a downhole hammer and a receiver on the surface to perform uphole measurements. Data are acquired for multiple receiver depths. An S-wave velocity-depth profile is computed from the shear-wave arrival times. A typical profile is shown in figure 19. Because the wave must travel from a source on the ground surface to a receiver located within a borehole at depth, data quality is very site specific. Near-surface coarse grained materials, such as riprap, can interfere with the coupling of the surface source to the ground, and can also scatter much of the high frequency energy before it reaches the receiver. Multiple interbedded firm and soft strata (high- and low-velocity layers) can cause much of

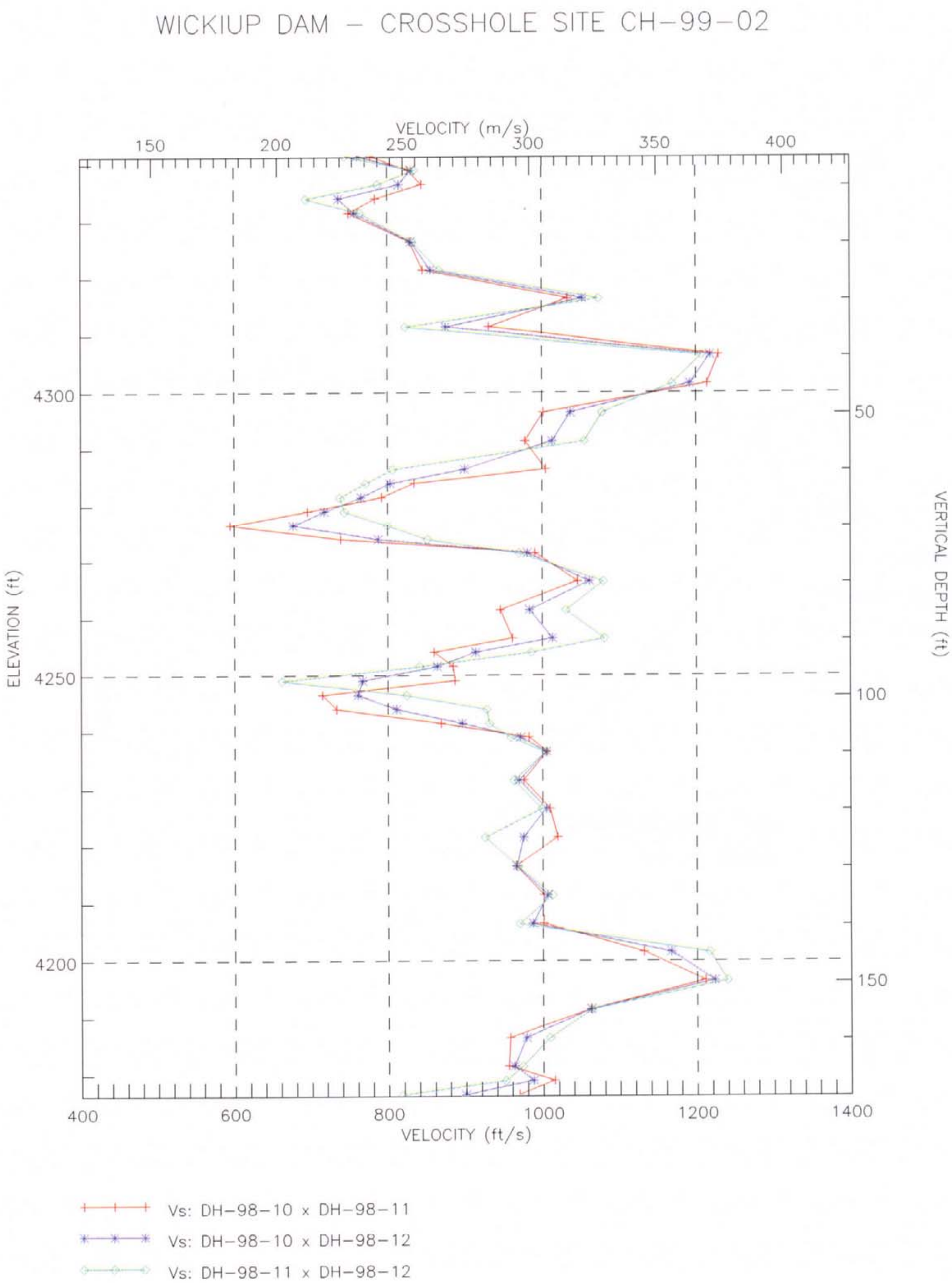


Figure 17.—Shear-wave-velocity profiles from crosshole survey.

the seismic energy to be reflected back toward the source rather than reach the receiver. Even under optimum conditions, the spatial resolution of the method decreases with depth because of scattering and absorption of the seismic energy. For these reasons, the downhole method is best applied where the materials to be investigated are at or near the surface, such as alluvium downstream of the toe of a dam, rather than on the slope or crest of a dam. The advantage of this method

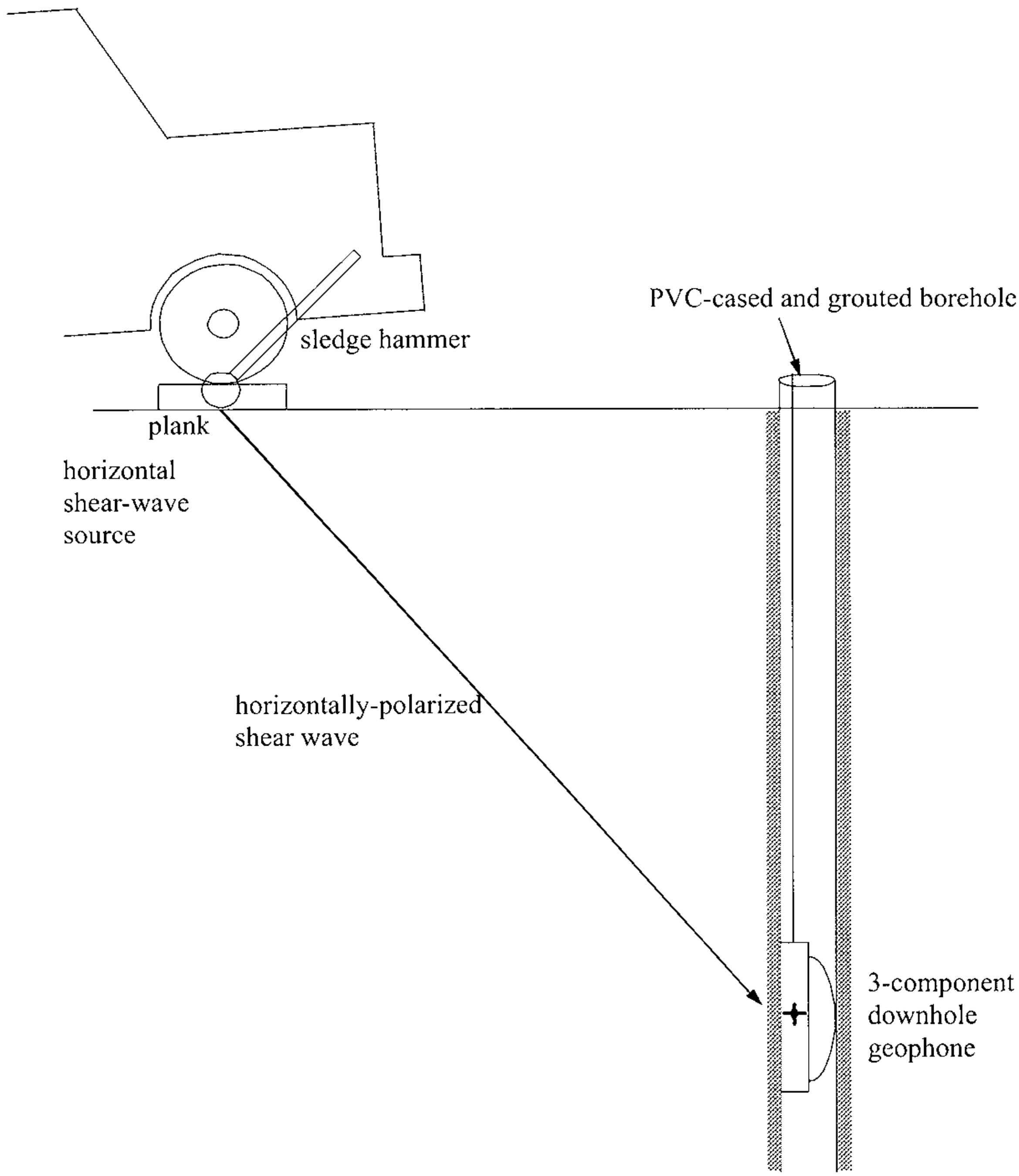


Figure 18.—Downhole shear-wave-velocity method.

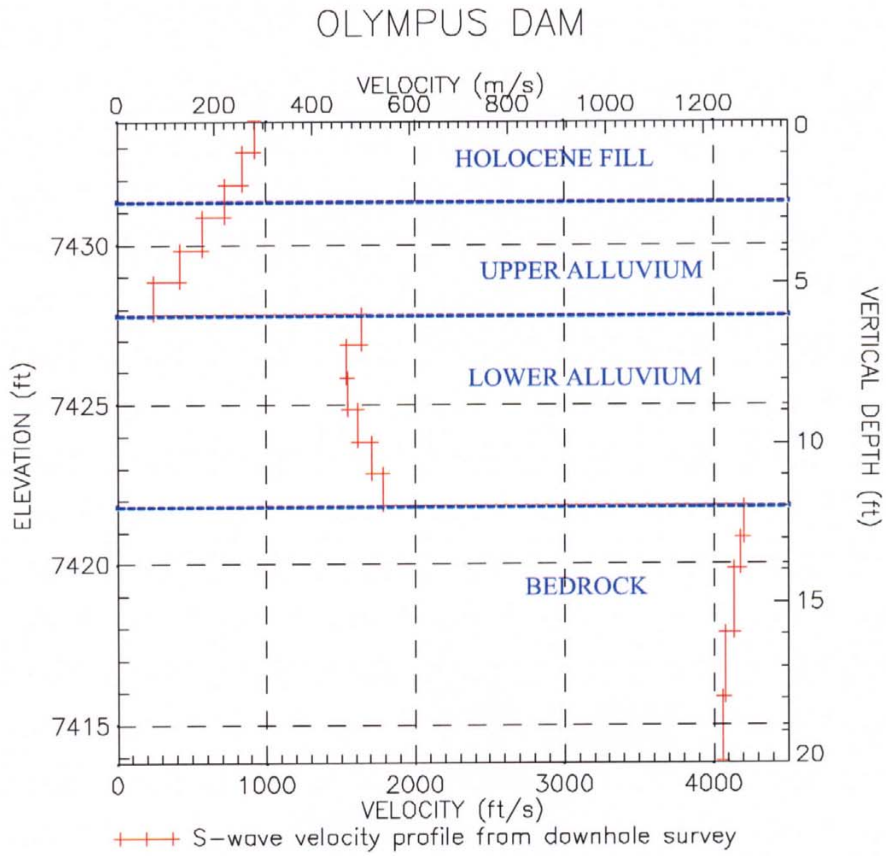


Figure 19.—Shear-wave-velocity profile from downhole survey.

over the crosshole method is that only one borehole is needed, rather than two or three. Downhole tests can be performed as part of a CPT investigation without drilling at all, using a seismic cone penetrometer (SCPT) that carries a receiver within the cone (This is discussed further under *Seismic Cone* on p. 109). The logistics of performing downhole surveys are less complicated than for suspension borehole logging (described below) because all the boreholes can be drilled and cased prior to geophysical testing. (Suspension logging is best performed in an open borehole as the drill casing is being removed.) In general, the downhole method yields a higher-resolution shear-wave velocity profile than does the SASW method.

Suspension Borehole Logger

The suspension borehole logger measures the S-wave velocity profile using a single borehole. The suspension logger contains a seismic source and two receivers (fig. 20). Seismic waveforms are recorded as the tool is moved continuously inside the borehole. Differences in shear-wave arrival times at the two receivers are used to compute the shear-wave velocity at each depth. Under some geologic conditions (most notably in fine-grained materials) reliable S-wave velocities can be obtained using the suspension logger inside a cased borehole.

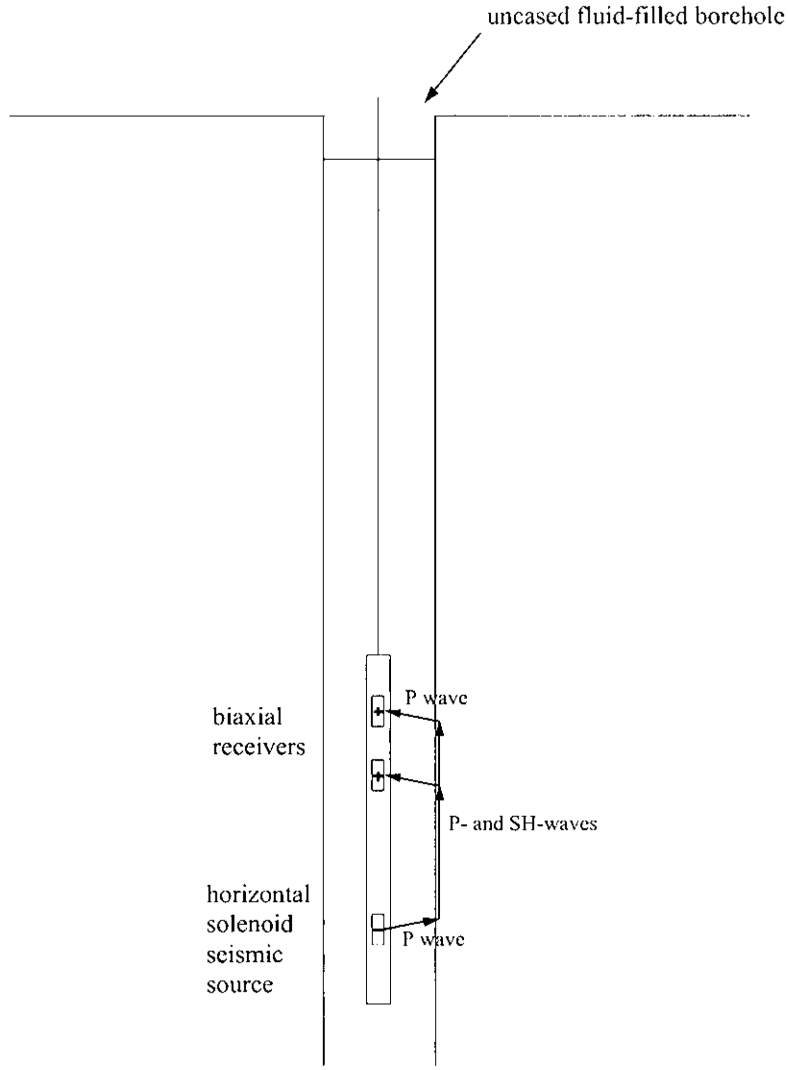


Figure 20.—Suspension logger for measuring shear-wave velocity.

However, experience at Reclamation dam sites (many of which have coarse-grained alluvial or glacial materials) indicates that suspension logger surveys must be run in open boreholes to obtain good results. Since open boreholes in granular soils are subject to collapse, measurements are normally performed in stages as the drill casing is removed from the borehole. An additional complication is that a contractor must be hired, because Reclamation does not have the equipment, nor is it readily available to rent. Although the logistics of running suspension logger surveys are more complex than for other types, this technique can yield a high-resolution S-wave velocity profile without the potential complications of grout travel sometimes associated with crosshole measurements in coarse-grained environments. It yields higher-resolution from a single borehole and can be applied at greater depths than a downhole shear-wave survey. Refer to figure 21 for a typical profile of measured shear-wave velocities.

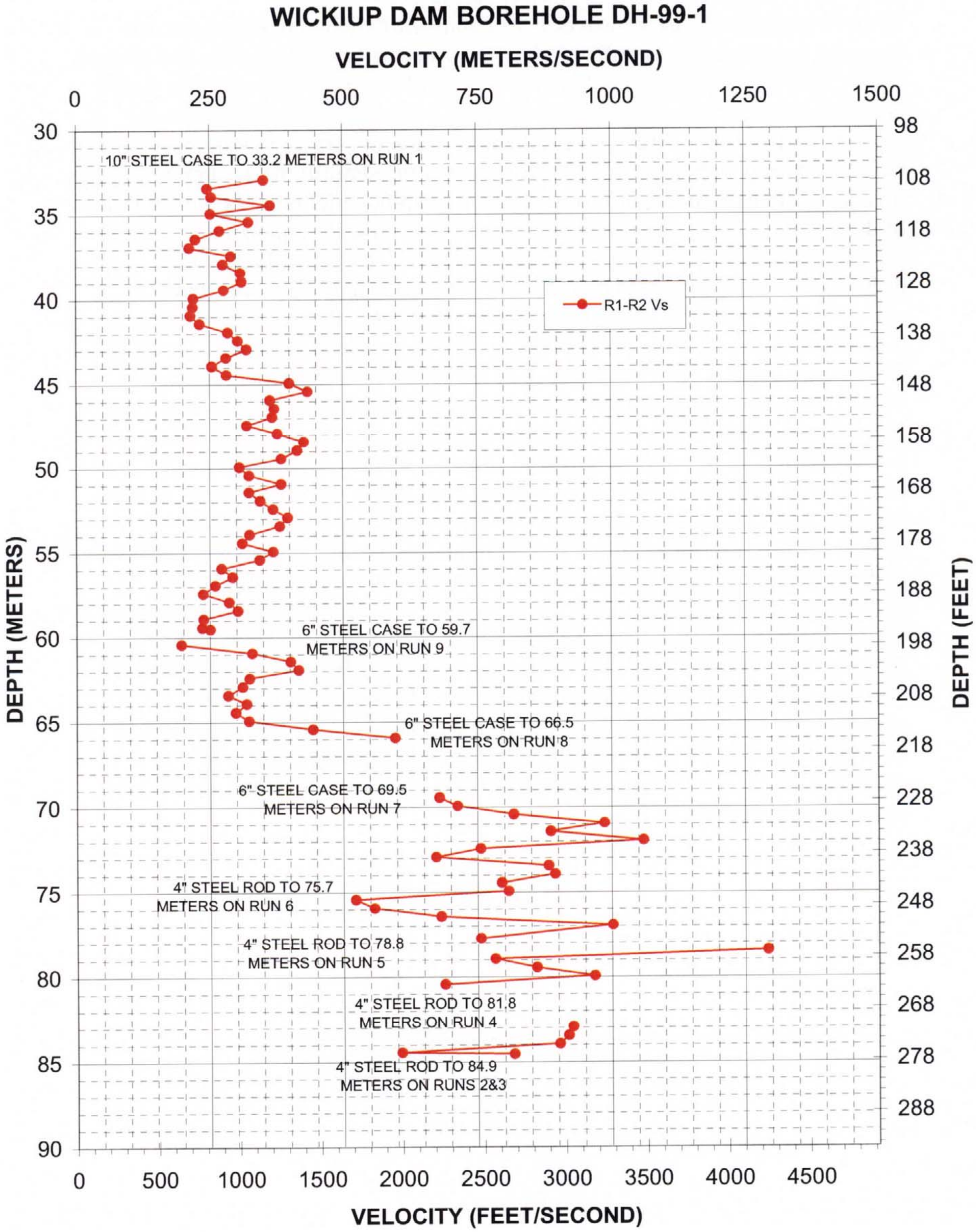


Figure 21.—Shear-wave-velocity profile from suspension logger.

Spectral Analysis of Surface Waves Method

Spectral analysis of surface waves is a geophysical method that uses surface waves to infer a vertical profile of shear-wave velocity (Stokoe, Joh, and Woods, 2004). The one advantage of this method over those described above is that it does not require the drilling of any boreholes. The variation of shear-wave velocity with depth is estimated from an inversion of surface vibrations recorded by an array of geophones (fig. 22). Because of the simplifying assumption in the inversion that the ground surface is flat, topographic relief reduces the reliability of the results. Hence, the testing is best performed on a topographically broad, flat area, away from dam slopes or canyon walls. The vertical resolution decreases with depth. In general, a layer whose thickness is greater than approximately one-fifth of its depth can be resolved with this method. Because of this limitation, the SASW method is not useful at a site where detection of thin liquefiable layers is critical. Because of the relatively low vertical resolution of the SASW method at depth, it is best applied for shallow investigations (less than about 50 feet, depending on the vertical resolution required). Results from SASW are not unique, in that several possible velocity profiles may fit the surface wave data equally well. For this reason, a range of possible velocity profiles is often reported, as in figure 23, rather than a single profile that can be considered "correct." It is also important to note that the SASW method can have difficulty in determining the shear-wave velocity in a softer (lower velocity) layer underlying a stiffer (higher velocity) layer at depth. Although SASW can detect such velocity reversals with depth, an accurate shear-wave velocity in the low velocity layer may not be possible. Because of this difficulty, SASW may be most useful when used in conjunction with another method. For example, crosshole testing could be used to determine an accurate velocity-depth profile at one location, with SASW used to determine (at least qualitatively) how the velocities vary laterally from the crosshole site, that is, whether a low-velocity layer thins or thickens.

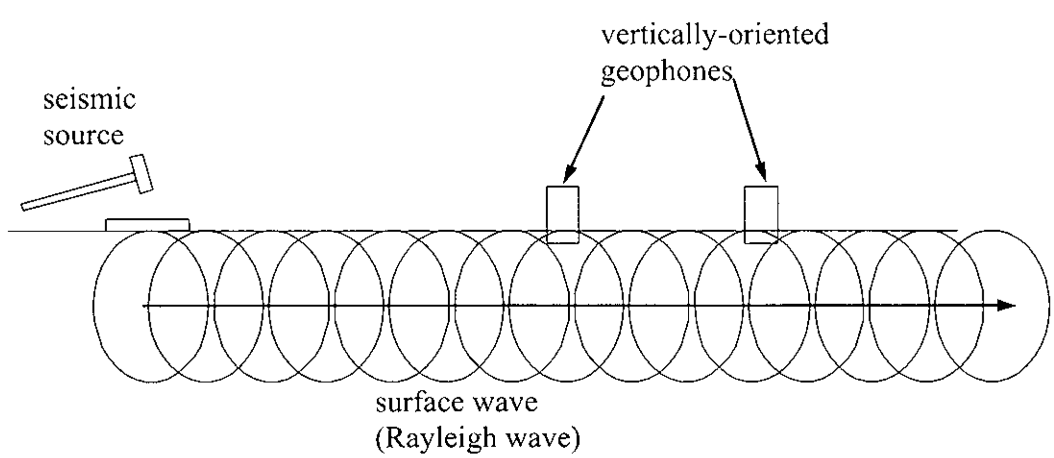
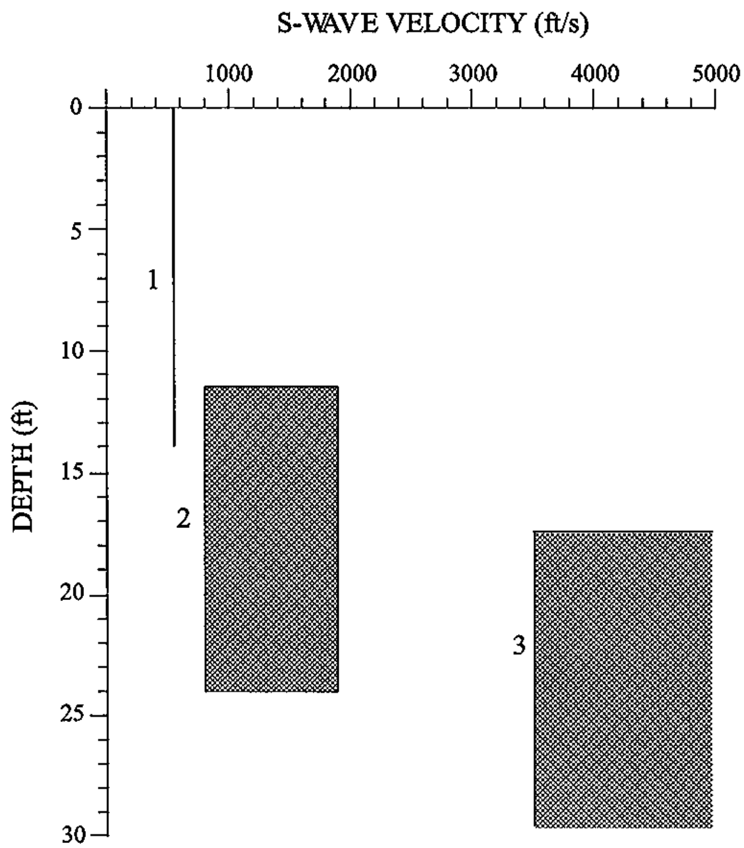


Figure 22.—Data acquisition for the SASW method.



LAYER NO.	INTERPRETED MATERIAL TYPE	S-WAVE VELOCITY (ft/s)	LAYER THICKNESS (ft)
1	alluvium/fill	540 - 555	11.5 - 14.0
2	weathered schist	800 - 1900	6.0 - 10.0
3	competent schist	>3500	unknown

Figure 23.—Shear-wave velocities from an SASW survey.

Data Acquisition and Processing Procedures

Crosshole Shear-Wave Survey

Crosshole shear-wave surveys are usually performed using three boreholes arranged in a straight line, that is, a crosshole triplet. For measuring SWV in soils, the borehole spacing is typically 10 to 12 feet, with larger spacings being used for deeper boreholes. The source is placed in a borehole at one end of the triplet, and receivers are placed in each of the other two boreholes at the same elevation (fig. 16). The instruments are moved up or down together so measurements can be made at depth increments of 2 to 5 feet.

A downhole shear-wave hammer is used to generate the seismic energy. Typically, this consists of a central cylinder that is locked inside the borehole casing with hydraulically powered pads, and a sliding arm with reversible impact directions. It is designed to preferentially generate shear-wave energy in the vertical plane (SV waves). At each recording depth, two records are acquired, one for each source polarization, in other words, one "down hit" and one "up hit." The time of impact (trigger time) is recorded by a high-frequency (2-kHz) geophone located inside the immobile, central portion of the shear-wave hammer. The output from this trigger geophone also signals the seismograph to start recording.

A vertically oriented geophone is used in each receiver borehole to record the S-wave data. Data are simultaneously recorded by a hydrophone in each receiver borehole for the purpose of recording P waves. Because the seismic source is optimal for generating shear-wave energy but not P-wave energy, good P-wave data are often not obtained above the saturated zone. The P-wave velocity profile can be useful for identifying the water table or saturation zone (above which liquefaction is unlikely), which may be important if piezometric data are not available. If accurate P-wave velocities are needed, a separate survey can be run using a source and receivers that are optimal for P-wave (instead of S-wave) measurement.

Seismic waveforms recorded in the near and far receiver boreholes are analyzed to determine the arrival times of the direct, horizontally propagating P and S waves. Where there are stiff layers close to soft layers, it is possible that waves refracted into the stiff layer could "outrun" the direct wave traveling within only the soft material. Arrivals that are believed to be strongly affected by refraction through nearby higher-velocity layers are not used. Precise trigger times are determined from the recorded trigger waveforms and subtracted from the P-wave and S-wave arrival times to obtain travel times. Crosshole distances are computed from borehole directional surveys and relative distances and azimuths between boreholes measured in the field. For each depth, two direct P- and S-wave velocities are then computed using the travel times from the source to the near and far receivers. One P-wave interval velocity and one S-wave interval velocity are also computed for each depth, using the differences in travel times between the near and far receivers. All velocity computations are done assuming straight ray paths.

It is preferred to use three boreholes in crosshole testing, although sometimes only two holes are used. It is better to use three because it enables a check on the timing of the source triggering (time 0 determination). With only two holes, one has to assume that the triggering is always functioning correctly with no way to confirm, and any triggering error results directly into an error in velocity computation. In addition, three boreholes enable better interpretation of the data, including understanding whether the arrivals in the waveforms are direct or refracted.

Typical shear-wave velocity profiles from cross-hole testing are shown on figure 17. These are from a cross-hole survey performed on the crest of Wickiup Dam in Oregon. Data were acquired at 2.5-foot depth intervals. The velocity curves shown were computed from travel times to the near receiver (DH-98-10 x DH-98-11), travel times to the far receiver (DH-98-10 x DH-98-12), and travel time differences between the near and far receivers (DH-98-11 x DH-98-12). The boreholes penetrate 30 feet of embankment materials. Underlying the embankment materials are Quaternary fluviolacustrine deposits, including silt, sand, gravel, and volcanic ash. The two low-SWV layers approximately 70 and 100 feet deep correlate with layers of loose fluviolacustrine diatomaceous silt and volcanic ash suspected of being loose and liquefiable.

Surface-to-Borehole (Downhole) Shear-Wave Survey

During a surface-to-borehole, or downhole, shear-wave survey, a source on the ground surface generates energy, and a geophone in a nearby borehole records it (fig. 18). Data are acquired at multiple receiver depths, usually at equal increments of depth. Depending on the depth of investigation, data may also be acquired with the source offset from the borehole by varying distances. As the geophone is moved deeper into the borehole, the source is moved farther from the borehole. This offset is used to reduce possible interference from energy traveling down the borehole casing. In addition, redundancy from data recorded with multiple source offsets can help to constrain the matrix inversion used during data processing.

The shear-wave surface source produces horizontally polarized shear waves (SH waves). Reclamation's S-wave source for downhole surveys consists of a long wooden plank anchored under the wheels of a vehicle, and struck on the end by a person with a large sledge hammer. Reversed-polarity records are obtained by hitting the opposite end of the plank. Another surface source occasionally used by Reclamation consists of a short heavy metal box anchored to the ground surface by metal spikes protruding from its bottom and the weight of a person standing on it.

Although the signal to the seismograph to start recording normally comes from a triggering mechanism located on the head of the sledge hammer, it is wise to also record a trigger waveform that can provide an accurate source impact time for use in data processing. A horizontally-polarized surface geophone located adjacent to the surface shear-wave source can be used for this. Alternatively, an electrical contact closure signal between the sledge hammer and the shear-wave source can be recorded to provide very precise impact times.

A three-component geophone clamped inside the borehole casing with a bow-spring commonly records the seismic data. One of the horizontal geophone components (the transverse component) is maintained parallel to the source polarization direction throughout the survey.

The data are analyzed to determine a layered S-wave velocity model. S-wave arrival times are determined from the transverse geophone component waveforms. The arrival times are corrected using times from recorded trigger traces. A layered S-wave velocity model is then created by fitting lines to a simple time-distance plot. (Precise source-receiver distances are computed using data from a borehole deviation survey.) This method does not account for possible refraction of seismic energy at layer interfaces, however. More detailed analyses can correct for this effect. It is always helpful to use information from geologic drill logs or geophysical logs to help determine or confirm the depths of the layer interfaces. Reclamation experience indicates that velocity interfaces from downhole surveys generally correlate well (within a couple of feet) with stratigraphic interfaces determined from drilling.

Figure 19 shows results from a typical downhole shear-wave survey performed at the downstream toe of Olympus Dam in Colorado. Data were acquired at 1-foot receiver depth intervals. Data from two source locations (with 5- and 7-ft offsets) were inverted simultaneously to obtain the S-wave velocity profile presented in figure 18. Correlation with geologic contacts obtained from drill logs and geophysical borehole logs are shown. The large changes in SWV correlate well with the material interfaces.

Suspension Borehole Logging

The suspension logging probe (which may be as long as 23 to 29 feet) contains a solenoid seismic source that creates horizontally polarized shear (SH) waves in the formation being tested. Waveforms are recorded by two biaxial receivers (with vertical and horizontal components) located at the other end of the tool (fig. 20). The receiver spacing is 3.28 feet (1 m) for the OYO Model 170 suspension logger, the model used most commonly.

The probe is suspended in a fluid-filled borehole with no direct coupling to the borehole wall. The seismic source does not actually generate SH waves. Instead, it generates a P wave in the surrounding borehole fluid. The P-wave energy is partially converted to an SH wave at the borehole wall. As the P and SH waves travel through the formation materials along the borehole, a P wave is propagated back into the borehole fluid and is recorded by the receivers. The seismic energy actually travels through a zone that extends into the formation at least several inches, and in many cases a foot or more; thus, the results are mostly influenced by the formation properties and only slightly by the "mud" cake on the borehole wall. To better identify the shear-wave arrivals, the solenoid source then is activated in two opposite directions. This process is repeated at intervals of 1 to 2 feet as the suspension logger is pulled out of the hole.

Arrival times of the P and SH waves are determined from the waveforms recorded by the two receivers. The differences in arrival time are used to compute the P- and S-wave velocity at each depth. These velocities represent average velocities for the interval between the two receivers, 3.28 feet or 1 meter for the OYO Model 170.

Figure 21 shows example S-wave velocities from a suspension logger survey performed in an open borehole at Wickiup Dam, Oregon. The data were acquired in stages as the steel drill casing was removed. Some gaps in the velocity curve were caused by collapse of the borehole as the drill casing was removed. Geologic materials less than approximately 215 feet (65 m) deep consist of Quaternary fluviolacustrine clay and silt, interbedded with sand, volcanic ash, and diatomaceous silt. The variability in S-wave velocity represents varying degrees of induration, as well as varying material composition. Mudflow debris was encountered just above bedrock, but few velocities were measured in these materials due to collapse of the borehole. S-wave velocities in the basalt bedrock (below about 70 m or 230 ft deep) are highly variable due to the presence of fractures and vesicles, but are clearly much higher than in the overlying sediments.

Spectral Analysis of Surface Waves

The SASW field procedure consists of laying out a linear array of two to four vertically polarized geophones and activating a source at one end of the line to generate surface wave energy (fig. 22). The source may be impulsive, such as a hammer hit on the ground surface (or a metal plate), or a continuous vibratory source. From the geophone data, the phase velocity of the surface wave is determined for a range of frequencies that is related to the geophone spacing. The geophone array is then rearranged to have larger spacings between instruments, and the data acquisition is repeated to determine the surface-wave phase velocity for a different frequency range. Typically, three to five data sets are collected in this manner. The data are combined into one dispersion curve, which is a plot of surface wave phase velocity versus frequency.

The surface-wave velocity is related to the S-wave velocity of subsurface materials. Shallow materials affect the high-frequency surface-wave energy (collected at short geophone spacings), and progressively deeper materials affect successively lower-frequency energy (collected at longer geophone spacings). Thus, the variation of S-wave velocity with depth can be estimated from the surface wave dispersion curve. The velocity profile represents the laterally averaged S-wave velocity structure beneath the geophone array.

When inverting the surface wave dispersion curve for an S-wave velocity profile, a model is created with the smallest number of layers that can provide a good fit to the data. Initially, the best-fitting model with two or three layers is found. If the fit to the data is not very good, additional layers are added one at a time to improve the fit of the theoretical dispersion curve computed from the model to the dispersion curve obtained from the data. When the fit to the data no longer improves, the modeling is terminated. Although more geologic layers may be present, they cannot always be determined from the SASW data. As a rule of thumb, a layer whose thickness is greater than approximately one-fifth of its depth can be resolved with this method. The inversion process is nonunique; in other words, more than one velocity model may fit the data equally well. For this reason, a range of possible velocity profiles is often presented rather than a single profile.

Typical results from an SASW survey are shown in figure 23. The survey was performed downstream of the toe of Rattlesnake Dam in Colorado. The hatched areas in the shear-wave velocity plot (fig. 23) represent the uncertainties in the estimated layer velocities and thicknesses. These uncertainties are simply qualitative estimates based on several modeling results and were not computed with any statistical technique. The correlation of velocity layers with geologic material type shown at the bottom of figure 23 was inferred from a geologic drill log from a borehole located about 250 feet away from the center of the SASW survey.

Evaluation of Liquefaction Potential from Shear-Wave Velocity

Recent publications reviewed for this report on the use of shear-wave velocity in liquefaction evaluation included the NCEER report (Youd and Idriss, 1997) and a 2004 paper by Andrus, Stokoe, and Juang (2004). These present two very similar correlations that are analogous to the liquefaction triggering curves used with SPT and CPT data. They each show a family of liquefaction boundary curves of shear-wave velocities (normalized for confining stress) plotted against the cyclic stress ratio. Each family consists of curves for different fines contents. The boundary curves were determined from field performance data. The 2004 paper used a database of 193 liquefaction and nonliquefaction case histories. For more in-depth discussion of the procedure and its application, see Andrus, Stokoe, and Juang (2004) and Youd and Idriss (1997). The triggering relationship for a magnitude 7.5 earthquake published by NCEER (based on earlier work by Andrus and others) is shown in figure 24. The equivalent relationship from the 2004 paper by Andrus and Stokoe is shown in figure 25. There appear to be only minor differences between the two. Note in figure 25 that there are a significant number of nonliquefaction data above the curve; the curves are intended to represent boundaries on liquefaction potential. Late in the process of preparing this report, another study was located. This 2004 paper by Kayen *et al.* (2004) used a somewhat larger data set and reliability-type calculations to estimate the probability of liquefaction as a function or loading, SWV, and fines content, as opposed to developing deterministic boundary curves. It was noted that Kayen *et al.* (2004) show cases of liquefaction occurring with SWV as high as 240 m/s (790 ft/s), significantly higher than the boundary curves in figures 24 and 25, but there has been no further evaluation of that document for this report.

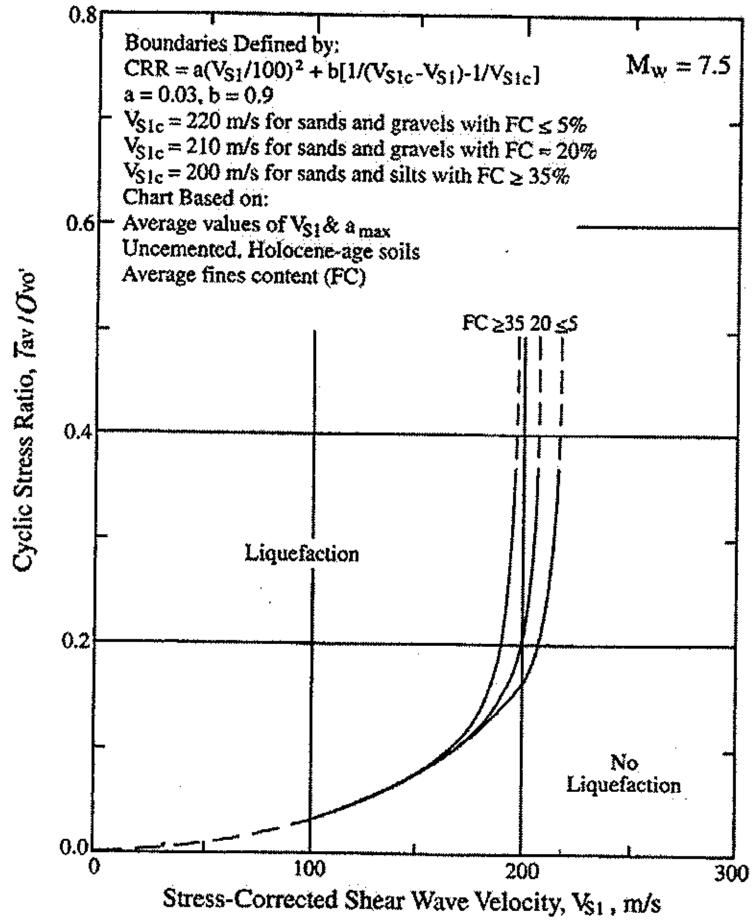


Figure 24.—V_s-based liquefaction triggering relationship from NCEER (1997).

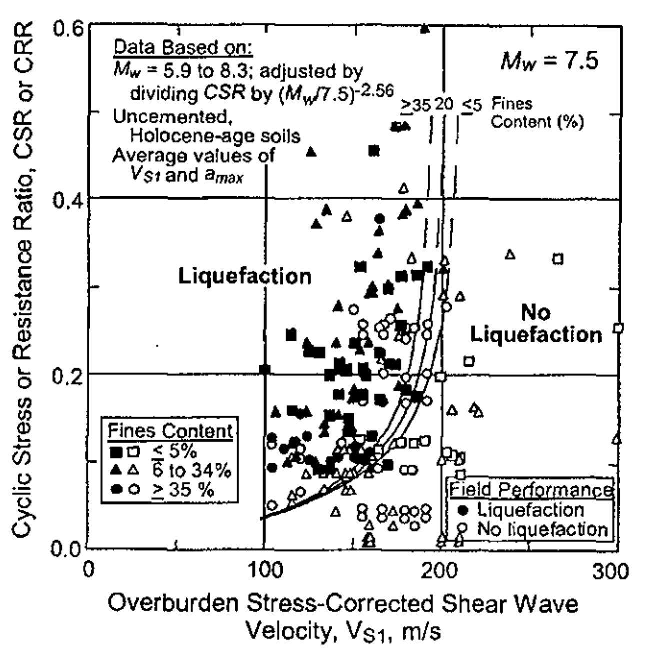


Figure 25.—V_s-based liquefaction triggering relationship from Andrus and Stokoe.

Application of *In Situ* Methods

The preceding section has attempted to describe each technique and the generally accepted application of that method for evaluating the liquefaction resistance of soils. (Again, the earthquake loading part of a liquefaction evaluation is not addressed in this report.) No judgments were applied to the specific validity or applicability of each method. This section looks in slightly more detail at each method and cites both advantages and disadvantages associated with each technique. In addition, the assumptions (or site conditions) on which the method is based are outlined. These discussions are derived from published papers, and are intended to give readers a broad perspective on the issues involved with the various *in situ* techniques. Where suggested procedures or observations in the published papers appear to conflict with current Reclamation philosophy, a note to that effect is included in the applicable discussion.

General Comparison of Methods

Use of SPT and CPT Tests for Evaluating the Liquefaction Resistance of Sands (Seed and de Alba, 1986)

When comparing the CPT and SPT methods, Seed and de Alba noted the following advantages and disadvantages:

<u>CPT</u>

- An advantage is that CPTs are more economical than SPTs
- Another advantage is that CPTs provide a continuous record of penetration resistance throughout the depth of a soil deposit, thereby providing a better portrayal of the variability of the deposit, at least in the vertical direction.
- A disadvantage stated is that liquefaction analysis depends on estimated fines content, whereas the CPT does not provide samples. However, liquefaction triggering correlations used as of 2006 (R. Seed, *et al.*, 2003; Youd and Idriss, 1997) do not require an estimate of fines content. Instead, they use the friction ratio directly, as an index of the effect of fines on liquefaction resistance.
- Another disadvantage is that CPT may not be able to penetrate very dense sands or sands at large depths, and may not be appropriate when dealing with very coarse or gravelly sands.
- A further disadvantage is that tip resistance can be influenced by soil several cone diameters below the tip (Note: although the authors did not mention it, this may also be true for the SPT).

SPT

- An advantage is that SPTs can provide samples for gradation testing. In addition, each SPT test can thus be directly related to the grain size characteristics of the sample, rather than having to infer this data.

- The large body of field data lends a relatively high degree of confidence with this method, although many more performance case histories have become available for both SPT and CPT in the intervening 20 years.

- The method can be used in coarse sands and even in sands containing small amounts of gravel; blows are recorded for each inch (or cumulative penetration is recorded after each blow).

Recent Advances in Evaluation and Mitigation of Liquefaction Hazard (R. Seed, 1996)

This paper summarizes some of the then-current practice in liquefaction evaluation, based largely on the NCEER workshop that was going on at the time. Regarding *in situ* investigation procedures, Seed wrote:

> There was strong consensus that four *in-situ* testing methods are currently sufficiently "mature" (well-documented, and well-calibrated and verified) as to serve as a useful engineering basis for evaluation of resistance to "triggering" of liquefaction, and these are (1) the Standard Penetration Test (SPT), (2) the cone penetration test (CPT), (3) the Becker Penetration Test (BPT) and (4) shear-wave velocity measurements (V_s). Of these, both the SPT and CPT are considered reliable for most sandy and low-plasticity silty soils. The BPT is a very large-scale test specifically applicable to coarser, gravelly soils and rockfill; it must be back-correlated with SPT-based empirical correlations originally developed for sandy soils, and so represents a somewhat less precise methodology. Shear-wave velocity testing can be performed over the widest variety of soil types (all types of potential interest), but V_s provides the least well-defined correlation with resistance to triggering of the four methods.

In comparing CPT to SPT, Seed gave advantages and disadvantages. Advantages of the CPT were identified as:

- The CPT is much more "standardized" and requires little or no correction for equipment or procedural variances.

- The CPT is a rapid and relatively inexpensive probe compared to the time and expense associated with boreholes and SPT.

- The CPT provides a continuous record so that thin layers and lenses are not missed between test depths.

- Thinner layers can be tested more reliably than with SPT.

Disadvantages of the CPT include:

- Its inability to provide a soil sample, leading to uncertainty of soils type, fines content, and character

- The smaller number of case histories to support the correlation between CPT and liquefaction resistance, relative to the SPT. (Note: although included by Seed in 1996, this is no longer considered a disadvantage by Seed or many others because of the much-larger case-history database now available [R. Seed *et al.*, 2003])

Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils **(Youd *et al.*, December 1997)**

Table 2, comparing the four main *in situ* techniques, was included.

Table 2.—NCEER list of advantages/disadvantages of various *in situ* techniques

Feature	Test type			
	SPT	CPT	V_s	BPT
Number of test measurements at liquefaction sites	Abundant	Abundant	Limited	Sparse
Type of stress-strain behavior influencing test	Partially drained, large strain	Drained, large strain	Small strain	Partially drained, large strain
Quality control and repeatability	Poor to good	Very good	Good	Poor
Detection of variability of soil deposits	Good	Very good	Fair	Fair
Soil types in which test is recommended	Nongravel	Nongravel	All	Primarily gravel
Test provides sample of soil	Yes	No	No	No
Test measures index or engineering property	Index	Index	Engineering property	Index

NCEER offered the following summary recommendation dealing with the use of *in situ* techniques:

> Four field tests are recommended for general use in evaluating liquefaction resistance—the cone penetration test (CPT), the standard penetration test (SPT), measurement of shear-wave velocity (V_s), and

for gravelly sites, the Becker penetration test (BPT). The workshop reviewed and revised criteria for each test to incorporate recent developments and to maximize compatibilities between liquefaction resistances determined via the various tests. Each field test has its advantages and limitations. The CPT provides the most detailed soil stratigraphy and provides a preliminary estimate of liquefaction resistance. The SPT has been used more widely and provides disturbed soil samples from which fines content and other grain characteristics can be determined. V_s measurements provide fundamental information for evaluation of small-strain constitutive relations and can be applied at gravelly sites where CPT and SPT may not be reliable. The BPT test has been used primarily at gravelly sites and requires use of rough correlations between BPT and SPT. In many instances, two or more test procedures should be applied to assure that both adequate definition of soil stratigraphy and a consistent evaluation of liquefaction resistance is attained.

Recent Advances in Soil Liquefaction Engineering: A Unified and Consistent Framework (Seed et *al.*, April 2003)

The authors commented on the comparative use of CPT and SPT methods, noting that:

- "Up to this point in time, the SPT-based correlations have been better defined, and have provided lesser levels of uncertainty. . . . CPT, however, is approaching near parity, and newly developed CPT-based correlations now represent nearly co-equal status with regard to accuracy and reliability relative to SPT-based correlations."

- "SPT-based correlations are currently ahead of 'existing' CPT-based correlations, due in large part to enhanced data base and better data processing and correlation development. . . . An additional very significant advantage of SPT is that a sample is retrieved with each test, and so can be examined and evaluated to ascertain with certainty the character (gradation, fines content, PI, etc) of the soils tested, as contrasted with CPT where soil character must be 'inferred' based on cone tip and sleeve friction resistance data."

- "The CPT offers advantages with regard to cost and efficiency (as no borehole is required). A second advantage is consistency, as variability between equipment and operators is small (in contrast to SPT). The most important advantage of CPT, however, is continuity of data over depth. . . . SPT can only be performed in 18-inch increments at vertical spacings of about 30 inches or more. . . . CPT, in contrast, is fully continuous and so 'misses' nothing." Although even the CPT has difficulty characterizing strata of less than 12 inches (due to penetration lengths needed to develop full tip resistance, the influence of softer underlying strata, and the drag length of the sleeve), it at least provides some indications of potentially problematic materials.

- Regarding which test is best, "The best answer is that both tests are far better when used together, as each offers significant advantages not available with the other."

SPT and CPT Applications and Cautions

Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils (Youd *et al.*, December 1997)

The NCEER workshop evaluated and updated the liquefaction procedures for the SPT and CPT methods and documented the state of practice for use of these methods. Virtually all of the enhancements to the methodology deal with the evaluation of the data, and little was done to modify the actual field procedures.

Workshop participants were able to reach consensus on all adjustments to the SPT methodology. However, consensus was not reached on the CPT method, with two members believing the procedure was either incorrectly developed or needed additional study prior to recommending it to the geotechnical engineering profession. Most of the other participants appear to endorse the method. Due to the increased amount of field performance data from CPT investigations, current methodology is largely based on direct correlation of liquefaction resistance with CPT data, unlike earlier approaches that converted CPT data to SPT blow counts and then applied SPT criteria.

Table 3 was included to show some limitations of the SPT test.

The method and associated curves currently in use for both CPT and SPT testing are based on the following general conditions:

- Holocene age, clean sand deposits

- Level or gently sloping ground

- Magnitude M = 7.5 earthquake

- Depth range of about 3 to 45 feet (About 84% of data are from depths < 30 feet)

- Representative average penetration resistance value is for the layer that was considered to have experienced liquefaction

The NCEER report cautions that care and judgment should be exercised when extrapolating the correlations outside of the above range of conditions.

Table 3.—Factors affecting the SPT (after Kulhawy and Mayne, 1990)

Cause	Effects	Influence on SPT N value
Inadequate cleaning of hole	SPT is not made in original *in-situ* soil, and therefore soil may become trapped in sampler and be compressed as sampler is driven, reducing recovery	Increases
Failure to maintain adequate head of water in borehole	Bottom of borehole may become quick	Decreases
Careless measure of hammer drop	Hammer energy varies (generally variations cluster on low side)	Increases
Hammer weight inaccurate	Hammer energy varies (driller supplies weight; variations of 5-7% common)	Increases or decreases
Hammer strikes drill rod collar eccentrically	Hammer energy reduced	Increases
Lack of hammer free fall because of ungreased sheaves, new stiff rope on weight, more than two turns on cathead, incomplete release of rope each drop	Hammer energy reduced	Increases
Sampler driven above bottom of casing	Sampler driven in disturbed, artificially densified soil	Increases greatly
Careless blow count	Inaccurate results	Increases or decreases
Use of nonstandard sampler	Corrections with standard sampler invalid	Increases or decreases
Coarse gravel or cobbles in soil	Sampler becomes clogged or impeded	Increases
Use of bent drill rods	Inhibited transfer of energy of sampler	Increases

Comparison of SPT-CPT Liquefaction Evaluations and CPT Interpretations (Baez *et al.*, Date unknown, but after 1997 NCEER)

In this interesting paper, Baez, Martin, and Youd review past papers and data regarding SPT-CPT correlations and fines content. They believe that the current CRR curves predicting liquefaction for the CPT and SPT may give somewhat different answers depending on which method is used. For clean-sand equivalent SPT blow counts, $(N_1)_{60\text{-}cs}$, less than about 21, the CRR predicted by the CPT may be up to 25 percent lower than that predicted by the SPT. This means the CPT method may predict a factor of safety against liquefaction lower than that predicted by the SPT. Conversely, for blow counts above 21, the CPT CRR may be up to 30 percent higher than the SPT value. The authors do not judge which method is more appropriate or accurate, although they suggest that the CPT *may* be a better model for low blow counts due to a more extensive database. They suggest that additional work should be done to improve the fit of the curves to the data.

***Probabilistic Assessment of Seismic Soil Liquefaction Using CPT* (Moss, R.E.S., and R.B. Seed, 2004)**

The authors used earthquake performance data in a probabilistic study to develop a correlation between liquefaction resistance and cone tip resistance, much like the SPT work by the same authors and others. Among the issues studied were normalization of tip resistance for overburden stress, and the effect of fines on liquefaction resistance. The former was shown to be a function of both friction ratio and tip resistance, and the results differed from previously used curves. The latter was considered, not using a measured or estimated fines content, but by correlation with the friction ratio, to estimate an adjustment Δ_{qc}. This way, the nature of the fines is included (albeit indirectly), and not just the quantity of fines. The study yielded curves of liquefaction probability as functions of CSR and normalized cone resistance.

***Evaluating soil liquefaction and post-earthquake deformations using the CPT* (Robertson, P.K., 2004.)**

This paper summarizes use of the CPT to predict liquefaction resistance of soils. The author suggests that the preferred method of using the CPT to evaluate liquefaction resistance is to plot a continuous profile of the computed CRR together with the profile of CSR for direct comparison, as shown in figure 26. While the correlation to predict liquefaction potential was based on the *average* tip resistance in the layer thought to have liquefied (thereby considering only one data point per boring), this portrayal allows evaluation of the entire soil column, including the lowest data. The author points out that use of the CPT simplified procedure with the *lowest* tip resistance is conservative because the empirical method was based on the *average.*

The author also discusses the use of "representative values" of penetration resistance in evaluating liquefaction resistance. Since the discussion relates more to use of penetration resistance values in establishing the minimum (liquefied) undrained shear strength, it is generally outside of the scope of this report. However, it is pertinent to the method of selecting representative blow counts for liquefaction triggering, and whether one should favor, the mean, the lowest, or perhaps something higher.

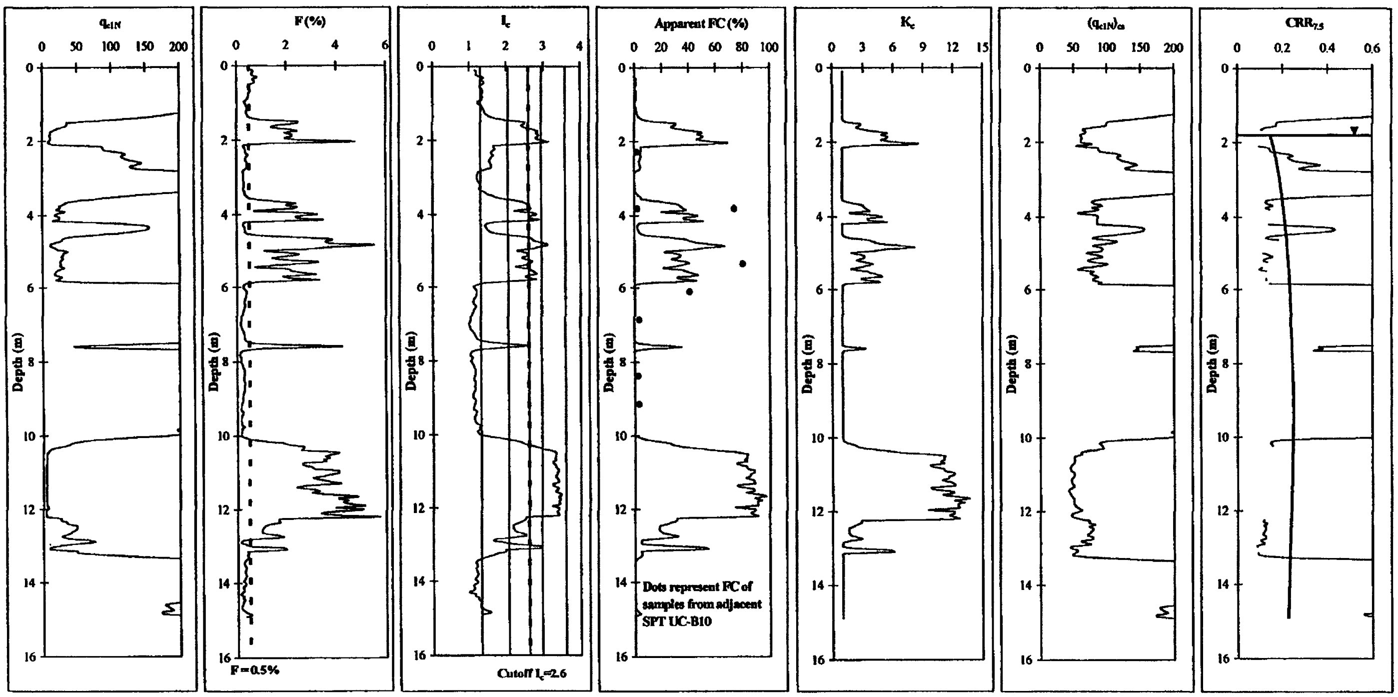


Figure 26.—Example of CPT liquefaction portrayal (from Robertson, 2004).

BPT Applications and Cautions

Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils **(Youd *et al.*, December 1997)**

The NCEER report made the following recommendations for the use of BPTs:

- The BPT should be conducted with newer AP-1000 drill rigs equipped with supercharged diesel hammers used to drive plugged (i.e., closed end) 6.6-inch (168-mm) outside diameter casing

- The measured BPT blow counts need to be adjusted to account for variations in hammer combustion efficiency. In the method of Harder and Seed, the raw blow count is adjusted using the bounce-chamber pressure to provide the adjusted value, N_{bc}, which is used to estimate the equivalent SPT N_{60}. For most routine applications, the correlations developed by Harder and Seed may be used for these adjustments. (Note: Safety investigations of high-hazard dams are generally not considered "routine applications.")

- Harder and Seed did not evaluate the effect of variation in casing friction; their correlation implicitly incorporates it, but not in a way that considers variation in the nature of the material overlying the test interval. This correlation has not been verified for depths greater than 100 feet (30 meters), and should not be relied upon as the sole method for sites with thick dense deposits overlying the loose materials being tested. For these conditions, mudded boreholes have been proposed as a possible solution to reduce casing friction, but with or without mud, wave-equation analyses such as the Sy and Campanella method should be applied to quantify the effects of shaft resistance. (Note: Reclamation has no experience with mudded holes for BPT testing, and the profession as a whole has very little. Caution is necessary, therefore. The analysis would have to account for the reduction of shaft resistance below "typical" values, so the Harder and Seed correlation would not be appropriate. If the bit diameter differs from 6.7 inches, the existing BPT-SPT correlations cannot be used at all. There could be significant environmental issues associated with containment and disposal of the used drill mud.)

Twentieth Canadian Geotechnical Colloquium: *Recent developments in the Becker penetration test: 1986-1996* (Sy, July 1997)

This paper by Alex Sy provides an excellent summary of advances in the use of the Becker penetration test. In addition, the author presents significant data on measured hammer energy and the effect of casing friction. The paper includes a clear explanation of the Sy and Campanella method.

(Note: In the paper, Sy makes a strong argument for the use of the Sy and Campanella approach (over the Harder and Seed method). However, as stated elsewhere in this report, Reclamation practice will generally be to use both methods for interpreting BPT data. If there is not at least general consensus between the two, it may be necessary to judge the relative likelihood that each is more correct, based on any available SPT or CPT data, and an understanding of site conditions such as whether the material overlying the test interval would tend to give very high or very low shaft resistance. For that particular issue, it may be helpful to refer to the literature on pile driving.)

Improvements to the Becker Penetration Test for Estimation of SPT Resistance in Gravelly Soils (Wightman *et al.*, September 1993)

The authors note that the Harder and Seed correlation between SPT and BPT blow counts does not explicitly address casing friction effects in the BPT. The Harder and Seed studies were based largely on data obtained from depths of less than 60 feet, a depth to which they concluded (from redriving tests) that variability in shaft resistance was negligible. More recent studies have indicated, however, that at depths greater than 60 feet, casing friction becomes more significant, and that redrive resistance measured at depth is a significant proportion of the total BPT blow count, which can lead to considerable uncertainty in the interpretation of equivalent SPT blow counts.

Potential impacts of casing friction include, firstly, that the friction resistance may mean the BPT blow count has little relation to the density of the soil at the Becker tip, especially when tests are performed in soft or loose soil layers of significant depth. Secondly, a BPT-SPT correlation that is based on data with significant casing friction will not be useful at sites where friction may not be significant.

The authors propose a new method of Becker drilling designed to minimize casing friction. The procedure consists of using an oversize bit (with smaller casing) as well as the use of drilling mud.

During a field testing program, the authors determined that the Harder and Seed correlation appeared to work well at their site to a depth of about 80 feet, below which the actual measured SPT blow counts were significantly lower than predicted by the BPT, due to the effects of casing friction. Thus the authors propose that BPTs below that depth would be better performed with drilling mud procedures.

(Note: Again, the use of drilling mud to reduce casing friction and the larger-diameter bit, as proposed by the authors, would differ from the conditions under which the two commonly used correlations were developed. Even with the standard 6.7-inch bit, the zero shaft resistance would need to be accounted for in the analysis. As mentioned previously, and noted under *Harder-Seed Method for Estimating Equivalent SPT Blowcounts* on page 25, the use of bentonite mud in BPT testing would need to be studied further before Reclamation would make use of it.)

Correlations of Mud-Injection Becker and Standard Penetration Tests (Sy and Lum, September 1996)

This paper compares blow counts derived from SPT, BPT, and mud-injected BPT at a sand site and at a gravel site. In order to reduce casing friction, the mud-injection technique, known as the Foundex Becker penetration test (FBPT) was developed. That consists of injecting bentonite mud just above the closed-end shoe through a series of holes in the casing. The authors tried two types of shoes, 6.7-inch (170-mm) diameter by 12 inches (305 mm) long, and another 8.7 inches (220 mm) in diameter and 16 inches (405 mm) long (6.7 inches is the standard diameter for penetration tests for assessing liquefaction potential). For this evaluation, the authors used the Sy and Campanella approach for estimating equivalent SPT blow counts from the BPT values. From the data developed for a sand foundation, it looked like casing friction became apparent at a depth of about 50 feet (15 meters) and became progressively more of a factor as depth increased.

The authors found that a reasonably consistent 1:1 correlation was observed between measured SPT values and the mud-injected 8.7-inch diameter BPT. BPT blow counts from the mud-injected 6.7-inch Becker shoe needed to be multiplied by 1.7 to get equivalent SPT blow counts. The authors concluded that the results were encouraging and illustrate the potential applicability of using mud-injected BPT to reduce casing friction and thus reducing potential uncertainty in SPT-BPT correlations. The authors concluded that this technique offers a quick and simple means of estimating equivalent SPT blow counts from a mud-injected BPT, without the need to determine casing friction.

Becker Research Proposal—Becker Penetration Testing (BPT) in Gravel Quarries with Known Properties (Farrar)

In discussing the benefits of a potential research proposal, Farrar makes the following observations:

- BPT correlations are complicated by variability in energy and shaft friction. Recent work (within the last 5 years) has resulted in adjustments for these, although they are not definitive, particularly with respect to friction.

- A significant drawback of the BPT is the lack of a sample at the precise location of the test.

- Whereas SPT-based liquefaction assessment is backed by the chamber testing that established relationships between penetration resistance and density, no such data exist for BPT (due in large part to the size of chamber that would be required).

- It has been estimated that particles from ¼ to ¾ of the diameter of the SPT penetrometer would begin to interfere with penetration of the sampler, causing overprediction of liquefaction resistance.

- If a similar phenomenon holds for BPT, particles larger than around 2 or 3 inches would have an impact on BPT blow counts, similarly causing overprediction of liquefaction resistance.

- In coarse sands and gravels, BPT should be more reliable because of the particle-size effects.

Shear-Wave Velocity Applications and Cautions

Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils (Youd *et al.*, December 1997)

For the shear-wave velocity method, NCEER published a curve for predicting liquefaction based on the CSR and normalized shear-wave velocity. For a magnitude 7.5 earthquake and clean sands, no liquefaction is predicted if the stress-normalized shear-wave velocity exceeds 720 ft/s. Whereas R. Seed, in the paper described below, indicates shear-wave testing should only be used as a screening method, NCEER did not make a similar recommendation.

(Note: Reclamation may at times rely mainly on shear-wave velocity testing to determine liquefaction resistance, but generally only in soils that are too coarse to yield reliable data with SPT or BPT. It is always preferred to use a second method if at all possible. Reclamation does conduct shear-wave testing in many seismic evaluations of dams, because it provides not only an indication of liquefaction resistance, but also necessary data for dynamic ground-response analyses.)

Recent Advances in Evaluation and Mitigation of Liquefaction Hazard (R. Seed, circa 1996)

In this paper, Ray Seed summarizes some of the latest practices in liquefaction evaluation, based largely on the NCEER workshop that was in progress at the time. Regarding shear-wave velocity procedures, Seed said:

> It should be noted that it was the strong consensus of the expert panel that V_s provides a much less well-defined correlation with liquefaction resistance than do the previously discussed penetration tests. Accordingly, they recommend that V_s be employed principally as a "screening" tool, and that evaluations for soils falling near to the boundary be considered inconclusive, so that one of the other, better defined (penetration-based) correlations should then be employed to resolve the uncertainty. This author further notes that: (a) V_s is a nondestructive, micro-strain measurement, whereas both liquefaction and penetration testing are more "destructive" and larger strain phenomena, and (b) V_s can be more strongly influenced by aging effects and subtle micro-cementation or bonding of particle contacts than liquefaction resistance, so that V_s correlations should be used with extra caution in deposits where these effects may be significant.

Recent Advances in Soil Liquefaction Engineering: A Unified and Consistent Framework (Seed *et al.*, April 2003)

Noted advantages of the shear-wave velocity method include:

- V_s can be measured with nonintrusive methods.
- V_s can be measured in coarse soils (gravelly soils and coarser) in which SPT and CPT can be obstructed by interference with coarse soil particles.
- As such, V_s-based methods can provide both a potentially rapid screening method and a method for assessment of coarse, gravelly soils which cannot be reliably penetrated or reliably characterized with small diameter penetrometers (SPT and CPT).

Noted disadvantages include:

- The correlation between liquefaction potential and cyclic stress ratio is less well defined than those for the SPT or CPT. This is due both to a considerably smaller database and to the fact that V_s is a small-strain measurement which cannot necessarily be expected to correlate well with a large-strain property like liquefaction resistance. Penetration resistance involves very large strains, in contrast.

- Small amounts of aging and particle cementation can cause V_s to increase more rapidly than the corollary increase in liquefaction resistance. The relationship between V_s and the CSR required to induce liquefaction can therefore vary significantly with the geologic age of the deposits.

- There is uncertainty regarding the appropriate normalization of V_s for the effective overburden stress.

In light of these issues and uncertainties, the authors of the referenced paper believe that the use of current V_s–based correlations to evaluate liquefaction triggering should be employed either conservatively, or as a preliminary or approximate screening tool to be supplemented by other methods.

Guide for Shear-Wave-Based Liquefaction Potential Evaluation (Andrus and Stokoe, May 2004)

This 2004 paper presents the authors' latest guidelines in assessing liquefaction potential with shear-wave velocity testing. According to their 1997 NCEER paper, their correlation data from 20 earthquakes and shear-wave velocity measurements at over 50 different sites, for a total of 193 liquefaction and nonliquefaction case histories. In this latest paper, an additional listing of the advantages and disadvantages of the various shear-wave tests is presented, along with guidance for applying age correction factors. In addition, two case histories are included, in which shear-wave velocities were measured at sites that had previously experienced liquefaction.

Both case histories involve the 1983 magnitude 6.9 Borah Peak earthquake in Idaho. The first site is referred to as the Andersen Gravel Bar Site and involves a gravel bar in the Big Lost River. The soils were described as sandy gravel with a few thin sandy silt layers. Liquefaction was observed by the presence of cracking and water spouts. The estimated ground motion at the site was 0.29g. Measured crosshole shear-wave velocities in the critical zone (corrected for overburden pressures) were all less than 575 ft/s (with about half less than 500 ft/s). The simplified procedure suggested a factor of safety against liquefaction ranging from about 0.75 to 0.95 for the approximate 12-foot depth. These results indicate good agreement between predicted and observed liquefaction behavior.

The second case history was the Larter Ranch site, on the Elkhorn alluvial fan. Liquefaction was indicated by observed water spouts, as well as the presence of sand boils, cracking, and sliding in other areas of the fan. The corrected crosshole shear-wave velocities measured within the critical zone ranged from 660 to 820 ft/s, while the estimated peak horizontal ground motion was estimated to be 0.5g. The resulting factors of safety within this 9-foot critical layer ranged from about 0.25 to 0.9, with about a 5-foot layer having safety factors between 0.24 and 0.4. This prediction of liquefaction agreed with the actual site observations.

In addition to these two case histories, Andrus and Stokoe made two conclusions which appear to be significant:

- "It is the authors' position that final site-specific liquefaction evaluations using only or primarily the V_s method should be limited to situations where (1) crosshole, downhole, suspension logger, or SASW tests are conducted such that high-quality V_s values are determined at intervals of at least one-quarter the thickness of the critical layer, (2) appropriate consideration is given to the limitations listed in table 2 (essentially a discussion of limitations/applications of the various shear-wave tests), (3) sufficient borings or soundings are conducted to identify the material type and to insure that thin liquefiable strata are not present, and (4) the critical layer is Holocene in age and contains little or no carbonate. In general, borings should always be a part of the field investigations."

- "The procedure by Andrus and Stokoe (2000) was developed using data limited to relatively level ground sites, uncemented soils of Holocene age, average depths less than about 10 m, groundwater table depths between 0.5m and 6m, and measurements from below the water table. Greater care should be exercised when applying the procedure to sites with different conditions."

Special Notes on Gravelly Soils

Liquefaction Assessment of Gravelly Soils for Dam Safety Evaluation (Yan and Lum, June 2003)

This paper by authors from BC Hydro discusses the difficulties of evaluating liquefaction potential in gravelly soils, as well as proposed methods for doing so. In addition, the authors provide a good summary of case histories of reported liquefaction in gravelly soils. Table 4 contains a listing of those case histories.

With respect to these case histories, the authors make the following conclusions:

- There is ample evidence that liquefaction of loose to medium dense gravelly soils can and does occur during earthquakes. SPT or equivalent SPT blow counts measured in the liquefied gravelly deposits are generally less than 15 blows/0.3 m.

- Based on the case histories examined, with the exception of the Friuli site, liquefaction of gravelly soils has not been reported for earthquakes with $M < 6.8$.

- Where gravel particles float in a finer-grained matrix, the matrix appears to control the liquefaction resistance.

- All cases where liquefaction was observed showed that either the gravelly soils have low permeability, or drainage was impeded by the presence of an impervious layer.

- Seed's liquefaction triggering chart developed for sands appears to be applicable to gravelly sands that have similar equivalent SPT-$(N_1)_{60}$ values.

Table 4.—Gravel liquefaction case histories (Yan and Lum, June 2003)

Site	Earthquake	Material	Density
Alluvial fan, Japan	Fukui M7.3, June 28, 1948	Gravelly sand	
Aluvial fan, Valdez, Alaska	Alaska M8.4 earthquake, Mar. 27, 1964	Gravelly sand with some silt	Loose to med.
Baihe Kam (66 m high), China	Tangshan M7.8 earthquake, July 28, 1976	Sand-gravel (u/s slope)	Dr < 60%
Pence Ranch, Whiskey Springs, Idaho, USA	Borah Peak M7.3 earthquake, Oct. 28, 1993	Clean gravelly sand to sandy gravel	$(N_1)_{60}$ = 5-14 $(N_1)_{60}$ = 8
Friuli, Italy	Friuli earthquake M6.1 to M5.2, 1976-1977	Gravelly sand	N = 10 to 20
Spitak Hwy. Embankment (site 1). Nalband Railway Embankment (site 3)	Armenia M6.8 earthquake, Dec. 7, 1988	Gravelly sand (site 1) Silty gravelly sand (site 3)	$(N_1)_{60}$ = 9-11 (site 1) $(N_1)_{60}$ = 3-11 (site 3)
Southern Hokkaido, Japan	Hokkaido-Nanseioki M7.8 earthquake, July 12, 1993	Gravelly volcanic debris	N = 8 to 16
Port Island, Japan	Kobe earthquake M6.9, Jan. 17, 1995	Coarse gravelly sand (decomposed granite)	Ancient evidence of liquefaction of gravelly soils
Lake Biwa, Japan	Unknown	Ancient evidence of liquefaction of gravelly soils	
Strait of Messina, Italy	Unknown	Evidence of liquefaction in recent Messina gravels	

Although outside of the scope of this report, it is worth noting that two of the case histories provided sufficient data to allow the residual shear strength of the gravelly soils to be back-calculated. From this effort, the authors noted that the calculated strength values were well within the typical data band relating blow counts to residual strength. Thus, they conclude: ". . . available limited case histories suggest no apparent difference in residual strengths between gravelly soils and silty sand with similar equivalent clean sand SPT-$(N_1)_{60}$ values. Until more field evidence is available, there is little justification to deviate from using the procedures for sandy soils in the evaluation of liquefaction of gravelly soils and its consequences."

Recent Advances in Soil Liquefaction Engineering: A Unified and Consistent Framework (Seed *et al.*, April 2003)

In this paper, Ray Seed and colleagues note that SPT can be used in gravelly soils, provided careful steps are taken to minimize gravel influence. They note that short-interval SPT can be effective when the minus ¼-inch fraction of the soil (all sizes smaller than gravel) comprise at least 50 percent of the total gradation. The authors report that this approach has correlated well with BPT tests in these types of soils. In this approach, blow counts are counted in 1-inch increments rather than 6-inch increments. Blow counts are plotted for each successive inch, for a total of 12 inches. The resulting plot would be similar to figure 4. (Note that Reclamation typically measures cumulative penetration after each blow, or for 0.1-foot intervals.) When the slope of the plot begins to increase, it is assumed to be due to gravel influence, and those blow counts are not considered representative. The gravel-adjusted blow count is the sum of the intervals that do not appear to show gravel interference, scaled to a 12-inch total length. The authors urge caution with this approach, however, since short-interval SPT blow counts can still be biased to the high side due to undetected influence of coarse particles. To account for this, it may be appropriate to use somewhat lower blow counts than average or typical in representing a given stratum.

Liquefaction Assessment of Gravelly Soils for Dam Safety Evaluation (Yan and Lum, June 2003)

A slightly different process used by BC Hydro to account for gravel influence in SPT tests is to record SPT blow counts for every 1 or 2 inches of penetration. The equivalent SPT blow count can be considered as the lowest 1- or 2-inch value multiplied by either 12 or 6. When this procedure was applied to Keenleyside Dam, the adjusted blow counts were naturally less than the blow counts for the full 12-inch test sum. However, the difference was less when the summary, or full, blow count was low, suggesting that lower blow counts are less affected by the gravel. The authors suggest that the equivalent blow count method based on penetration for small increments might better characterize the penetration resistance of the finer grained matrix material within gravelly soils. (Note: Although not specifically stated in the reference, some caution should be applied in the use of this approach, as it could lead to unnecessary conservatism. It is possible that very short intervals of low blow count material might be characterizing a condition where there is insufficient continuity in the matrix material to lead to the development of an entire shear plane representative of the low blow count. There is also the question of how gravel, or simply variability in density within the 12-inch test interval, may have influenced the blow counts used for the SPT liquefaction correlation.)

Probabilistic Relationships for Use in Risk Analyses

Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils (Youd *et al.*, December 1997)

The NCEER report discussed (and included a paper on) the use of probabilistic analyses of case history data for predicting liquefaction. The final report stated that, "Although risk analyses for several localities and facilities have been made using probabilistic criteria, the workshop attendees agreed that probabilistic procedures are still outside the mainstream of standard practice." However, the participants did agree that research and development in this area should continue. Following are some of the specific conclusions in the paper dealing with probabilistic approaches:

- Liao *et al.* (1988) conducted probabilistic regression analyses for clean sands (fines contents less than or equal 12 percent) and silty sands (fines contents greater than 12 percent). The proposed contours of liquefaction probability for clean sands are shown in figure 27. The Liao *et al.* analysis indicates that

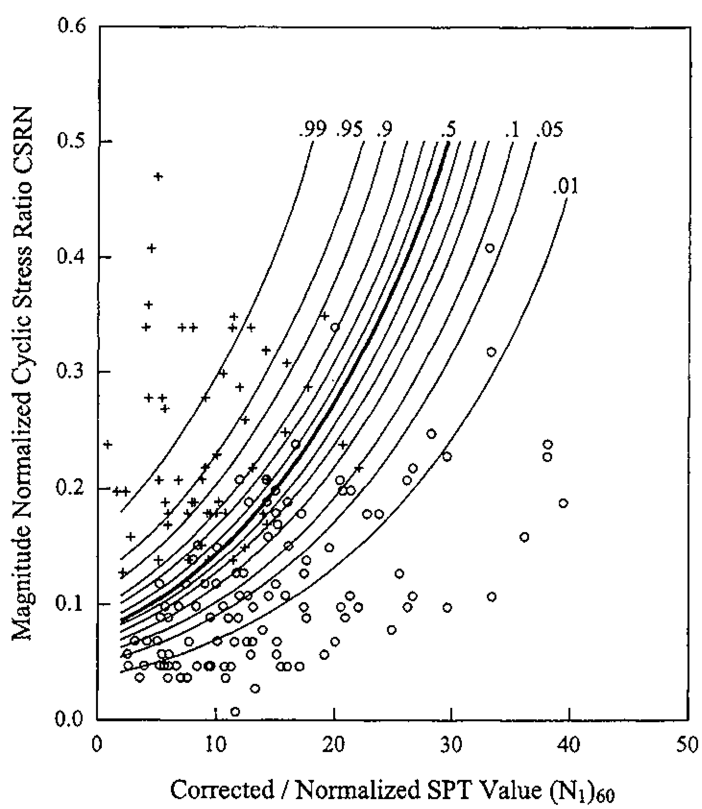


Figure 27.—Liao *et al.* probablistic curve for SPT liquefaction resistance of clean sands (as shown in NCEER, modified from Liao, 1996).

the simplified base curve for clean sands (NCEER, 1997, or Seed and Idriss, 1971) is characterized by a probability of liquefaction of about 20 percent for (N1)60 between 12 and 28.

- The 50 percent probability curve for silty sands (shown on fig. 28) determined by Liao *et al.* lies to the left of the 50 percent probability curve for clean sands, indicating that on average, silty sands are more resistant to liquefaction than clean sands for a given value of $(N_1)_{60}$. This result is consistent with the fines content adjustment incorporated into the Seed-Idriss simplified procedure. For a particular value of CRR, the difference in the blow counts corresponding to 10 percent probability and to 90 percent probability is very large, particularly for silty sands. There is considerable overlap between the band of CRR-$(N_1)_{60}$ data points for liquefaction, and the band of nonliquefaction points. The difference between the 10-percent blow count and the 90-percent blow count is so large that use of probabilistic curves for silty sands is questionable, at least at low- and high-probability levels.

- Youd and Noble used the probabilistic regression technique of Liao *et al.*., but with magnitude added as an independent variable and using an enlarged case history data set. Sands with fines contents as high as 35 percent were used by adjusting $(N_1)_{60}$ for fines content. The resulting curves are shown in

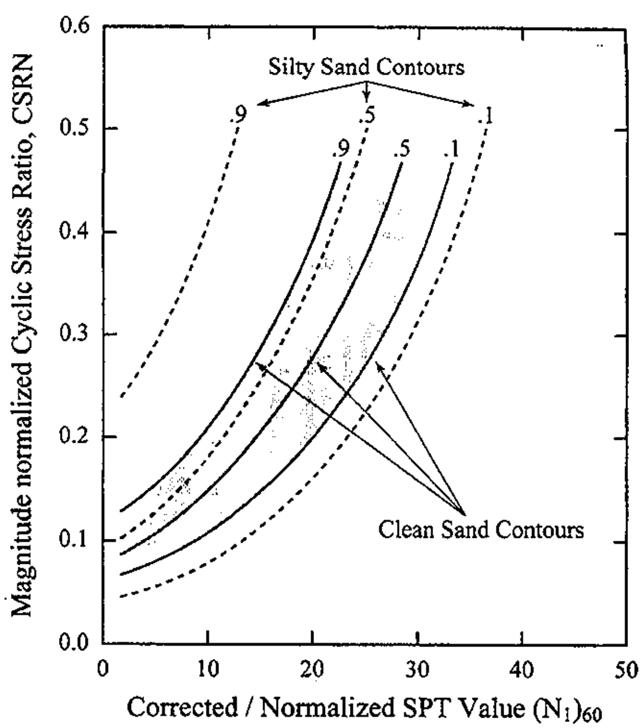


Figure 28.—Comparison of Liao *et al.* probabilistic curves for liquefaction resistance of clean sands and silty sands (as shown in NCEER, modified from Liao, 1996).

figure 29. CRR values predicted by this procedure vary from Liao *et al.* in several key aspects. The curves by Liao *et al.* have the same general shape as those by Youd and Noble but are flatter in aspect and intersect the CRR axis at a higher CRR. Also, the spread between the 20- and 50-percent curves of Youd and Noble is greater than in Liao's work. It should also be noted that Youd and Noble's regressed magnitude adjustment is quite different from the ones developed by Seed and Idriss, and by Idriss, also reported in the NCEER volume.

- The Youd and Noble analyses indicate that the Seed-Idriss simplified base curve for magnitude 7.5 earthquakes is characterized by probabilities ranging from 20 to 50 percent for $(N_1)_{60}$ ranging between 5 and 25. This implies that the simplified base curve is not as conservative as previously thought, and therefore might not be appropriate for many engineering applications.

- Probabilistic procedures provide more statistically rigorous criteria for defining liquefaction resistance than was used in the original development of the simplified procedure. As with all empirical methods, however, the quality of the results strongly depends on the quantity and quality of the compiled input data. Between magnitudes of 5.75 and 7.75, and up to cyclic resistance ratios of 0.4, the probabilistic curves appear to be well constrained by data. Extrapolation beyond these limits leads to uncertain and perhaps erroneous results.

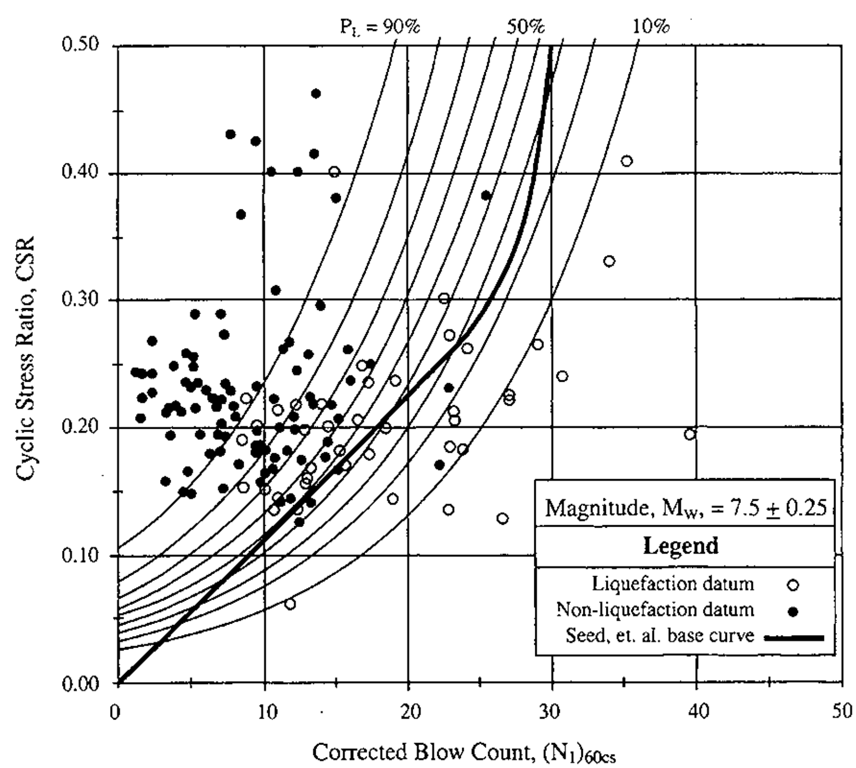


Figure 29.—Youd and Noble probabilistic curve for SPT liquefaction resistance of clean sands (as shown in NCEER).

Recent Advances in Soil Liquefaction Engineering: A Unified and Consistent Framework **(Seed *et al.*, April 2003)**

One key contribution of this comprehensive paper by Seed, Cetin, and others is an updating of the simplified method for liquefaction prediction and the development of a new probabilistic curve for SPT triggering in clean sands, advancing earlier work done by Liao *et al.*, Youd and Noble, and Tarprak *et al.* The proposed curve, shown in figure 30, features a family of five curves representing probabilities of 5, 20, 50, 80, and 95 percent. These curves are much closer to each other than those from the previous studies; that is, for a given CRR, the difference in blow count between 10 percent and 90 percent probability is much smaller. A number of improvements and adjustments were made to the simplified procedure in order to update that method. According to the authors:

> Key elements in the development of this new correlation were: (1) accumulation of a significantly expanded database of field performance case histories, (2) use of improved knowledge and understanding of factors affecting interpretation of SPT data, (3) incorporation of improved understanding of factors affecting site-specific ground motions (including directivity effects, site-specific response, etc.), (4) use of improved methods for assessment of *in-situ* cyclic shear stress ratio (CSR), (5) screening of field data case histories on a quality/uncertainty basis, and (6) use of higher-order probabilistic tools (Bayesian Updating). . . . The resulting relationships not only provide greatly reduced uncertainty, they also help to resolve a number of corollary issues that have long been difficult and controversial, including: (1) magnitude-correlated duration weighting factors, (2) adjustments for fines content, and (3) corrections for effective overburden stress.

Using essentially the same approach, the authors have also updated the CPT simplified method, and developed a probabilistic curve for CPT triggering, shown in figure 31. According to the authors:

> Overall, the new correlations are in very good overall agreement with previous, similar CPT-based efforts with regard to "clean sands." . . . It is principally when dealing with silt and silty, sandy, clayey soils that the new correlations differ significantly from earlier and widely used CPT-based correlations. The new relationships reflect a much smaller adjustment (increase) in modified CPT tip resistance ($q_{c,1,mod}$) as apparent fines content and plasticity increase than the earlier relationship of Robertson and Wride (1997), suggesting that the earlier relationship can be significantly unconservative for these soils. . . Overall, the new CPT-based relationships appear to be largely compatible with the similarly improved SPT-based relationships proposed by Seed *et al.* (2001), and the new CPT-based relationship appears to have similar levels (only marginally higher) of uncertainty

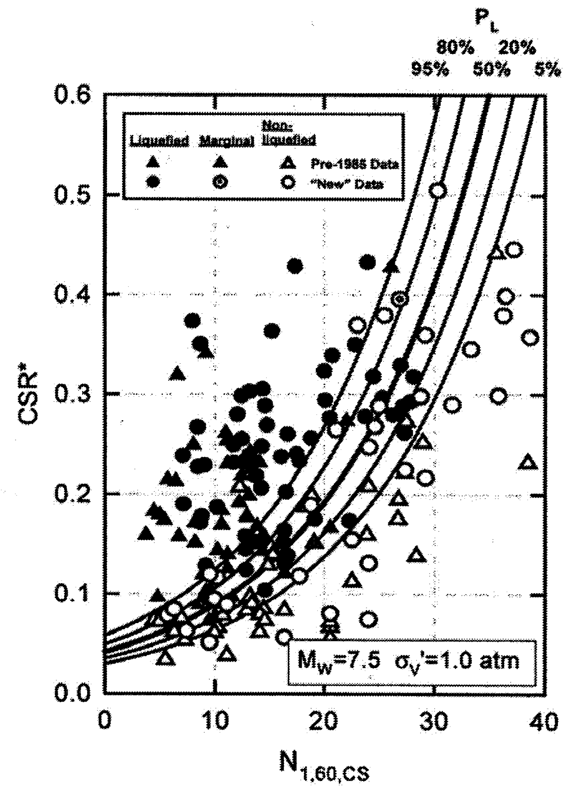


Figure 30.—Probabilistic curve for SPT liquefaction evaluation.

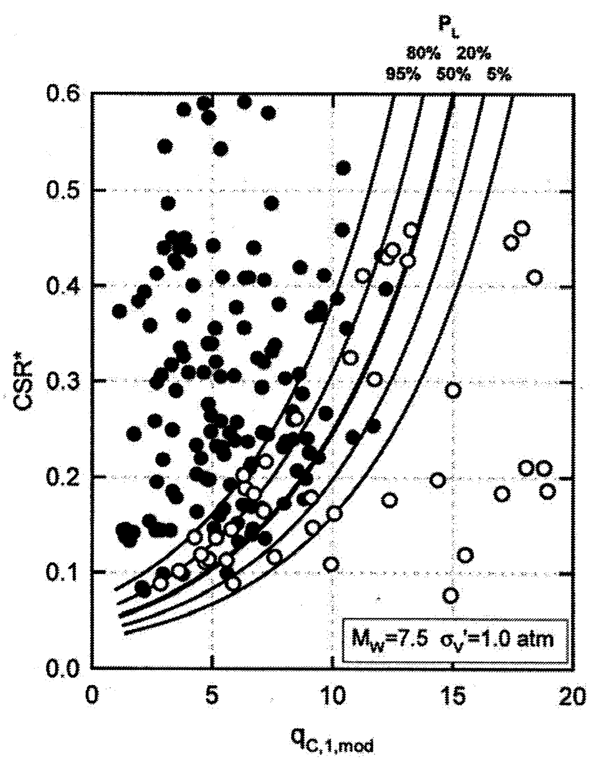


Figure 31.—Probabilistic curve for CPT liquefaction evaluation.

(or variance) associated with assessment of liquefaction triggering potential as the new SPT-based relationship. This does not mean that the SPT-based relationships are intrinsically "better": the use of CPT offers important advantages with regard to continuity of penetration data, and also the ability to discern and characterize thinner strata, than SPT (while the SPT offers increased certainty as to soil type and character, especially invariably stratified soils). Accordingly, both methods have significant relative advantages, and both are likely to be of continued significant value to working engineers.

(Note: Reclamation frequently employs probabilistic techniques for evaluating dam safety issues; the probabilistic curves of liquefaction resistance are important tools. At this time, Reclamation tends to use all of the curves discussed herein, often assigning weighting factors to each based on assessments of the applicability of each curve to the specific site being evaluated.)

Comparison With In-Place Density Data

An important measure of an *in situ* technique's reliability would be to compare it to cases where in-place densities are known. In other words, how well does a penetration value or shear-wave velocity correlate to a relative density or percent compaction value? (Density, while the most important factor in evaluating a soil's liquefaction potential, is not the only factor. Other factors, including aging or cementation, mode of deposition and compaction, and prior cyclic loading history, all play potentially significant roles as well. Thus, prediction of density is not the ultimate test of an *in situ* method.)

An important difficulty in comparing measurements from *in situ* techniques with density is that there may be as many uncertainties associated with the measurement of in-place density and relative density as there are with the *in situ* techniques. Farrar (1998, 1999) and H. Seed and de Alba (1986) discuss these concerns. Following are some of the issues and potential difficulties that can lead to uncertainty in the measurement of in-place densities and laboratory maximum/minimum values.

- Soils susceptible to liquefaction are typically saturated, loose, and relatively cohesionless. These materials are notoriously difficult to sample, and it is extremely difficult to obtain and transport an undisturbed sample. Sample disturbance is critical when determining the liquefaction resistance of loose soils.

- Test pits can be used to sample and conduct in-place density tests in relatively shallow soil deposits. However, easily accessible shallow soils are frequently not saturated and therefore not susceptible to liquefaction. The deposits of interest for liquefaction are below the water table (at least part of the time), so test pits for sampling or density measurements must be thoroughly dewatered before excavation begins. Often the critical layers for liquefaction susceptibility are beyond the depth of convenient test pit excavations.

- If test pits or trenches are used, disturbance and densification can result from the vibrations of the excavation equipment used to dig the exploration hole.

- If dewatering is used in conjunction with a test pit or trench, the soils may densify as a result of increased capillary stresses. However, in clean sands (which are not highly compressible), this may not result in any significant effect.

- To test deeper soil deposits, large diameter shafts, essentially deeper test pits, are sometimes used. However, shafts are typically expensive and time-consuming. Like test pits, densification of the soil can result from dewatering or from the equipment used to excavate the test shaft and drive the large-diameter casing required.

- In sand-cone or large ring density tests taken in test pits or shafts, hole squeezing in wet soils can lead to significant overestimation of in-place density. This generally is not a factor in free-draining, cohesionless sands.

- As soils get coarser, larger in-place density tests are required. These tests are difficult and time-consuming, and even then, "scalping" of oversize material (exceeding some particle size that depends on the dimensions of testing equipment, up to 3 inches) may be required. In this case, the minimum and maximum densities for the smaller fraction are tested in the lab, and adjustments are made for the oversize fraction. The adjustment introduces considerable uncertainty, particularly if the oversize material is a large fraction of the whole soil. In addition, corrections may be required for the roughness of the sides of density test holes in coarse material. Failure to correct for roughness could result in relative density differences of 10 percent, which could make the difference between a material that is dense enough to resist liquefaction and one that could be problematic in an earthquake. The 10-percent density difference is a value observed by Reclamation during testing of alluvium at Bradbury Dam (Farrar, 1998).

- If different layers of soils are encountered in an in-place density test hole, the laboratory maximum and minimum densities could be misleading. Generally, a well-graded soil with a mixture of particle sizes has a higher maximum density than one with a more uniform gradation. If a layered soil deposit were excavated for a density test, then mixed and tested for maximum density, it would likely have a higher maximum density than the individual layers would. The calculated relative densities of the individual sublayers of soil within the test pit would likely be significantly lower than they really are if the maximum density of the "mixed" composite soil sample were used as a reference.

- The relative density test itself has a number of potential difficulties. These include variability among operators in arriving at the minimum density and the need for careful and frequent calibration of shaking tables for the maximum-density determination. In addition, different maximum densities can be determined depending upon whether the dry or wet method is used. In some cases, these differences can be quite large, and result in relative density values that may vary by tens of percent. The two values potentially could indicate very different expectations of the soil's behavior under cyclic loading, possibly even the difference between liquefaction and dilative behavior. (Note: It is recommended that both methods of determining maximum density be performed at the beginning of a testing program to determine which is more suitable.) All relative density testing should strictly follow the procedures outlined by ASTM International (ASTM) or Reclamation for determining minimum and maximum densities for cohesionless soils.

- As an alternate to test pits or shafts, sophisticated sampling procedures have been developed for soils at depth. One involves ground freezing and coring of the frozen soil. Unfortunately, such methods are difficult, expensive, and time consuming. There is also potential for some soil disturbance in spite of careful application of sophisticated sampling techniques.

- Finally, as H. Bolton Seed and de Alba discuss (1986), some natural deposits of soil, particularly sand, are very nonuniform. The establishment of representative soil properties requires a substantial number of tests on samples from different critical parts of the soil deposit. Whereas this can be accomplished relatively easily and economically with *in situ* tests such as CPT, SPT, BPT, and V_s measurement, it becomes far more difficult, expensive, and time consuming with test pits and exploration shafts for in-place density testing.

Laboratory Tests and Past Evaluations of Liquefaction Utilizing Density Measurement

Prior to discussing actual case histories of in-place density measurements with *in situ* investigation values, some of the original work which helped shape the thinking about liquefaction potential and its relationship to relative density and *in situ* techniques will be reviewed.

Laboratory Tests Comparing SPT N-Values to Relative Density

Some of the earliest published work was undertaken by Gibbs and Holtz of the Bureau of Reclamation, who conducted SPTs in "chamber tests," which essentially involved placing sands with controlled density in large containers and conducting the tests in the containers at different confining pressures. Gibbs and Holtz found that a reasonable correlation exists among N values, relative density, and confining pressure. They displayed this relationship by means of the plot shown in figure 32.

In the 1970s, researchers at the U.S. Army Waterways Experiment Station (WES) conducted studies comparing N values to in-place density and relative density. (Bieganousky and Marcuson, 1976; Bieganousky and Marcuson, 1977; Marcuson, 1977) The referenced reports discuss limitations and difficulties associated with measuring in-place density, determining relative density, and obtaining SPT blow counts. A significant conclusion of this work was that the sampling process was found to change the density of tube and piston samples (which may well have played a role in moving the state-of-the-practice at that time toward *in situ* testing). As a part of this study, tests were conducted at WES to gain additional understanding of the relationships. The result was that "the SPT is not sufficiently accurate to be recommended for final evaluation of the density or relative density at a site, unless site-specific correlations are developed." Also, "The spread of data derived from testing four sands under optimum laboratory conditions suggests that a simplified family of curves correlating SPT N values, relative density, and overburden pressure for all cohesionless soils under all conditions is not valid." They recommended that "additional research is required to evaluate these factors."

Other researchers, particularly in Japan, have conducted extensive investigations into the relationship between N value and density, using densities and void ratios determined from undisturbed samples obtained by ground freezing.

A recent Ph.D. thesis by Jiaer (Jerry) Wu at the University of California at Berkeley, compared relative density to SPT blow count (Wu, R. Seed, and Pestana, 2003). Figure 33, from that publication, shows relatively good agreement among correlations from a number of researchers, although the comparison is not exactly direct because different curves use different forms of the SPT blow count (N, N_1, $N_{1\text{-}60}$, or N_1 determined by the Japanese standard). The Tokimatsu and Seed curves and "This Study" are the most similar, with one using $N_{1\text{-}60}$ and the other $N_{1\text{-}60\text{-}cs}$. Note that the 1957 Meyerhof correlation is based totally on the Gibbs and Holtz work cited above. The term "apparent relative density" is used to mean the value that would be predicted by each correlation.

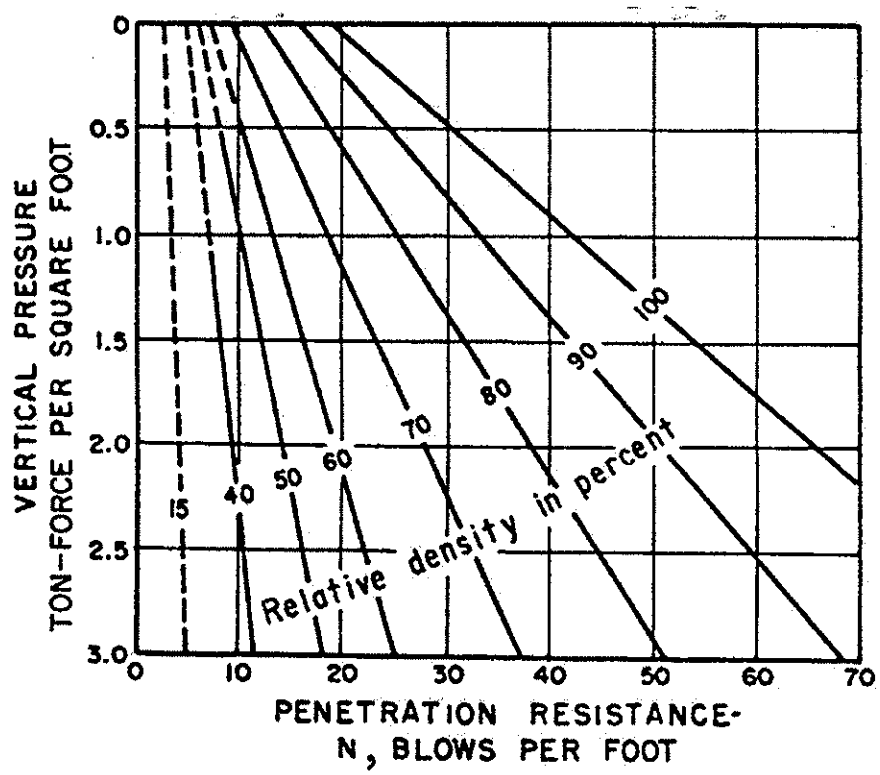


Figure 32.—Gibbs and Holtz correlation.

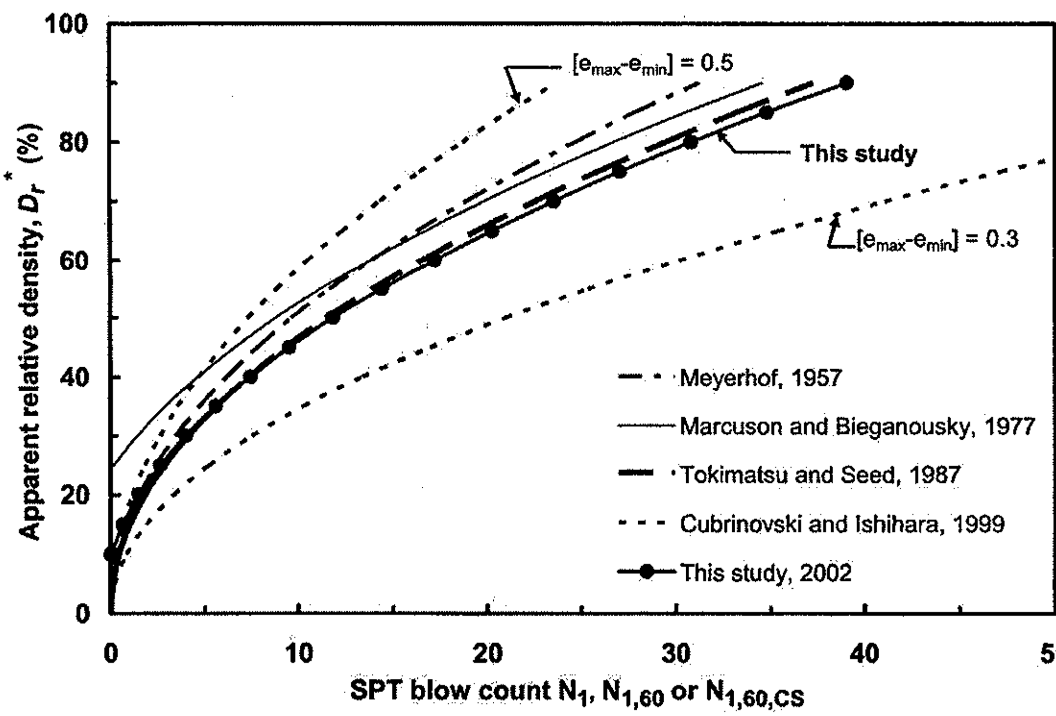


Figure 33.—Various correlations between apparent (predicted) relative density and SPT blow count (Wu *et al.*, 2003).

Early Efforts Relating Liquefaction Potential to Relative Density

Whereas the Seed-Lee-Idriss simplified procedure is a correlation between liquefaction and SPT blow counts used as an index of density, an earlier paper (November 1970) by H. Seed and Idriss presented a correlation between liquefaction and relative density. This early work was based primarily on laboratory cyclic triaxial testing of two sands at varied relative densities and confining pressures. Figure 34 shows the correlation in a form similar to the familiar correlation with SPT.

They performed additional analysis of field case histories of liquefaction. Since relative density information was not available for most of the sites that had experienced liquefaction, the authors extended the study by converting the more readily available SPT blow counts to relative density by using the Gibbs and Holtz correlation. They used the field performance data to create boundary plots for liquefaction potential under a known depth of water table, ground shaking, and relative density. An example is shown in figure 35.

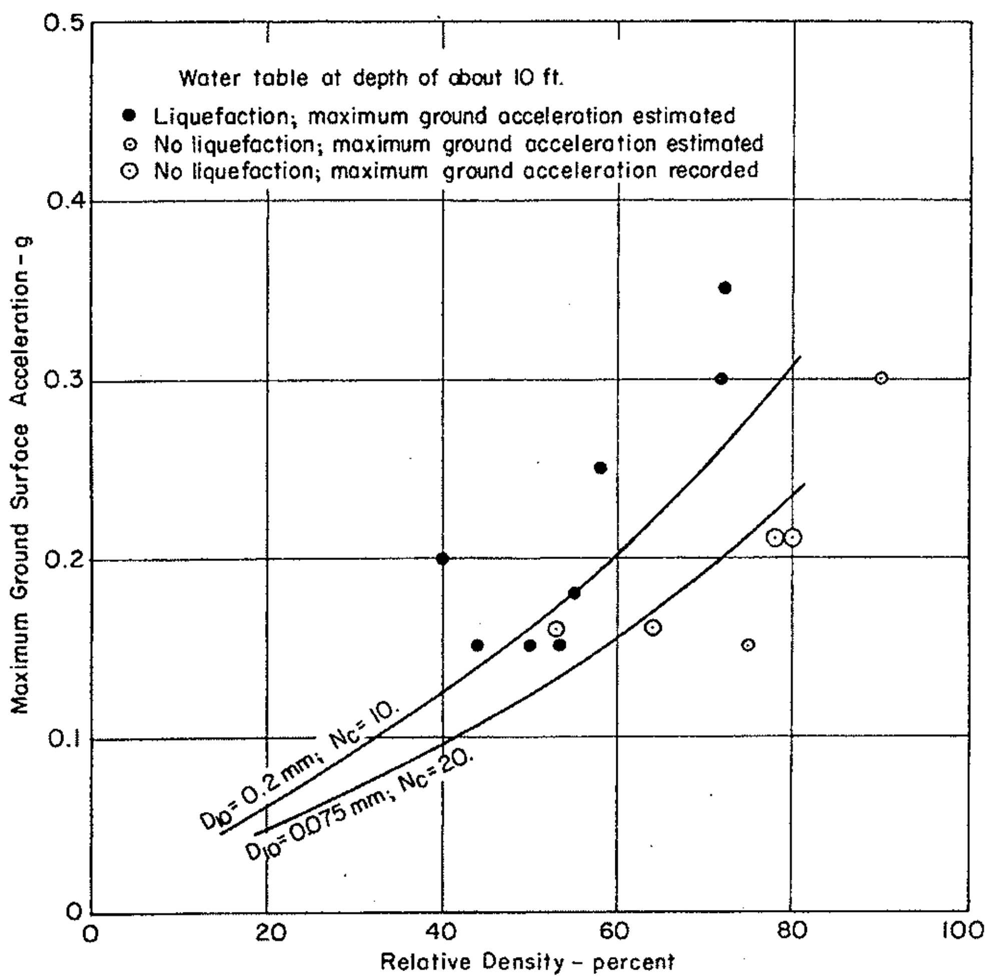


Figure 34.—Evaluation of liquefaction potential for fine sands water table 10 ft below ground surface (H. Seed and Idriss, 1970).

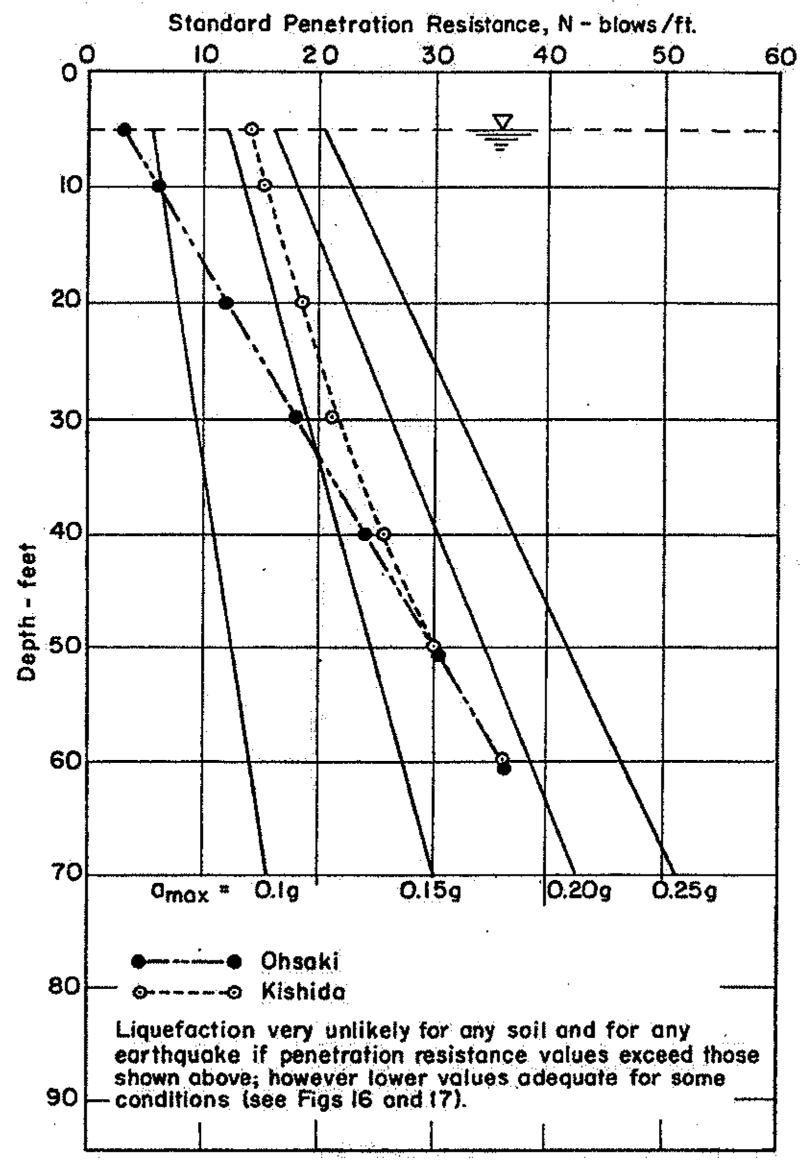


Figure 35.—Penetration resistance values for which liquefaction is very unlikely to occur under any conditions (H. Seed and Idriss, 1970).

The following year (August 1971), Seed and Peacock published a follow-up paper, which took the methodology a step further. The cyclic stress ratio was developed as a representation of the earthquake loading and other conditions (depth to water table and depth of liquefiable material). This triggering load was compared to relative density. Figure 36 shows the results.

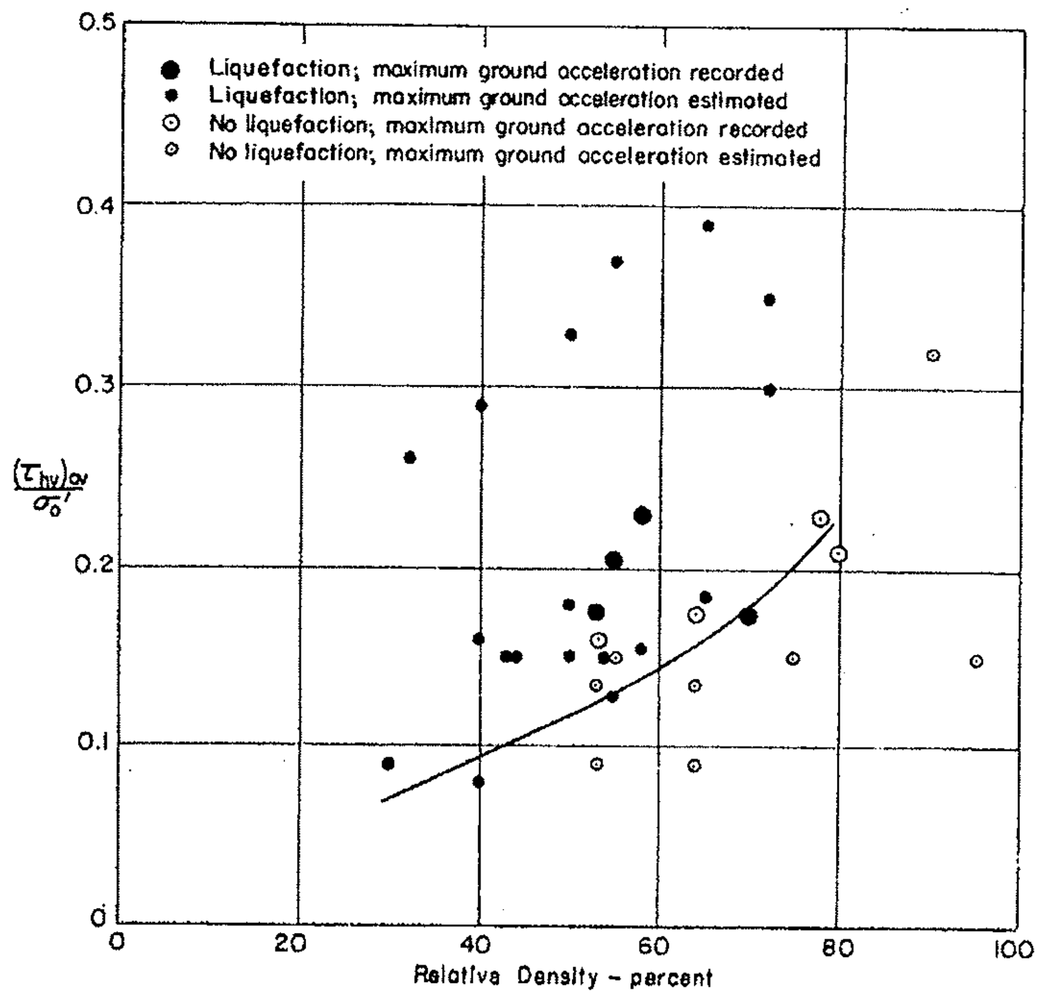


Figure 36.—Relationship between CSR and relative density for known cases of liquefaction and nonliquefaction (H. Seed and Peacock, 1971).

Relationship Between Relative Density and Relative (or Percent) Compaction

Some researchers have related liquefaction potential to "relative compaction" or "percent compaction," instead of to relative density. This is sometimes done to avoid some of the shortcomings of the relative density test, particularly the difficulties in measuring minimum density. Other times, it is because the material contains appreciable fines and thus is not particularly applicable to relative density testing. In essence, relative compaction is simply the ratio of the in-place density of the soil to the maximum density measured in the laboratory corresponding to a specified method. The maximum laboratory density might be obtained from a vibratory table, Proctor testing, or other methods.

In a 1971 technical note, Lee and Singh describe the defining equations for relative density and relative compaction, and then relate the two mathematically, as follows:

$RC = RO/(1 - D_r(1 - RO))$
Where RC is relative compaction, D_r is relative density, and
$RO = \gamma d_{min}/\gamma d_{max}$

The authors reviewed data for 47 different soils ranging from fine silty sand to coarse sand, and from uniform to well graded. They found that the average RO was 81.8 percent, with a standard deviation of 7.7 percent. This was judged to be a value with relatively small uncertainty, thus providing a reliable value to substitute into the above equation.

This equation for comparing relative density and relative compaction can therefore be approximated by the simple equation:

$$RC = 80 + 0.2\,(D_r)$$

where D_r is expressed as a percentage. For example, by this equation, a relative density of 50 percent would correspond to an approximate relative compaction of 90 percent.

Caution is required in using this correlation, according to more recent work by R. Seed and others at the University of California at Berkeley. They have found that this equation does not work well with the maximum dry density determined by a direct impact compaction test in cases where that test does not agree well with the vibratory procedure. The difference is, in part, that the vibratory test does not break down particles, which would alter the soil's gradation in such a way that the maximum density would be increased (as the smaller pieces could fit among the larger particles). For this reason, direct-impact compaction is disallowed for maximum density tests for relative density determination. In contrast, relative (or percent) compaction generally is based on impact compaction tests. Berkeley researchers have found that the approximation for relative compaction shown above is *generally* valid as long as is it is used with the Modified American Association of State Highway and Transporation Officials (ASTM D-1557) Compaction Test. When using Standard Proctor compaction values (ASTM D-698 or the similar USBR 5500), the results may be misleading because the maximum density so determined is lower than that from the vibratory test for soils where either can be used.

Reclamation has had some similar experiences which suggest this relationship does not always agree well with actual compaction data. In particular, the relationship should be used with caution when the fines content approaches 15 to 20 percent. This appears to result from the difficulties in determining minimum densities when then fines contents are nonnegligible. Berkeley researchers have found that this relationship may become variable as fines contents as low as 10 percent.

Case Histories Comparing Density to *In Situ* Techniques

The major focus of this study was to search for and review available data and published papers presenting results from liquefaction evaluations using *in situ* techniques. One key observation from this study is that there is not a great deal of in-place density data available in the literature. In large part, this is probably attributable to the difficulties in obtaining such data, particularly since the liquefiable soil deposits are saturated and located at some depth below ground surface. Secondly, as already mentioned, it is very time consuming and expensive to obtain sufficient numbers of representative in-place densities to properly characterize a soil unit for liquefaction evaluation.

Following are discussions of each project located in which some form of in-place density data was available for a liquefaction evaluation where *in situ* techniques were also used. Most of these discussions are relatively brief, due to limited information from the available papers or reports, while a few projects include a more detailed discussion due to more in-depth evaluation in the source reference. At the end of this section, the project data are summarized in tabular form.

Rock Debris Avalanche Gravel Liquefaction—*Case Study of Rock Debris Avalanche Gravel Liquefied during 1993 Hokkaido-Nansei-Oki Earthquake* (Kokusho *et al.*, 1995)

This paper describes a thoroughly investigated gravel deposit in Hokkaido, in northern Japan, that was shown to have liquefied during the subject earthquake. This gravel layer, believed to have resulted from a rock avalanche, is angular andesitic material consisting of approximately 80 percent gravel and only minor fines. The site was investigated with the Japanese large penetrometer test (LPT), which resembles the SPT test but with larger sampler and hammer, shear-wave velocity, and in-place densities from frozen samples, as well as comparing the data to SPT values determined by other researchers. (LPT testing is discussed briefly under *Large Penetration Test (LPT)* on p. 108.)

Shear-wave velocities were measured from the surface (by refraction surveys) and downhole. The upper 12 feet (not including 3 feet of overlying ash material) showed low velocities from both methods, on the order of 300 to 400 ft/s. (These values are not normalized for overburden stress.)

SPT values in the gravels were obviously influenced by the particle size and therefore considered to not be representative. Therefore, two LPT holes were located in the gravels, with blow counts converted to equivalent N_{60} values. In the upper portion of the gravel deposit, LPT blow counts were 4 to 8, which implies equivalent SPT blow counts of 8 to 16. The LPT profiles also correlated well with downhole shear-wave velocity trends.

Five relative densities were determined from frozen samples in the upper portion of the gravels; these values ranged from 12 to 27 percent relative density. The shear-wave velocities were quite low, consistent with the low measured relative densities. However, by Wu's correlation (Wu, R. Seed, and Pestana, 2003) in figure 33 above, one would expect lower equivalent SPT values (estimated from the LPT). All three methods indicated the gravels to be loose and subject to liquefaction during earthquake loading.

Jackson Lake Dam—*Measurement of In-Place Relative Density in Coarse Grained Alluvium for Comparison to Penetration Tests* (Farrar, December 1999)

This case history deals with the evaluation of liquefaction potential at Reclamation's Jackson Lake Dam in northwestern Wyoming. In-place densities were obtained in the hydraulic fill embankment by using 8-inch diameter sand cones in 4-foot diameter shafts. The soils were typically 3-inch minus, poorly graded to silty gravels (GP-GM) with an average gravel content of around 60 percent. From 10 of these tests, 7 ranged from about 48 to 68 percent relative density. One test gave about 92 percent; two tests gave relative densities below 0, apparently due to combining layers of different soils.

Nine SPT tests in the hydraulic fill showed uncorrected blow counts ranging from about 4 to 35, and 7 of the 9 were between 4 and 15. Plotting the SPT values on the Gibbs and Holtz relative density chart indicated the 7 tests had relative densities ranging from less than 40 percent to slightly more than 60 percent. The measured and SPT-inferred relative densities and densities were in reasonably good agreement, with some suggestion that the measured insitu relative densities were slightly higher than those predicted by the SPT.

Bradbury Dam—*Measurement of In-Place Relative Density in Coarse Grained Alluvium for Comparison to Penetration Tests* (Farrar, December 1999)

Reclamation's Bradbury Dam near Santa Barbara, California was modified because a number of different *in situ* tests (including SPT, BPT, and shear-wave velocity) indicated the downstream alluvium had the potential to liquefy during a large earthquake. Consequently, an excavate-and-replace modification was designed and constructed. This site provided a unique opportunity to compare in-place relative density data to penetration resistance data, since densities were measured in the previously tested foundation soils as the downstream alluvium was being excavated. A total of 14 in-place density tests (with a 20-inch diameter sand cone) were taken at 10 locations, all near the site of SPT or BPT borings. A number of different materials were tested, including a silt layer, silty sands, and poorly to well-graded gravels and sands. All but one soil consisted of minus 3-inch materials. Data are shown in table 5.

Table 5.—Summary of Bradbury Dam Data

Sample No.	Soil class.	% gravel	% sand	% fines	SPT N_{60}	BPT N_{60}	SPT RD	BPT RD	In-place RD
1A	SM	0	62	38	53	15	92	52	
1B	(SP)g	29	70	1	33	15	75	52	33
1C	SM	0	83	17	16	22	53	64	73
2	SM	4	82	14	6-14	22-31	<40	63-71	85
3	(SW)g	46	52	2	33-36	23-25	67-69	60-62	58
4	(GW)s	64	34	2	54	34-39	89	73-77	75
5	(GP)s	54	42	4	32-36	14-16	69-73	45-50	44
8A	SP	0	98	2	22-23		57-60		63
9	(GW)s	74	23	3		21-22		57-58	35
10A	(GW)sc	76	22	2		29		73	19
10B	(SP)g	26	71	3		31		76	74
10C	(GW)sc	61	36	3		35		80	33

The relative densities from the SPT and BPT tests shown in the above table were determined using the Gibbs and Holtz method of relating blow counts to density. Other correlations may show different results, but this was the approach preferred by the author of the referenced report. (Wu's correlation predicted relative densities that were generally 2 to 5 percent lower than shown in the "SPT RD" column—very close agreement.)

Additional observations made in the report include:

- The gravelly soils were described as loose and easily excavated by hand.
- Four of five tests in the gravels had measured relative densities less than 50%.
- The two silty sand samples had some of the highest measured in-place relative densities.
- Sand and gravel mixtures had in-place relative densities between those of the silty sands and the gravels.
- The BPT closely matched in-place relative density near sample numbers 3, 4, 5, and 10B.

- The corresponding SPT values at these locations were higher, likely due to gravel interference.

- The SPT matched the in-place relative density well at only one location.

- The higher in-place relative densities predicted by the BPT at locations 10A and 10C are likely due to the fact than particles larger than 3 inches existed and caused interference with the 6.7-inch penetrometer.

It is obvious that a number of different soils were sampled at this site, and relatively few data are available to draw strong conclusions. At least on the surface, it is not apparent that strong correlations exist between the three different methods of estimating the relative density. However, it is likely that not all in-place determinations of relative density are fully accurate, and it is further likely that the presence of significant gravels and some cobbles make both the SPT and BPT subject to some errors. The data do indicate that BPT and SPT tests tend to overpredict relative density when gravel is present. Where the BPT and relative density agree, Farrar reports that the particles were smaller than 2 to 3 inches, which suggests when larger particle sizes are involved, liquefaction evaluation becomes more problematic.

Mormon Island Auxiliary Dam—*Seismic Stability Evaluation of Folsom Dam and Reservoir Project—Report 4—Mormon Island Auxiliary Dam—Phase I* (Hynes *et al.*, March 1988)

In this U.S. Army Corps of Engineers (USACE) evaluation of Mormon Island Auxiliary Dam (near Folsom, California) in the 1980s, the collected data included in-place densities, BPTs, and shear-wave velocity measurements in the downstream foundation comprised of dredged tailings. The dredged tailings were about 50 feet in depth and consisted of loose gravels. Based on the USACE testing, the average gradation of the tailings was about 10 percent cobbles, 80 percent gravels, 7 percent sand, and 3 percent fines. In-place density tests (total of 12 tests) were taken in the upper 7 feet of the tailings beyond the downstream toe of the dam. Shear-wave velocities (3 separate crosshole locations—1 beneath shell and 2 at toe) and BPTs (1 hole beneath slope and 2 at toe—sampling holes not included) were measured in the tailings about midway down the downstream slope and beyond the downstream toe. Table 6 is a summary of these data.

Table 6.—Summary of Mormon Island data

Location of tailings	Depth of tailings (ft)	Relative density (%)	Shear-wave velocity (not normalized) (ft/s)	BPT (equivalent $(N_1)_{60}$
Beneath downstream shell	45 to 95	average 35 (range of 15 to 50)	average 625	10 to 17
Downstream toe of dam	0 to 50	not measured	400 to 525 in upper 35 feet, 900 below	6 to 10

As can be seen from the data, there is good agreement among all three methods that the tailings are loose, with low relative density, low shear-wave velocities, and low blow counts.

The USACE calculated cyclic stress ratios in the tailings of around 0.085 to 0.13 in the downstream toe area, and from 0.13 to 0.19 beneath the downstream shell. Using the latest SPT curves (from NCEER and Seed *et al.*, 2003), liquefaction would be predicted by the BPT blow counts that were converted into $(N_1)_{60}$ values for both areas of tailings. Likewise, after normalizing the shear-wave velocities and using the NCEER shear-wave curve, both areas of tailings would also classify as liquefiable using that method. Thus, for these data, it appears that there is good correlation among relative density, BPT, and shear-wave velocity methods for predicting liquefaction potential.

Re-Evaluation of Lower San Fernando Dam

Re-Evaluation of the Lower San Fernando Dam—Report 1—An Investigation of the February 9, 1971 Slide *(Castro* et al.*, September 1989)*

The slide of the upstream slope of Lower San Fernando Dam is one of the best-documented cases of earthquake-induced instability. The hydraulic-fill portion of the embankment was liquefied, and a portion moved a very large distance upstream. This left only about 3 feet of freeboard on the remnant embankment.

Among numerous other studies, Castro and fellow GEI Consultants (GEI) investigators conducted a number of explorations on the downstream portion of the reconstructed dam (located northwest of Los Angeles, California) to determine the properties of the hydraulic fill portion of the embankment, which was believed to be reasonably representative of the upstream portion that failed. The 1985 exploration program included 6 SPT borings, 12 CPT soundings, undisturbed sample borings adjacent to 5 selected SPT/CPT locations, and a 6-foot diameter, 85-foot deep exploration shaft located adjacent to an SPT/CPT location for mapping, sampling, and performing in-place density tests.

Based on their investigations, they determined the critical layer (both in terms of exploration data plus re-evaluation of slide information) was a layer in the hydraulic fill shell approximately from elevations 1005 to 1020. A batch sample representing this layer had about 50 percent fines and was classified as SM-ML. The groundwater elevation, based on two observation wells installed, was somewhere between elevations 1011 and 1016. The representative $(N_1)_{60}$ blow count for this layer was judged to be about 11 to 13. Five in-place density tests were done in this layer, with the following results: 98.6, 96.7, 98.1, 95.8, and 100.7 lb/ft^3. No relative density tests were run, as the material was too fine-grained for the vibrated maximum density test or the minimum density test. The Proctor maximum density was approximately 116 lb/ft^3. The five samples in this layer therefore had compactions ranging from 83 to 87 percent. The SPT blow counts are in qualitative agreement with the in-place density tests, as both indicate low density.

Re-Evaluation of the Lower San Fernando Dam—Report 2—Examination of the Post-Earthquake Slide of February 9, 1971 *(Seed* et al.*, September 1989)*

This paper by Seed *et al.* also looks at the Lower San Fernando slide based on 1985 explorations. An interesting observation from this paper was that the range between the minimum and maximum dry densities in the silty sands, based on the original 1971 work, was approximately 25 lb/ft^3. (These soils apparently had lower fines contents than the material tested by Castro, Keller, and Boynton (1989), as the minimum-density test could be performed.) Assuming the maximum density is the 116-lb/ft^3 value determined by the GEI testing, that would make the minimum density about 91 lb/ft^3. Based on these values, the average relative density for the five GEI values in the critical layer ranges from 24 to 46 percent. Thus, the empirical method (SPT blow counts) and the measured low relative densities correlate well and would likely lead to the same conclusion that the hydraulic fill materials would have liquefied.

***Pinopolis West Dam, Santee-Cooper Project—Seismic Studies—Pinopolis West Dam—Santee Cooper Project* (May 1985)**

This consulting report by Castro and GEI colleagues presents an evaluation of the subject dam based primarily on steady-state strengths. However, like the GEI reevaluation of Lower San Fernando Dam (Castro, Keller, and Boynton, 1989), there are some interesting data comparing SPT blow counts to in-place densities.

Pinopolis West Dam, part of the Santee Cooper Project, is a homogeneous embankment located approximately 30 miles northwest of Charleston, South Carolina. The problem soils beneath the dam were stratified layers of sand within about a 10- to 30-foot thick layer immediately under the embankment. These soils consisted of fine sands and fine to medium sands, typically containing from 7 to 17 percent clayey or silty fines. GEI conducted SPT tests, as well as determining in-place densities from undisturbed sampling holes and sand cone densities conducted in two 6-foot diameter cased shafts.

The layer best represented by the data from this study was a silty/clayey sand located 12 to 14 feet below ground at the downstream toe. Data from 6 SPT tests (in three holes) indicated uncorrected blow counts ranging from 1 to 7. Five in-place density tests taken in the 6-foot test shaft ranged from 95.9 to 97.5 lb/ft^3.

Proctor maximum densities determined for two samples were 117.2 and 123.6 lb/ft^3, which indicated 78 and 81 percent compaction in the foundation, respectively. The vibratory maximum and the minimum densities (as used in calculating relative density) were also determined, with resulting values of 101 and about 80 lb/ft^3. However, the data were a little confusing, as there were large differences in densities determined by dry and wet vibratory methods, and those tests are not reliable in clayey material. With materials suited to both the Proctor and the vibrated maximum density tests, the difference is generally not as large as 101 to 117 lb/ft^3.

Assuming that the vibrated maximum density used for determining relative density is equal to the 117 lb/ft^3 found in the Proctor test, and that the minimum density is 25 lb/ft^3 lower, similar to what was found by Seed *et al.* (1989) for Lower San Fernando materials, one would estimate a minimum of 92 lb/ft^3. This implies that the measured density corresponds to a relative density of roughly 20 percent. According to the Gibbs and Holtz relationship, the blow counts of 1 to 7 would indicate the relative density would be between 0 and 50 percent. Thus, there is qualitative consistency between in-place relative density and the SPT blow counts. It should be noted, however, that using relative density for a soil with 7 to 17 percent fines is not ideal, particularly if the fines are clayey; the relative density test is most suited to cohesionless soils with less than 5 percent fines.

Keenleyside Dam—*Liquefaction Assessment of Gravelly Soils for Dam Safety Evaluation* (Yan and Lum, June 2003)

Yan and Lum of British Columbia Hydropower (BC Hydro) discuss the difficulties of evaluating liquefaction potential in gravelly soils, as well as proposing methods for doing so. In addition, they provide a good summary of the BC Hydro evaluation of liquefaction potential of barge-dumped fill incorporated into Keenleyside Dam, located on the Columbia River about 50 miles north of the Canada-United States border. This material is well-graded, clean sand and gravel, with cobbles and boulders up to 18 inches in size. The gravel content is typically 45 to 80 percent, and the fines content is less than 5 percent.

As described earlier, BC Hydro sometimes accounts for gravel influence in SPT tests by recording SPT blow counts for every 1 or 2 inches of penetration. The equivalent SPT blow count can be considered the lowest 1- or 2-inch value, multiplied by either 12 or 6. When this procedure was applied to Keenleyside Dam, the adjusted blow counts were naturally less than the blow counts for the full 12-inch test sum. However, the difference was less when the summary, or full, blow count was low, suggesting that lower blow counts are less affected by the gravel. The authors suggest that the equivalent blow count method based on penetration for small increments better characterizes the penetration resistance of the finer-grained matrix material within gravelly soils.

BC Hydro conducted a number of BPT tests at Keenleyside Dam, and reduced the data using bounce-chamber pressures and the Harder and Seed method, as well as the energy approach developed by Sy and Campanella. These two approaches gave similar results, so the Harder and Seed method was used for most of the BPT data because it was simpler and less costly.

Comparison of the SPT and BPT data from Keenleyside Dam led to the following observations:

- Both SPT and BPT indicated a large variation in penetration values for the gravel layer in question.
- The lower bound value for SPT $(N_1)_{60}$ was about 10.
- The 30th percentile and median BPT $(N_1)_{60}$ values were about 8 and 10, respectively.
- From this, the authors concluded the agreement between methods was good.
- These values indicated that liquefaction was likely for the design earthquake.

At Keenleyside Dam, BC Hydro also conducted three different shear-wave velocity measurements, using downhole, cross hole, and SASW methods. In general, the authors noted good agreement among the three methods, except that the SASW method gave what appeared to be unreliable results near the crest and downstream slope. In these occasional areas, it appears that boundary effects may have been created problems with the data. When normalized, the measured shear-wave velocity of the questionable gravel layer was about 200 m/s, which is right on the boundary separating liquefaction and nonliquefaction.

Finally, in-place density testing was performed. Samples of the gravelly material were excavated from the borrow area and tested for maximum and minimum laboratory densities (151.6 and 124.2 lb/ft^3, respectively). In-place density tests with 2- and 3-foot steel rings were taken in the gravels about groundwater level and in the upper 6 feet of the deposit. The resulting in-place density averaged about 45 percent relative density or 89 percent relative compaction. This value is in good agreement with an estimated representative blow count of 8 to 10.

Pineview Dam (Block, 2000; Reclamation, 2001a; Shaffner, 1999; Sneddon, 2001; Reclamation, 2001b)

Reclamation's Pineview Dam located near Ogden, Utah is a site with a variety of information dealing with the liquefaction potential of foundation deposits. Collected information includes SPT, BPT, LPT, and shear-wave velocity measurement, as well as in-place densities taken during modification construction to mitigate the liquefaction potential. (The LPT is identical to the SPT in concept, but uses a larger sampler and a larger hammer. There exist at least two LPT "standards," one from Japan and one used primarily in the US, which uses the so-called Dames and Moore sampler.)

The foundation geology at Pineview is extremely complex, and the soils are highly variable, with materials from a variety of very different depositional environments. This results in part from the confluence of the high-energy Wheeler Creek with the Odgen River just downstream of the dam, and the inundation of the area by lakes many times in recent geologic history, including the well documented, prehistoric Lake Bonneville. Fluctuations in lake levels over several thousand years resulted in deposition of materials ranging from gravelly sand to varved silts and clays. The complicated foundation geology results from the transgression and regression of the various small lakes in combination with the simultaneous and subsequent fluvial processes. These difficult geologic conditions make correlation and compilation of soil properties exceedingly difficult, with major differences in soils properties showing up in exploration holes located just 10 feet apart.

In general, the *in situ* investigation techniques did, in combination, point out two foundation units that appeared to have the lowest density and be susceptible to liquefaction under the design seismic loadings. These units were Qbs1B and Qbs1C, both primarily silty sand units (with some gravel in places).

Following are some general conclusions/observations gained through a review of available exploration data and studies for Pineview Dam:

- Use of the LPT was somewhat helpful in identifying materials (because of its ability to sample larger material), but the complicated geology prevented any meaningful correlation with nearby SPT holes. Also, the LPTs and SPTs were alternated in each drill hole, rather than being done at the same elevation in side-by-side holes. Without the ability to generate an SPT-LPT correlation, the LPT was of limited use in ascertaining liquefaction potential at this site. The LPT used here was not the Japanese LPT, for which correlations with SPT have been established.

- Using the SPT to evaluate liquefaction potential required a detailed look at the sand portions of the SPT tests, since gravel tended to mask the overall picture. ("N Sand Equivalent" was used, which essentially looked at the slope of the penetration plot for each 1-foot test to determine if a mix of materials was evident, as described under *Data Collection and Reduction* on page 10.) This technique provides a similar end result to the BC Hydro sampling described in the previous case history.

- In general, there was agreement between the shear-wave velocity measurements and SPT $(N_1)_{60}$ values as far as indicating the presence of potentially liquefiable foundation soils. However, due to the difficult drilling conditions, including high water losses in gravelly layers, there were problems with drilling and establishing the grouted casing for shear-wave testing, which led to some uncertainties in those measurements.

- Only limited BPT explorations were conducted at this site. However, based on the available data from nine BPT holes and six companion SPT holes, there was a reasonably good correlation between equivalent $(N_1)_{60}$ values between the two methods. Obviously, due to the extreme variability of the foundation soils, there was scatter in the data. However, both the SPT and BPT compared favorably in identifying both softer areas and similar $(N_1)_{60}$ blow count values at similar elevations.

- BPT data were reduced using both the Harder and Seed correlation and the Sy and Campanella correlation. Although both methods produced similar trends, there was considerable variation in the resulting $(N_1)_{60}$ values from each method. In some areas, one method gave significantly higher $(N_1)_{60}$ values, while in other areas the other method would do so. The graph shown in figure 37, reproduced from the Pineview study, shows some of the scatter, but also shows an overall strong relationship between the two data reduction methods. Note that for very low blow counts, say less than 10, the Harder and Seed correlation predicted higher $(N_1)_{60}$ values. However, for blow counts greater than about 20, the Sy and Campanella correlation typically produced higher $(N_1)_{60}$ values. Note that in a blow count range often of interest in liquefaction evaluation, $(N_1)_{60}$ ranging from 10 to 20, the methods appear to provide very similar results, at least at this site.

Another interesting aspect of data collection at Pineview Dam came during the construction, essentially an excavation of a shear key through the weak material near the downstream toe, and replacing it with dense, compacted fill. It is critical to note that nearly all of Pineview's careful, detailed testing occurred near the toe of the dam and berm, for the purpose of establishing the need for a modification. However, the excavated shear key is downstream of the berm, and only one of the previous drill holes was located within the footprint of the excavation. Therefore, the foundation materials that were exposed in the excavation were not necessarily the same as those tested further upstream. For the modification design, the units of concern were projected through this area based on the limited data. Mapping of geologic units during excavation showed fairly good correlation with the projections.

A few in-place density tests were performed during excavation. Of these tests, only one density test was performed in unit Qbs1B, and this test was outside the excavation in an area where construction traffic likely disturbed the material. One in-place density test was performed in the sandy unit Qbs1C, but this test was difficult to correlate with SPT data since blow counts in nearby drill holes ranged from N=11 to N=35. It was difficult to correlate between these two density tests performed hundreds of feet downstream of dozens of most of the SPT data and develop any meaningful conclusions.

Deer Creek Dam

This Reclamation dam on the Provo River upstream from Provo, Utah featured a comprehensive liquefaction evaluation that ultimately led to a modification to

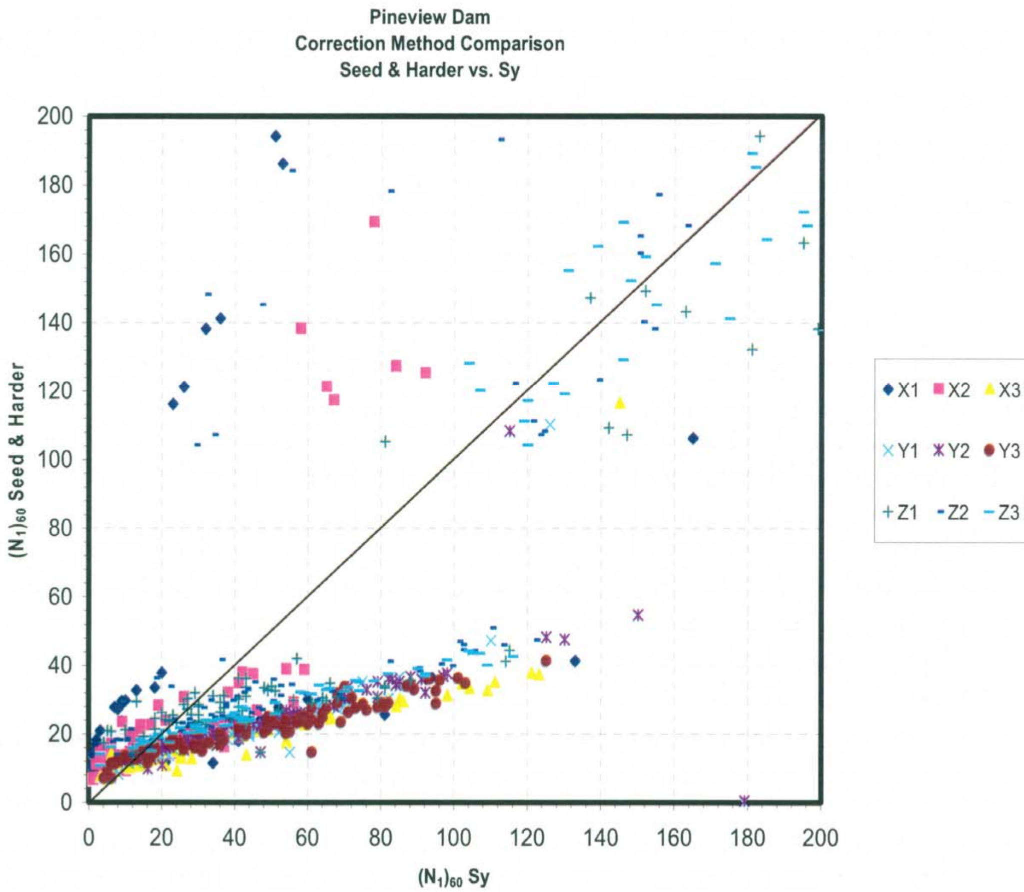


Figure 37.—Comparison of Seed-Harder and Sy-Campanella correlations.

mitigate the potential for foundation liquefaction. Like Pineview Dam, several investigation methods were utilized to evaluate liquefaction potential, and in-place density testing was conducted during the modification construction.

Following is a summary of Reclamation conclusions regarding the correlation of in-place density data with preconstruction investigation data at Deer Creek Dam.

- Of the 20 in-place density tests taken in the foundation during excavation of the downstream cutoff trench, seven were in the Qalg2 layer of the foundation, at which excavation was terminated. These tests were taken primarily to confirm the higher densities anticipated in this material based on the preexcavation BPT explorations. Although one sand-cone density test indicated a percent compaction of 88.9 percent, the other six densities ranged from 95.3 to 98.5 percent. (Most of these were measured more appropriately with a ring apparatus, which is better able to test material with large gravel or cobbles.) This generally supported the expectations from the preexcavation explorations.

- Six in-place density tests were taken in the Qaf. This deposit is highly variable, and a definitive conclusion based on preexcavation explorations had not been reached. The principal objective of the in-place densities was to provide information for evaluating the need for further remediation in areas not originally included in the phase I excavation.

- Of the seven remaining tests, three were in the Qalg2, which had previously been identified by the field explorations as apparently having low densities, the removal of which was a major objective of the excavation.

- The remaining four tests were equally divided between the Qal and Qalg1s, both generally anticipated to have low densities, based on the preexcavation explorations.

- Only the in-place densities from the seven tests in layers anticipated to have low densities (Qalg2, Qal, and Qalg1s) are directly applicable for evaluating whether the BPT explorations accurately predict the actual *in-situ* density of the deposits.

- In looking at the densities obtained in the Qalg2, the maximum density for sample D01 obtained using Proctor compaction was only 82.4 lb/ft^3. The optimum moisture content was 34.4 percent, higher than the liquid limit (LL) of 33.8 percent and significantly higher than the plastic limit of 27.3 percent; either of these conditions is sufficient to cast serious doubt on the credibility of the maximum density results. The remaining two tests in the Qalg2 used the vibratory method to obtain maximum densities. The percent compaction values were 88.1 and 93.0 percent. Minimum densities were not provided but it is probably reasonable to estimate that these percents compaction correspond to relative densities of about 45 percent and 60 percent, respectively. However, given the inherent difficulties of obtaining consistent densities using the vibratory method and the general variation in obtaining *in-situ* densities (2 to 3 lb/ft^3), the actual relative densities could easily be as low as 35 and 50 percent, respectively. This illustrates the difficulty in drawing definitive conclusions from a small number of samples. In either case, it would be difficult to refute the validity of the preexploration investigations interpretations on the basis of these two tests.

- The two tests in the Qal also have very low maximum densities and associated optimum moisture contents that are quite high when compared to the LL and plastic limit (PL). This casts serious doubt that the percent compaction determined from these tests is representative of the actual deposit.

- Of the two samples taken in the Qalg1s, sample D12 was taken to the left of station 8+00 in an area of higher BPT values. The preexcavation explorations had indicated this area to be significantly denser than the same deposit to the right of station 8+00. Thus, only sample D14 appears to represent the portion of the Qalg1s that was expected to have lower *in-situ* densities. This sample also has a somewhat lower than expected maximum density. It is, however, much closer to what would be expected than were the maximum densities mentioned above for Qals and Qalg2 that were so low as not to be credible.

Of the 20 *in-situ* density tests taken at Deer Creek Dam in the excavation for the key trench, only two samples in the Qalg2 and one sample in the Qalg1s were considered to be directly usable for comparison to preexcavation explorations used to identify potential low density areas of the foundation. The two samples in the Qalg2 indicated the estimated relative density in the deposit could range from 60 to 45 percent, values that do not necessarily preclude liquefaction or significant strength loss when subjected to strong cyclic loading. Further, due to the inherent variability in the test methods the actual relative densities for these samples could be even lower.

Summary of In-Place Density Comparisons

The findings from these nine case histories are summarized in table 7 to show comparisons between in-place relative density or percent of maximum density, and indirect indications of density from *in-situ* measurements used in liquefaction investigations. The degree to which measured densities matched with the respective *in-situ* measurements (such as low density to low penetration resistance) is judged subjectively using verbal descriptors of excellent, good, fair, and poor. (If left blank, there were no comparison data from the project.) It is recognized that another reviewer might form different judgments about the degree of agreement between methods.

The various *in-situ* investigation techniques generally compare well to measured in-place density or relative density measurements where sufficient data exist to draw comparisons. With only one noticeable exception (Bradbury Dam), investigators report good correlation. Although a limited number of cases is shown, the data support the industry's use of *in-situ* investigation methods and suggest that the current state of practice in using these methods to characterize liquefaction potential is both sound and reasonably reliable, in spite of some limits and uncertainties in the methods.

Table 7.—Comparison of relative density to *in situ* measurements of liquefaction potential

Project	Comparison of Relative Density to SPT	Comparison of Relative Density to BPT	Comparison of Relative Density to LPT	Comparison of Relative Density to V_s
Avalanche Gravel			Good	Good
Jackson Lake Dam	Good			
Bradbury Dam	Poor	Fair		
Mormon Island Dam		Good		Good
Lower San Fernando Dam	Good			
Pinopolis West Dam	Good			
Keenleyside Dam	Good	Good		Fair
Pineview Dam	Insufficient data	Insufficient data	Insufficient data	Insufficient data
Deer Creek Dam	Insufficient data	Insufficient data	Insufficient data	Insufficient data

Correlations Among Various *In Situ* Techniques

Whereas the preceding section of this document reported the ability of the various *in-situ* investigations to correlate with measured in-place or relative densities at a site, this section discusses how well the various techniques correlate among each other relative to predicting liquefaction potential. Many of the case histories in the preceding section, in addition to reporting correlations with density, also reported correlations among methods. Judging from the published case histories, it appears that typically more than one technique is utilized at a site to help evaluate liquefaction potential. Not a single paper that was reviewed in the course of this study included a site where only one technique was used to measure the soil's liquefaction potential. This is viewed as very positive, and shows that the state of the practice includes use of more than one means of analyzing liquefaction potential.

Case Histories Comparing Results from the Various *In-Situ* Techniques

In addition to most of the cases previously described under *Laboratory Tests and Past Evaluations of Liquefaction Utilizing Density Measurement* on page 69, the following published accounts of liquefaction evaluations were also reviewed. As was the case for the previous section, many of these discussions are relatively brief, due to limited information from the available papers or reports, while a few projects include a more detailed discussion due to a more in-depth evaluation in the source reference. At the end of this section, the relevant project data from both this section and *Case Histories Comparing Density to In Situ Techniques* on page 76 are summarized in tabular form to show the comparison among the various *in-situ* investigation techniques.

Industrial Site in China—*Analysis of Liquefaction Potential by* In Situ *Testing* (Wong, 1986)

This paper describes the evaluation of liquefaction potential at an industrial site in east central China. There was no mention of previous occurrences of liquefaction in the area. The design earthquake was judged to have a surface peak acceleration of 0.18g. The foundation consisted of three distinct layers of potentially liquefiable materials: clayey sandy silt, silty fine sand, and fine sand. There did not appear to be any appreciable oversize materials, and most materials were fine sand size or smaller. Three separate tools were used to characterize liquefaction potential: SPTs, CPTs, and shear-wave velocity measurements. All three methods predicted that the soils would not liquefy under the design loading, with reasonably similar safety factors against liquefaction (around 2 or greater).

Calaveras Dam—*Recent Advances in Soil Liquefaction Engineering: A Unified and Consistent Framework* (Seed *et al.*, April 2003)

In this paper by Seed, Cetin, and others, the authors discuss a liquefaction evaluation at Calaveras Dam in the Bay Area of California, which was investigated by both BPT and short-interval SPT. Soils at the site were highly variable, with fines contents ranging from low to high, and gravel contents ranging from a few percent to over 50 percent. BPTs were driven in 10-foot continuous lengths, halted and withdrawn 5 feet, and then redriven 5 feet before starting the next 10-foot length. Whereas the first foot of redriving was considered as "reseating" of the penetrometer, driving resistance for the second foot of redriving was assumed to be almost entirely from casing friction. Casing friction was found to typically provide between 5 and 45 percent of the total BPT resistance, with an average value of around 19 percent. The casing friction from this second foot was used to adjust (reduce) the total driving resistance, but it is not clear how that was done. Although not stated in the paper, it is surmised that the Harder and Seed correlation was used to convert the "corrected" BPT blow counts to equivalent $(N_1)_{60}$ values.

The data accumulated at this site are shown in table 8. The authors concluded there was "a generally good level of agreement between the results of the short-interval SPT and the corrected BPT data, suggesting that these two methods can both be used in variable soils of these types with some reliability." While the two tests agree remarkably well in some zones, the differences are quite substantial in others (ratios as high as 2). There is no consistent trend in the differences; in some units, the SPT results were higher, and in others the equivalent SPT blow counts estimated from the BPT were higher. In several of the units where agreement was not good, the number of comparison data was small. Even though the design earthquake at this site is reportedly quite large, it is not entirely clear from table 8 whether liquefaction would have been consistently predicted between the SPT and BPT data.

Table 8.—Representative blow counts, Calaveras Dam

Zone	Zone description	Sub-zone	30th percentile $(N_1)_{60}$ SPT	30th percentile $(N_1)_{60}$ BPT	50th percentile $(N_1)_{60}$ SPT	50th percentile $(N_1)_{60}$ BPT	Repre-sentative fines content	ΔN_{CS} (for fines)
I	Rock berm(placed in the 1970s)		N/D	22	N/D	29	15(F)	N/A
II	Dumped Weathered Rock fill	II(M)	17	19(B)	21	23	14	1.5
		II(TD)	9	8	12	8	7	1
		II(US)	23	21	22	20	10	1
III	Cobbly gravel fill		N/D	7	N/D	8	20(F)	1.5
IV	Rolled fill		17	23(L)	22(L)	25	48	N/A
V	Mixed Dumped and Sedimented Hydraulic fill		13	19	16	23	20	1.5
			12	17	17	23	15(F)	1.5
			17	17	20	22	19	1.5
V(R)	Mixed Hydraulic and Rolled fill		21	14(L)	24	18	15(F)	1.5
VI	Disturbed and Mixed Hydraulic, Dumped, and Rock fill	VI	10	N/D	17	N/D	11	1
		VI(F)	11	22(L)	18	36(L)	59	N/A
		VI(G)-Res	7	N/D	8	N/D	11	1
		VI(G)-Emb	27	22	40	31	11	1
		VI(R)	12(L)	N/D	12(L)	N/D	15	1.5
VII	Sedimented hydraulic fill		10	N/D	13	N/D	62	N/A
VIII	Base alluvium		19	20	30	26	8	1
X	Mixed fill		12	17	13	26	19(F)	1
XI	Rocky colluvium		32	36	34	43	N/D	0

(L): Limited penetration data available
(B): Based on data at bottom of zone
(F): Calibrated field-estimated fines contents were also considered
N/A: Not applicable (high CL content)
N/D: Not determined

East Dam and East Dike Extension, Santee-Cooper Project—*Phase IV Pilot Testing Program and Supplemental Subsurface Investigation—East Dam and East Dike Extension—Santee Cooper Project* (Paul C. Rizzo Associates, December 1995)

The emphasis of this study by Paul C. Rizzo Associates was to test whether stone columns would be an effective means to treat the liquefaction potential beneath the East Dam and Dike, located near Charleston, South Carolina. The soils tested comprised about a 22-foot-thick zone (from a depth of 8 to 30 feet below ground surface) underlying a stiff silt/clay layer. The critical materials included some clean sands, a silt layer or two, and mostly silty and clayey sands (with varying fines contents from 5 to 45 percent). SPTs, CPTs, and shear-wave velocity measurements were conducted both before and after installation of stone columns. The shear-wave velocity was apparently measured through the use of a seismic cone, but no details were provided. Test results are presented in table 9.

Table 9.—Summary of data for East Dam and East Dike

Test Location	Depth (feet)	Soil type	N range (blow count)	Q range (ton/ft^2)	V_s (not normalized) (ft/s)
TA-345-3-1B/1A/6A	8-14	SC	4-7	25-50	750-1,050
TA-345-3-1B/1A/6A	14-17	SM	4	15 to 50	700-750
TA-345-3-1B/1A/6A	19-28	SP/SC/SM	4-15	25-100	625-850
TA-345-1-2B/2A	8-16	SC	2-3	20-50	700-1,050
TA-345-1-2B/2A	16-23	SP/SC/SM	5-13	30-50	625-700
TA-250-3-1B/6A/7A	19-23	SM	6	30-50	425-625
TA-250-3-1B/6A/7A	23-30	SC	0-6	5-35	215-425
TA-250-3-2B/2A	19-25	SC	3-8	15-80	550-650
TA-250-3-2B/2A	25-30	SM	1-3	5-25	550-575
TA-17.5-3-1B/1A	8-12	SC/SM	10-12	50-130	1,075
TA-17.5-3-1B/1A	12-20	SC	1-4	15-70	425-1,150
TA-17.5-3-1B/1A	20-26	SP/SC/SM	1-2	15-50	625-700
TA-17.5-1-1B/1A	8-17	SC	2-5	15-75	600-1,025
TA-17.5-1-1B/1A	20-26	SP/SC/SM/ML	0-1	10-25	525-900

The authors concluded the following, regarding the use of the CPT method:

> The CPT Q value (tip resistance) generally followed the trend of the SPT blow count. In those zones where significant changes in blow count occurred, a significant change in Q occurred. However, we

> observed large measures of variation in the CPT, particularly with the "After" data, so much so that we conclude that the CPT is not an adequate substitute for the SPT, given the particular situation at the East Dam Extension . . .
>
> (Note: It is not clear from a review of the data what the authors are referring to as the large variations in the CPT data. It is possible that the variation is seen more in the CPT data as a result of having a continuous profile of data as opposed to the relatively few data points from an SPT test boring.)
>
> The CPT can only measure the mechanical behavior of soils that are being penetrated. The penetration data from a site is compared to "standard" data to classify the soil. For this project, we often obtained different classifications from the CPT and SPT for the same soil.
>
> (Note: The difference in classifications is due to the "gray" zone in CPT with soils containing 5 to 45 percent fines. Soil behavior groups are mixed in that region. In addition, the evaluation techniques for CPT were not as advanced in 1995 as today.)

Only "raw" data, not normalized for overburden stress, were presented, and no liquefaction triggering analyses were performed, since earlier studies had demonstrated that liquefiable materials existed. Instead, the study focused on the before and after values of the three tests, to determine any improvement achieved by stone columns. A total of six locations featured a comparison of all three testing methods.

Detailed analysis of data in the reports reviewed was beyond the scope of our study, but some general observations can be made from a more cursory review. It is fairly obvious, given the low values of N, Q, and V_s throughout many of the intervals, that liquefaction would be predicted in many intervals by all three methods. It also appears that there is reasonably good correlation among SPT, CPT, and V_s measurements for this site, although the shear-wave velocities tended to be somewhat high relative to the other values. In some locations, low blow counts (N values as low as 2 to 3) coincided with high shear-wave velocities indicative of moderately dense material that would not liquefy under any loading (700 to more than 1,000 ft/s).

Skookumchuck Dam—*Liquefaction Potential Evaluation for the Skookumchuck Dam Site* (Shannon and Wilson, Inc., November 2001)

In this consulting report, Shannon and Wilson, Inc. evaluated the liquefaction potential of gravelly alluvium beneath Skookumchuck Dam about 20 miles south of Olympia, Washington. The alluvium generally consisted of 55 to 80 percent gravel and 3 to 12 percent fines, with the remainder consisting of gravelly sands and silts. Maximum particle size of the gravels was from 1 to 2 inches. Because of the coarseness of the deposit, only two SPT holes were drilled at the site, and only one of those adjacent to a BPT hole. This SPT hole, after penetration testing, was completed for geophysics and downhole shear-wave velocities were measured. Several BPT holes were drilled, and one was adjacent to the SPT hole of interest. Due to the low water level (about 45 feet below ground surface), only three depths of SPT and BPT data were in potentially liquefiable materials—mostly gravels. Normalized and corrected values were either reported or independently estimated, utilizing available data from the paper. The authors included three methods of converting the BPT data to equivalent $(N_1)_{60}$ blow counts; the Harder and Seed approach, Sy's method with pullback tests and no energy equation, and Sy's energy method (using energy measurements and wave equations) in conjunction with pullback tests. The SPT, BPT, and V_s data are compared in table 10, and the penetration data are compared in figure 38.

Table 10.—Summary of data from Skookumchuck Dam

Depth (ft)	N value from SPT	$(N_1)_{60}$	Equivalent $(N_1)_{60}$ from Harder and Seed	Equivalent $(N_1)_{60}$ from Sy with only pullback data	Equivalent $(N_1)_{60}$ using Sy energy eq. and pullback data	V_s (ft/s)	Normalized V_s
55	26	14	17	9	13	1,000	750
62.5	40	21	18	44	21	1,530	1,120
65	52	27	19	19	16	1,550	1,130

One observation that can be made is the relatively good agreement between the SPT $(N_1)_{60}$ and the BPT equivalent $(N_1)_{60}$, estimated using the complete Sy method with the pile-driving analyzer used for energy measurements only and pullback tests for shaft resistance, correction in two of the three test intervals. (Obviously, this is a very small sample.) At the same two depths (55 and 62.5 feet), the Harder and Seed correlation also gave similar equivalent SPT blow counts. At the 65-foot depth, neither method matched the SPT blow counts very well. Using only the pullback data to predict the equivalent SPT blow count worked poorly, probably because it was necessary to assume a constant energy output from the hammer. Also noteworthy is that, as in the case described in the previous section under *East Dam and East Dike Extension, Santee-Cooper Project*, the downhole shear-wave velocities appear to be high relative to the blow counts. However, few details were given on the procedures for collecting the shear-wave data, and the results are somewhat method dependent.

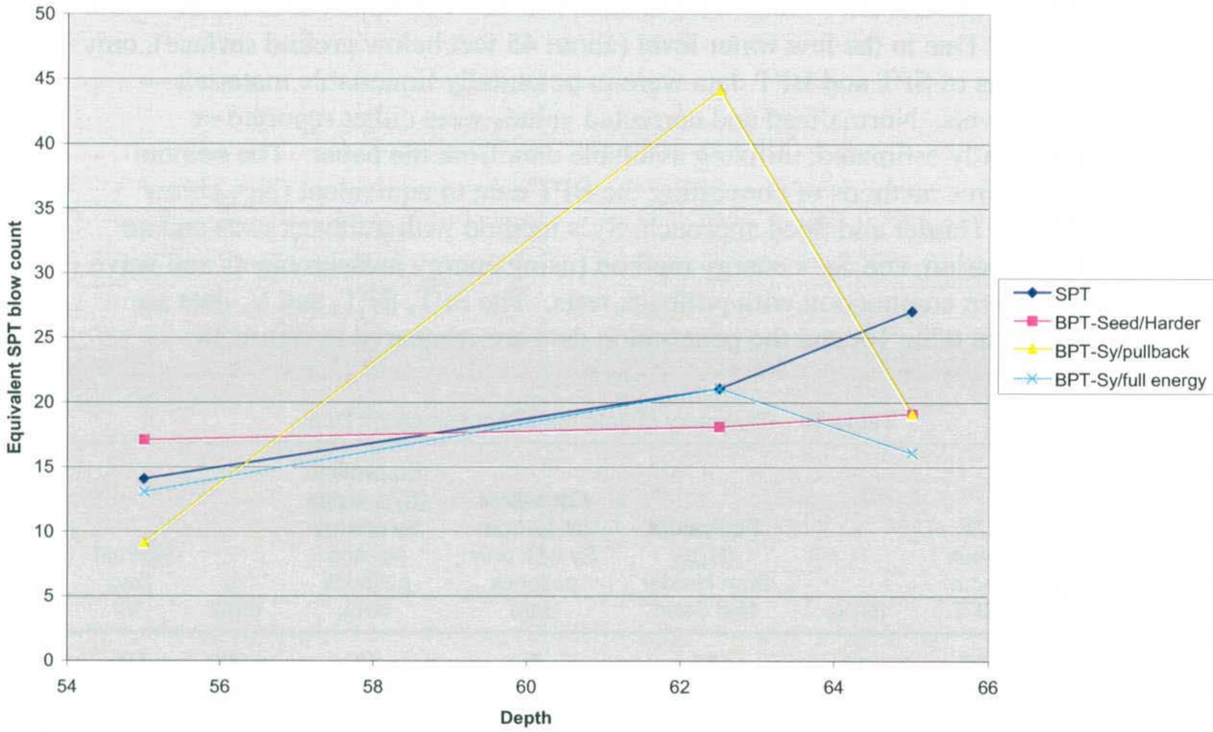


Figure 38.—Comparison of corrected/equivalent SPT blow counts at Skookumchuck Dam.

Steel Creek Dam—*Steel Creek Dam Foundation Densification* (Keller *et al.*, April 1987)

Steel Creek Dam is located within the Savannah River Plant facility in South Carolina. A primary purpose of this paper was to discuss the improvement in foundation soils in test sections treated by dynamic compaction and stone columns, by means of *in situ* testing before and after treatment. However, it was also interesting to note the general indicators of soil condition before improvement as provided by SPT, CPT, and V_s measurement. Plots of the test values provide a good indication of the values. In these liquefiable soils, described as loose, clayey sands, shear-wave velocities as measured in cross holes were in the range of 400 to 450 ft/s. Uncorrected SPT blow counts ranged from 1 to 10. CPT tip resistance looked to range from mostly 20 to 50 ton/ft^2. All three of these measurements are consistently low, showing good agreement among the methods, and each method would almost surely predict liquefaction, in spite of the fines content.

BC Hydro Study—In-Situ *Measurements of Dynamic Soil Properties and Liquefaction Resistances of Gravelly Soils at Keenleyside Dam* (Lum and Yan, October 1994)

This paper by Lum and Yan includes much of the data presented in their study of Keenleyside Dam discussed under Keenleyside Dam on page 82. An interesting addition was a discussion of the correlations between shear-wave velocity and SPT blow counts in gravelly soils. In addition to listing three previous correlations based on field data, the authors developed a new relationship based on their data obtained from Keenleyside Dam. Table 11 shows these correlations.

Table 11.—Various correlations between V_s and N-value

Reference	Shear-wave velocity, V_s (m/s)	Database	SPT energy ratio, ER*
Ohta and Goto (1978)	$V_s = 94.2(N_{67})^{0.34}$	Field data	Japanese rope and cathead, donut hammer, ER=67%
Imai and Tonouchi (1982)	$V_s = 75.4(N_{67})^{0.351}$	Field data	Japanese rope and cathead, donut hammer, ER=67%
Yoshida *et al.*. (1988)	$V_s = 125(N_{78})^{0.25}(\sigma_v'/Pa)^{0.14}$	Lab data	Japanese Tombi, donut hammer, ER=78%
Lum and Yan (1994)	$V_s = 116(N_{60})^{0.274}$	Keenleyside Dam data	Equivalent SPT blow counts from BPT with Sy method

* Note: Energy ratios were estimated using 1985 paper by H.B. Seed and others

Figure 39 shows the data and the relationships. The authors noted that the existing Japanese correlations compare reasonably well with their best fit curve, with the exception of the Imai and Tonouchi relationship, which gives lower shear-wave velocity values. They also note that significant scatter exists in the data, suggesting that significant errors could be expected when attempting to predict penetration resistance from shear-wave velocity or *vice versa*.

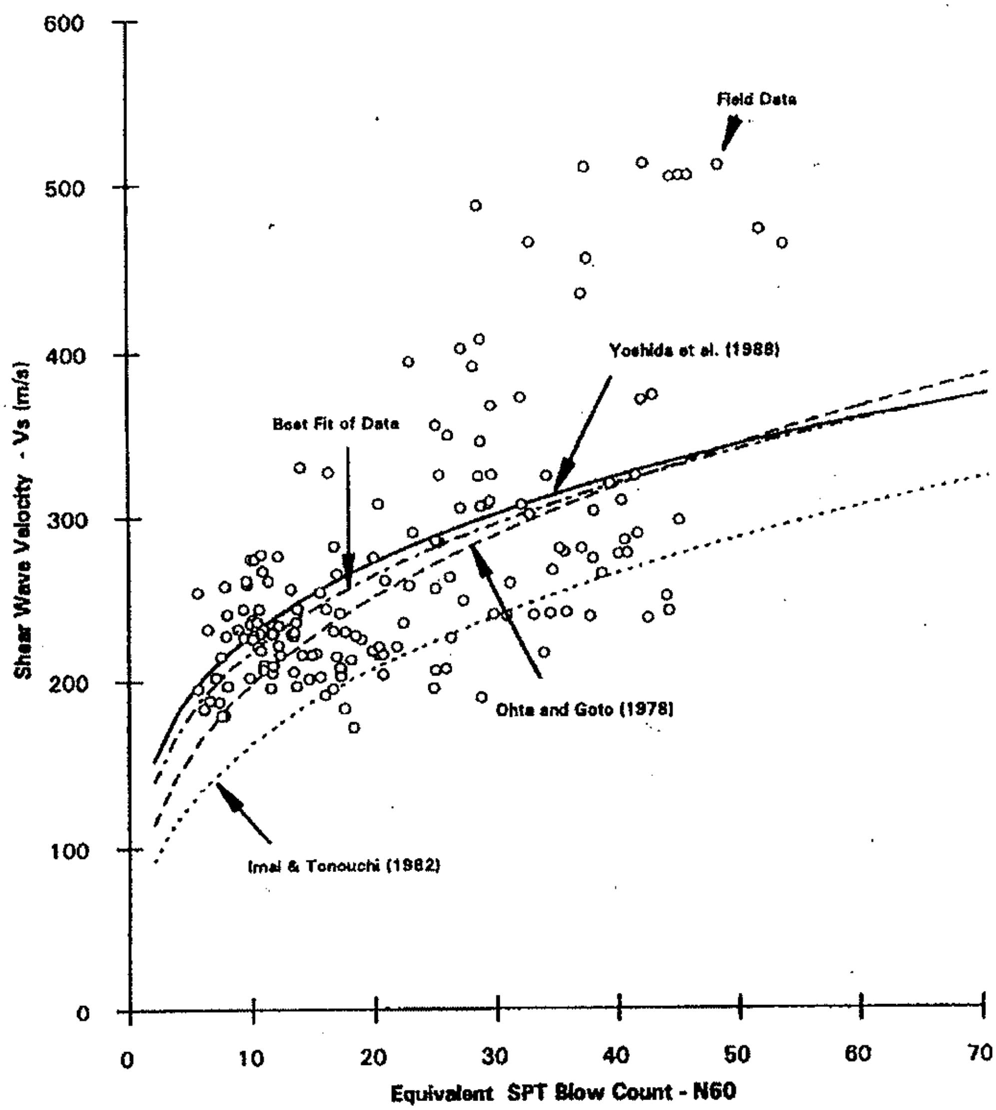


Figure 39.—BC Hydro study of SPT-V_s correlation.

V_s-BPT Research—*Implications of V_s-BPT $(N_1)_{60}$ Correlations for Liquefaction Assessment in Gravels* (Rollins *et al.*, August 1998)

In this paper, Rollins *et al.* report a study of a data set consisting of 379 pairs of shear-wave-velocity and BPT data from 11 gravel sites in the western United States and Canada. Correlations were developed between shear-wave velocity and equivalent SPT blow counts estimated from BPT. Shear-wave velocity measurements included downhole, cross-hole, and SASW. In some cases, two or more techniques were used, and the authors noted that "the agreement between the methods was quite good." BPT data were converted to equivalent SPT blow counts using the Harder and Seed approach only.

Following are the conclusions of this study:

- Correlations between shear-wave velocity and penetration resistance exhibit significant scatter and separate correlations are necessary for Pleistocene age, Holocene age, and recent fill materials. For a given blow count, velocities in Pleistocene gravels are 30 to 70 percent higher than in Holocene gravels. There is almost no correlation between velocity and penetration resistance for embankment fills.

- Correlations between shear-wave velocity and penetration resistance become poorer when blow count and velocity are normalized for confining pressure effects.

- Because of the relatively poor correlation between velocity and penetration resistance, liquefaction evaluations based on the two measurements are likely to produce contradictory conclusions in a significant number of cases. The potential for discrepancies increases for Pleistocene gravels and when V_{s1} is employed rather than V_s. Discrepancies will also be likely in fills where V_s is not dependent on $(N_1)_{60}$.

- Separate liquefaction resistance curves based on geologic age would be desirable when evaluating liquefaction using shear-wave velocity.

(Note: The Harder and Seed correlation was exclusively used. It is possible that evaluation of the BPT data by the Sy and Campanella method would result in stronger correlations with the shear-wave velocities.)

Containment Facility in Washington—*Hidden Hazard: Liquefaction Assessment for a Buried Glacial Stream Valley* (Womack et al., August 1998)

The authors detail a liquefaction assessment for a containment facility near Tacoma, Washington. Foundation conditions resulted from a combination of marine, alluvial, and glacial processes, and the soils range from fine silty and clayey sands to gravels. To investigate liquefaction potential, SPT, CPT, and shear-wave-velocity measurements were taken. Liquefaction potential was evaluated in apparent agreement with NCEER procedures. The factors of safety against liquefaction (i.e., the estimated cyclic resistance of the soil divided by the cyclic loading, or CSR) were computed for most of the foundation deposits by each of the three methods; the results are shown in table 12. The authors noted that the three methods consistently predicted liquefaction in the same layers. Factors of safety among the methods were similar in the silty/clayey sands, but showed more variation in gravelly layers. In addition, it should be noted that in borehole OCF-33, shear-wave velocity measurements appeared to show that liquefaction was unlikely, while the penetrometers indicated the opposite in half of the intervals. (Note: This last observation is not unusual. A number of investigators have noted that shear-wave velocity measurements can indicate a low potential for liquefaction while other *in situ* techniques indicate that the soils are potentially liquefiable. This may be a result of aging and/or cementation, which can increase shear-wave velocities without increasing cyclic resistance, as noted under *Vs-BPT Research* in the previous section.)

Table 12.—Factors of safety against liquefaction—Washington containment facility

Bore hole No.	Cone hole No.	Depth (ft)	Stratum	5% passing #200	D50 (mm)	Liquefaction FSL by SPT	CPT	V_S (cone)	V_S (down-hole)
OCF-27	CH-OCF3[1]	0-7	FILL: sand and gravel	22	2.5	Above water table			
		7-15	ALLUVIUM: silty sand	32	0.2	0.6	0.4	0.5	0.5
		15-18	ALLUVIUM: silty gravel	48	0.1	1.5+	0.8		0.8
		18-23	ALLUVIUM: gravel in OCF27, sandy silt in CH-OCF3	35[2]	0.3	0.6	0.3	0.5	0.6
		23-25	ALLUVIUM: sandy silt			0.6	0.4	0.4	0.6
OCF-32	CH-OCF4[3]	0-9	SLAG						>>1
		9-14	FILL: gravelly sand	8	1.5	0.4			0.9
		14-16	MARINE: sandy gravel			>1			0.7
		16-18	MARINE: sandy gravel	8[4]	0.7	0.7			0.7
		18-22	ALLUVIUM: silty sand	21	0.7	0.5	0.4		0.6
		22-24[5]	ALLUVIUM: sandy			>>1	>>1	>>1	0.5
		24-39	ALLUVIUM: sandy clayey silt	58		>1[6]			
		39-43	ALLUVIUM: silty clayey sand	43	0.12	0.4	0.3	0.5	0.4
		43-49	ALLUVIUM: gravelly silt in OCF32, sandy silt in CH-OCF4	48[4]	0.1	>1	0.4		0.4
OCF-33	CH-OCF1	0-15	SLAG						>>1
		15-22	FILL: gravelly sand				0.4		0.4
		22-24	MARINE: sand	8[4]	0.7	0.4	0.3	0.5	>>1
		24-33	MARINE: silty gravel			>>1			>>1
		33-40	MARINE: sandy gravel	8	5	0.6			>>1
		40-55	ALLUVIUM: sandy clayey silt	78		>1[6]			>>1

[1] CPT Hole CH-OCF3 is located about 50 feet east of BH-OCF27.
[2] Data from OCF-28
[3] CPT Hole CH-OCF4 is located about 70 feet southeast of BH-OCF32.
[4] Data from OCF-10
[5] CPT Hole CH-OCF4 encountered material with V_S of about 1,000 fps, indicating FS>>1 at this depth (22 to 24 feet) not detected by the shear wave survey in OCF32.
[6] FS assumed to be >1 because soils appear to be too clayey to liquefy.

Casitas Dam—*Evaluation of Liquefaction Potential—Casitas Dam* (Wilson and Major, 1998)

This Reclamation technical memorandum presents the results of an evaluation of the liquefaction potential of gravelly, alluvial soils beneath Casitas Dam near Santa Barbara, California. The alluvium was relatively variable in that gravel contents appeared to vary widely, from less than 20 percent to greater than 60 percent. These soils were tested by SPT, BPT, and measurement of shear-wave velocity. Both the SPT and the BPT triggering analyses indicated that widespread liquefaction would take place in the upper alluvial layer. However, a comparison of the actual SPT and BPT $(N_1)_{60}$ values indicated that there was not particularly strong agreement between the tests. Perhaps due to gravel influence, the SPT blow counts were generally higher than the BPT values, but not always. The trends or patterns of the plotted data were not consistent. Dr. L. Harder, who was hired as an independent consultant to review the data and analysis, offered the following conclusions:

- The procedures used by Reclamation to normalize the SPT and BPT data are "acceptably accurate."
- The lower alluvium is "generally nonliquefiable."
- There is discrepancy in the results of the SPT and BPT data in the upper alluvium below the waste berm. Only about half of the BPT and SPT data compare favorably, and there is insufficient data to resolve this problem.

Shear-wave velocities were measured in the upper alluvium in six locations. Most of the normalized shear-wave velocities indicated the alluvium would not be liquefiable, but the data were only slightly above the velocity thought to preclude liquefaction at this site, around 650 ft/s.

Both the BPT data and the shear-wave velocity data indicated the coarser lower layer of alluvium would not be susceptible to liquefaction.

Keechelus Dam—*Liquefaction Analyses—Keechelus Dam* (Tierney and Hill, February 2001)

This Reclamation technical memorandum assesses the liquefaction potential of glacial soils beneath Keechelus Dam in the Yakima Valley in Washington. SPT and cross-hole shear-wave velocity measurement were the two techniques utilized. The weakest foundation unit was found to be a glacial lacustrine unit, with a fines content that varied widely from 15 to 75 percent and a gravel content of generally less than 15 percent. The normalized shear-wave velocities in this lacustrine unit ranged from 750 to 820 ft/s (230 to 250 m/s), which were above the approximately 690 ft/s (210 m/s) considered to preclude liquefaction (NCEER 1997). The few SPT tests in this unit indicated that some localized areas of the foundation (with $(N_1)_{60}$ values in the range of 9 to 13) might liquefy under a

50,000-year earthquake loading of about 0.31g, but not under a 10,000-year loading of 0.16g.

Salmon Lake Dam—*Salmon Lake Dam Modifications—Evaluation of Foundation Treatment Performance with Vibro-Stone Columns and Wick Drains* (Snorteland, February 2003)

This Reclamation technical memorandum presents data from the evaluation and treatment of potentially liquefiable foundation soils beneath Salmon Lake Dam located in north central Washington. The suspect foundation soils are fluvio-lacustrine sediments consisting primarily of silty sand with interbeds and lenses of sandy silt, poorly graded sand, and silty sand with gravel. In determining liquefaction potential, SPT, BPT, and shear-wave velocities were used. Pre- and posttreatment CPTs and SPTs were used to monitor the effectiveness of stone columns for densifying the soils.

A total of five cross hole shear-wave velocity profiles were measured, using one doublet at the dam crest, and one triplet and three doublets spaced along the downstream toe of the dam. In the uppermost 60 feet of the foundation (where the lowest density deposits were identified), the vast majority of the shear-wave velocities below the crest of the dam ranged from 700 to 800 ft/s (unadjusted), with the low being 697 ft/s. At the downstream toe, the raw shear-wave velocities ranged from around 500 to 900 ft/s, with a low of 460 ft/s and the majority of the values averaging around 600 ft/s. When normalized based on estimated properties and water levels, the shear-wave velocities below the crest and at the toe appear to range from around 500 to 750 ft/s.

The SPT and BPT results compared very favorably, both showing similar patterns or trends with depth, as well as displaying very similar $(N_1)_{60}$ blow counts. The majority of the $(N_1)_{60}$ values in the 60 feet of interest generally ranged from 5 to 25, with a few lower values and several higher values. There were widespread areas where the $(N_1)_{60}$ values were less than 15.

Thus, at this site, SPT, BPT, and shear-wave velocities all indicated the potential for widespread liquefaction of the silty sand foundation soils.

Both the SPT and CPT proved to be effective tools for monitoring the effectiveness of stone columns in densifying these foundation soils. No specific correlations or comparisons of CPT-SPT values were studied, although the data show that both corrected CPT tip resistance and SPT blow counts were well to the right of the boundary for liquefaction. The designers did note that at depths greater than 40 feet, the CPT was more problematic because of difficulties with penetration at that depth of treated (dense) soils.

Wickiup Dam—*Wickiup Dam Modification Design Summary* (Reclamation internal document, April 2004)

This Reclamation design summary presents data from the evaluation and treatment of potentially liquefiable foundation soils beneath Wickiup Dam located in central Oregon. This embankment dam included a left wing dike over 10,000 feet long that was founded on fluvio-lacustrine sediments that included interbedded layers of sand (Qfs), gravel (Qfg), volcanic ash (Qfv), diatomaceous silt (Qfd), dense silt and sand (Qfds), and clay and silt (Qf). To determine liquefaction potential beneath the left wing dike, SPT and shear-wave velocities were used.

Four cross-hole, shear-wave-velocity profiles were measured, using three triplets at different locations along the dam crest and one triplet at the downstream toe of the dam. Each of these test locations had an SPT hole located nearby (and one location had two). Table 13 presents a summary of the liquefaction triggering analysis at the four locations. In all of the holes, a diatomaceous silt layer was found between elevations 4240 and 4250. Both SPT and V_s testing indicated that this layer was prone to liquefaction under the assumed earthquake loading. The $(N_1)_{60}$ values measured in this layer were very low (always less than 10 and usually less than 5), and the normalized shear-wave velocities in the layer ranged from 460 to 590 ft/s. Another Qfd layer adjacent to a volcanic ash (Qfv) layer occurred between elevations 4265 and 4280. The Qfd was the more susceptible layer, with low $(N_1)_{60}$ values similar to at the lower elevation. Most normalized V_s values were low as well, often ranging from 400 to 490 ft/s. The Qfv layer typically had $(N_1)_{60}$ values higher than 10, and normalized V_s values generally ranged from 400 to 600 ft/s, although the values were greater than 800 ft/s in the downstream toe drill hole.

The SPT and V_s results compared favorably, showing similar trends with depth as well as both indicating liquefaction in the same critical layers. Thus, at this site, SPT and shear-wave velocities both indicated the potential for widespread liquefaction of the soils.

Table 13.—Predicted liquefaction locations—Wickiup Dam

Elevation	Crest station 19+00 Liquefaction prediction			Crest station 30+00 Liquefaction prediction		Crest station 40+00 Liquefaction prediction		D/S toe station 40+00 Liquefaction prediction	
	SPT		V_s	SPT	V_s	SPT	V_s	SPT	V_s
4315	N	N	N	N	N	N	N	N	N
4310	N	N	N	N	N	N	N	N	N
4305	N	N	N	N	N	N	N	N	N
4300	N	N	N	N	N	N	N	N	N
4295	N	?	N	?	N	N	N	Y	N
4290	N	N	N	N	N	N	N	?	N
4285	N	N	?	N	N	N	N	N	N
4280	N	N	?	N	N	Y	Y	Y	N
4275	?	?	Y	N	N	Y	Y	Y	N
4270	Y	Y	Y	Y	Y	N	N	Y	Y
4265	Y	Y	Y	N	Y	N	N	N	N
4260	N	N	Y	N	Y	N	N	N	N
4255	N	N	Y	N	Y	N	N	N	N
4250	?	Y		?	Y	N	Y	Y	Y
4245	Y			Y	Y	Y	?	Y	Y
4240				Y	Y	Y	N	N	N
4235				N	N	N	N	N	N
4230				N	N	N	N	N	N
4225				?		N	N	N	N
4220				Y		N	N	N	N
4215				N		N	N	N	N
4210				N		N	N	N	N
4205				Y		N	N	?	N
4200				N		N	N	Y	N
4195				?		N	N	N	N
4190				N		N	N	N	N
4185				N		N	N	N	N
4180				N		N	N	N	?
4175								N	?
4170									?

Y = yes, liquefaction indicated by *in situ* testing
N = no, liquefaction not indicated
? = borderline, too close to call

Carrefour Shopping Center—*High-Modulus Columns for Liquefaction Mitigation* (Martin *et al.*, June 2004)

This recent paper by Martin *et al.* documents a very unique case history of a shopping center site in northwestern Turkey. It is the only known Class A prediction for liquefaction, meaning that the site had been evaluated for liquefaction potential, and subsequently experienced an earthquake that tested the prediction. The earthquake occurred part way through construction of ground improvement, so it provided a test of both treated and untreated ground. SPT and CPT measurements conducted during design revealed the presence of a loose silty sand layer at a depth of about 20 feet, with a typical $(N_1)_{60}$ blow count of about 13, and a typical CPT tip resistance of 50 ton/ft^2. Testing after the earthquake indicated that the shear-wave velocity was about 460 ft/s. Designers anticipated that this silty sand layer would liquefy during a significant earthquake. Immediately above and below this layer were ML/CL deposits. These silt/clay strata had typical $(N_1)_{60}$ values of around 3 to 6, a CPT tip resistance of about 10 ton/ft^2, and a shear-wave velocity of around 330 ft/s. Because the latter soils contained 50 percent clay fines, liquefaction was not expected.

During construction, the 1999 Kocaeli earthquake resulted in a peak ground acceleration at the site estimated at 0.24g. In areas of the shopping center foundation that were not treated, liquefaction was observed in the form of ground settlements typically ranging from 2 to 5 inches. No sand boils were observed. It appeared that liquefaction occurred in both the silty sand unit and the adjacent silt/clay layers. In areas of the foundation that had been improved with jet grouting, no liquefaction settlements were observed.

Of particular relevance to this study of *in situ* techniques, this case history demonstrates that SPT, CPT, and V_s methods would have all predicted liquefaction. The magnitude of the earthquake and measured ground motions would have resulted in a CSR of about 0.27. When this CSR value is compared to the measured $(N_1)_{60}$, q_c, and V_s values, liquefaction would clearly be predicted for the silty sand layer as well as the silt/clay layers. There is good agreement among the three methods.

Other interesting aspects of the paper by Martin *et al.* (but beyond the scope of this study) include the uncertainties of predicting liquefaction in silty/clayey soils using the traditional or more recent evolutions of the "Chinese criteria" that address liquefaction potential of fine-grained soils on the basis of index properties, and the effectiveness of jet grouting in preventing liquefaction-induced ground settlements.

Summary of *In Situ* Method Comparisons

The findings from these 12 case histories, as well as 6 of the case histories in the previous section that utilized more than one method, are summarized in table 14 in an attempt to show the relative comparisons in the abilities of various *in situ* liquefaction investigation techniques to predict liquefaction. The degree to which

Table 14.—Comparison of *in situ* investigations techniques in assessing liquefaction potential

Project	SPT-CPT correlation	SPT-BPT correlation	SPT-lpt correlation	SPT-V_s correlation	CPT-V_s correlation	BPT-V_s correlation	LPT-V_s correlation
Chinese Industrial site	Good			Good	Good		
Calaveras Dam		Fair to good					
East Dam and East Dike	Fair to good			Fair	Fair		
Skookumchuck Dam		Good		Fair		Fair	
Steel Creek Dam	Good			Good	Good		
BC Hydro (Keenleyside)				Fair		Fair	
V_s-BPT research						Fair	
Washington containment	Fair to good			Fair to good	Fair to good		
Casitas Dam		Fair		Fair		Fair	
Keechelus Dam				Fair to good			
Salmon Lake Dam		Good		Fair		Fair	
Wickiup Dam				Good			
Avalanche gravel			Poor (SPT had gravel effect)				Good
Bradbury Dam		Poor to fair (gravel effect)					
Mormon Island Dam						Good	
Keenleyside Dam		Good		Fair		Fair	
Pineview Dam		Good	Poor (geology was complex)	Fair			
Deer Creek Dam							
Carrefour Shopping Center	Good			Good	Good		

the conclusions on liquefaction potential obtained from the various methods agree with each other is judged subjectively using verbal descriptors of excellent, good, fair, and poor. (If left blank, the project did not feature any data.) It is recognized that a different evaluator might form different judgments.

In general, table 14 suggests that there is a fair to good correlation among most of the techniques. Table 15 breaks down these subjective ratings.

Table 15.—Summary of ratings of *in situ* method comparisons

Rating	Number	Percent of total
Good	16	40
Fair to good	6	15
Fair	15	38
Poor to fair	1	3
Poor	2	5

Two of the three poor (or poor to fair) ratings were due to the influence of gravel (Bradbury Dam and the avalanche gravel site), which prevented any meaningful values from SPT testing. The third poor rating (Pineview Dam) was due to an extremely complex and heterogeneous foundation, which showed little correlation or continuity of materials even at distances as close as 10 feet.

Interestingly, some of the best reported correlations were between the SPT and BPT, where more than half the ratings were good. This is viewed as a positive endorsement of the evolving BPT procedure.

Whereas the SPT, CPT, and BPT penetrometer techniques tended to show relatively good correlations with each other, the correlation between penetrometer techniques and shear-wave velocity was not quite as strong. Most of the penetrometer-V_s correlations were generally rated as either fair or good, with slightly better correlation with the SPT and CPT, than with the BPT. This finding tends to support the general theme in recent literature indicating that shear-wave velocity testing should be used with caution, rarely as a sole means of identifying liquefaction potential, and generally as a verification technique when other methods are used.

An attempt was made to correlate the ratings with such factors as soil age, method of deposition, gradation, depth of deposit, and water-level conditions. Information was generally lacking for the soil age. No obvious correlation was observed for any of the above factors, except that some correlation did appear for clayey soils. Four sites appear to consist of clayey sands to some extent; the Chinese Industrial site, East Dam and Dike, Steel Creek Dam, and the Carrefour Shopping Center. Interestingly, three of these four showed good correlations between SPT and CPT, between SPT and V_s, and between CPTand V_s. The fourth (East Dam and Dike) showed fair or fair to good correlations. It should not be surprising that SPT and CPT correlate well in finer soils. The limited data in this study suggest that shear-wave velocity measurements correlate best with penetration data in clayey soils.

Alternate Technologies

In the course of the literature review for this study, several papers were found that discussed alternate *in situ* methods of evaluating liquefaction potential. None of these techniques has had wide use or a significant base of verification testing to be generally accepted as reliable. For this reason, they are not covered in detail here; rather, only a brief summary of the available literature is provided. Nonetheless, some of these methods have promise and may warrant additional research and perhaps trials at sites when opportunities and funding sources are available.

Large Penetration Test (LPT)—*A Method for Correlating Large Penetration Test (LPT) to Standard Penetration Test (SPT) Blow Counts* (Daniel *et al.*, January 2003)

Daniel, Howie, and Sy summarize available LPT data and present a method for predicting LPT-SPT correlations. The authors point out that several LPT samplers are being used. Whereas the inside diameter of the SPT sampler barrel is 1.5 inches, these large samplers have diameters ranging from around 2 to more than 4 inches. Four of the most well-known examples of LPTs include the Japanese LPT, the Burmister LPT, the Italian LPT, and the North American (Alaskan) LPT (NALPT).

Through an evaluation of penetration resistance factors, energy input to the sampler, and the use of wave equation modeling to model damping effects, the authors developed a method to correlate various LPTs to each other and to the SPT test. Using this method, they calculated correlation factors for blow counts from an SPT test and the four LPT tests. These calculated values were in good agreement with values computed by other researchers.

The procedure was field tested on a sand site near Vancouver, British Columbia, which had been well characterized by previous liquefaction research efforts. The mean grain size of the sand in this area ranged from 0.19 to 0.56 mm, or somewhere between the No. 100 and No. 30 U.S. Standard Sieve (the coarse end of "fine sand"). Blow counts were measured by the SPT and the North American LPT. A ratio of 1.29 was determined between the SPT and NALPT. Once the SPT data were multiplied by that factor, good agreement was observed between the two penetration methods. There was relatively little scatter or variability between the two methods, and they also demonstrated the same pattern or trend as CPT tests in the same material.

The authors viewed this study as promising, but encouraged the collection and evaluation of additional comparative data on different soil sites.

Seismic Cone–*Seismic Cone Penetration Test for Evaluating Liquefaction Potential Under Cyclic Loading* (Robertson *et al.*, April 1992)

This technique is not so much an alternate technology, as a combination of two existing methods: CPT and shear-wave velocity measurement. Inasmuch as an ongoing theme from the literature is a recommendation to use more than one *in situ* evaluation method on a site, this tool seems to show particular promise. Since Reclamation should have its own seismic cone CPT rig in 2006, future standard procedure for Reclamation CPT testing may well involve the use of this procedure.

Robertson, Woeller, and Finn describe *in situ* methods for predicting liquefaction, concentrating on CPT and shear-wave procedures. They propose a correlation between normalized shear-wave velocity and cyclic resistance. They further illustrate how a seismic cone can effectively evaluate liquefaction by utilizing both the CPT and shear-wave procedures. The seismic cone they used collected CPT data every 2 inches, and shear-wave measurements were made every 3 feet. Field testing on a site in Canada showed that both procedures predicted essentially the same liquefaction resistance of the foundation soil being tested. The authors believe this method to be promising and encourage additional data collection and evaluation.

For soils with higher fines content (>35 percent), the authors are concerned the CPT penetration process involves undrained shearing of the soil, and penetration resistance becomes too insensitive to evaluate liquefaction resistance. Samples in these materials then become important for evaluation against the Chinese criteria, or the more recent adaptations that are now more widely accepted. The authors opine that the influence of fines content and soil plasticity on liquefaction susceptibility is an area that still requires additional research and understanding.

Vibroseis Machine—*Field Evaluation of Liquefaction Resistance at Previous Liquefaction Sites in Southern California* (Stokoe and Rathje)

This paper or research proposal (date unknown), prepared by Professors Stokoe and Rathje, describes a field method being developed by the University of Texas at Austin to directly measure liquefaction resistance of granular soils. Essentially, a soil deposit is instrumented (for motions and pore pressures) and then dynamically loaded incrementally until liquefaction is achieved. Loading is provided by a Vibroseis machine originally designed to provide excitation for seismic geophysical surveys. This is a new procedure that is still in the testing and field evaluation mode. Rigorous testing at sites that are well characterized by other methods has apparently not yet occurred. In limited testing, liquefaction has been measured within about 5 feet of the ground surface, but the researchers believe it should work within about 15 feet of the ground surface. Although this is relatively shallow, the authors point out that about 50 percent of reported case histories of liquefaction have been within this depth.

In this method, a Vibroseis machine mounted on a truck is used to induce a dynamic vertical load to a saturated, granular soil. Instrumentation embedded in the soil deposit includes pore pressure transducers and geophones. The Vibroseis generates surface waves that in turn induce shear strains and pore pressures increases in the soil. The loading is staged up until significant excess pore pressure (liquefaction) is achieved. Plots of shear strain and excess pore pressures versus time are generated during the testing.

In order to further develop and test this method, Stokoe and Rathje plan to conduct tests at sites in Southern California where liquefaction has been documented and the site has been well tested.

This method is promoted as a means to *directly* measure liquefaction potential *in situ*. Other available and currently utilized methods identify potentially liquefiable soils *indirectly* based on geological criteria, empirical correlations that relate liquefaction potential with field test parameters, and laboratory cyclic tests on undisturbed samples or reconstituted samples. The laboratory tests measure the liquefaction characteristics of the soil more directly, but they too suffer from limitations due to heterogeneity of deposits, and boundary conditions and strain limitations in the test apparatus. In contrast, the Vibroseis test can actually demonstrate liquefaction potential of the deposit in the field with no concern about sampling disturbance, and less concern about boundary conditions.

The authors included a listing of problems with the traditional empirical and laboratory testing procedures for evaluating liquefaction potential.

Problems with empirical methods in general

- Very few acceleration measurements are available at liquefied sites, so researchers have had to estimate the surface accelerations. Due to the large spatial variability in earthquake ground motions, the estimates are highly uncertain.

- Surface expressions of liquefaction have been used to indicate site liquefaction, while such surface evidence may not occur at some sites that experience liquefaction.

- These types of uncertainty reduce the reliability of the procedure (particularly near the threshold line), which may be why some of the empirical data do not plot on the appropriate side of the liquefaction threshold line.

- Empirical procedures do not explicitly account for the duration of earthquake shaking. The CSR is estimated for the number of uniform cycles expected during a magnitude 7.5 event and then corrected for the anticipated number of cycles during the site earthquake. This assumes that the relation of CSR and number of cycles is the same for all soils, and that the number of cycles is reasonably well represented by the earthquake magnitude, neither of which is always true.

- Empirical methods cannot predict partial porewater pressure generation, nor the strain and flow potential of liquefiable soils.

<u>Problems with laboratory testing</u>

- Any disturbance of samples can make laboratory tests unreliable.

- Sampling methods that attempt to minimize soil disturbance (block sampling, freezing) are time consuming, difficult, and expensive.

- Testing of reconstituted or remolded specimens changes the soil fabric, structure, and stress history, any or all of which may play a key role in liquefaction potential.

- Laboratory tests do not faithfully replicate the boundary conditions and stress path that soil in the ground would be subjected to.

(Note: Although not listed by the authors, the proposed Vibroseis method also has significant limitations or problems, as identified elsewhere herein:

- It is very early in the development stage as a tool for liquefaction evaluation, and thus has yet to gain any widespread acceptance.

- The load boundary conditions and strains induced by the Vibroseis do not faithfully reproduce those in an actual earthquake. It may therefore not be able to quantify the cyclic resistance of the soil, even though it could be very effective for qualitative evaluation of soil behavior in an earthquake.

- The limitation of a 15-foot depth for testing would severely restrict the application of this method. Although the authors note that about 50 percent of *observed* soil liquefaction has occurred at depths of less than 15 feet, that may be because it is easier to "observe" shallow liquefaction. In seismic analyses of major earth dams, liquefaction potential often has to be accurately and reliably assessed at depths of 200 feet and more.

- The Vibroseis "tests" for liquefaction potential by actually liquefying some fraction of the suspect soil units. This could be potentially dangerous in situations where large or critical structures (like a major earth dam) impose large static driving shear stresses, as the liquefaction induced by the Vibroseis might result in potentially catastrophic failure of the structure. The use of explosives to compact liquefiable soils at depths is typically not considered at dams for this same reason, which would appear to suggest a limited use of the Vibroseis at embankment dam sites.)

Nuclear Gauges—*Liquefaction Study of Coal Refuse by Nuclear Gauges* (Cowherd, 1986)

In place densities and moisture contents in deposits of coal refuse were measured using a nuclear probe. Small augers were used to advance a hole and steel casing was installed to depths of 100 feet or more. The nuclear gauges could then be lowered into the casing and used to measure density and moisture content in a nearly continuously and in a relatively short time.

The author believed the SPT procedure was best for relating liquefaction potential to not only relative density, but also the coefficient of pressure at rest (K_0), the over-consolidation ratio (OCR), and the stress history. However, since fine coal refuse is deposited by settling in water, it is a young (geologically) deposit with a K_0 of about 0.4 to 0.5 and an OCR of 1.0. Thus, density is the only important parameter for liquefaction of coal slurries, according to the author. This suggests that the nuclear gauge method would be the best technique for evaluating liquefaction potential of coal slurries. The author does note that coal slurries typically have more than 15 percent fines, so Proctor testing was utilized to obtain the maximum density for relative density determination. Resulting relative densities were then input into the Seed and Idriss simplified liquefaction procedure. It is not clear whether the author used the earlier version based on relative density, or converted the relative density to N_{60} values by some means.

To ensure accuracy, samples of the material are collected for calibration. Remolded densities are prepared and calibrated to the nuclear gauge readings to develop a calibration curve for the material to be sampled.

In several case histories, densities and moisture contents from the nuclear gauges compared well with the values determined from sampling by Shelby tubes.

However, it was recognized that getting true densities and moisture contents of a slurry-type material by Shelby sampling may not have a high degree of accuracy.

Notes:

- For tailings deposited by settling underwater, fabric variability, overconsolidation, and cyclic load history would not have major influences on liquefaction potential, so that density is the principal variable in determining liquefaction resistance. This is not true of most natural soil deposits and embankment fill units, so the applicability of this technique at embankment dam sites is limited.

- Material-specific correlations would be required, unless a "robust" correlation can ever be established. Variation in mineralogy may affect the correlation.

- In addition, the risk of losing a nuclear gauge in the ground has generated some concern in the use of this technique.

Special Undisturbed Sampling Techniques

This approach is another example, like the seismic cone, that may not be considered an alternate technique, but is a means of determining densities in a soil deposit, which is the principal behind the use of nuclear gauges in the section above.

Seismic Studies—Pinopolis West Dam—Santee-Cooper Project, South Carolina (GEI [Castro *et al.*], 1985)

This report presents the evaluation of the liquefaction potential of sand layers in the foundation of Pinopolis West Dam. Undisturbed samples were taken and tested in the laboratory to determine steady-state strengths of critical soil layers; the potential for instability was then evaluated by comparing the steady-state strengths to the strengths needed to resist a slope stability failure of the dam. As stated in the beginning of this report, a discussion of the steady-state strength approach to liquefaction evaluation (requiring undisturbed sampling and laboratory testing) is not within the scope of this study. However, since density is widely viewed as the most important factor in determining liquefaction resistance, a brief description of the typical "GEI approach" to undisturbed sampling is appropriate.

As described in the report (and in many others by GEI), great care is taken in obtaining as undisturbed a soil sample as is possible for both density and strength testing. For shallow soil layers, densities were measured in test pits by means of "undisturbed tripod tube samples." At Pinopolis West Dam, test pits were excavated by a caisson rig with a 6-foot diameter auger, using casing as needed. This excavation method was used to get within 2 to 3 feet of the suspected problem soil layers. The remaining excavation was performed by hand. Half or quarter circles of plywood were placed at the bottom of the test pit to provide platforms for the person performing the hand excavation or sampling. At desired sampling depths, the bottom of the excavation was carefully leveled and smoothed.

A tripod was placed over the sample location, and a sampling tube and follower were placed in the tripod, with the tube resting on the ground surface. Spoons and small cutting tools were used to carefully hand excavate down and around the tube. After short distances (about ½ inch) of excavation, the tube and follower were carefully pushed down. The process continued in this manner until the depth reached the height of the sampler. Measurements were taken throughout the process to see if the top of the sample had changed at all (indicating, and allowing for correction of, expansion or contraction). After extraction of the tube (using sheet metal at the bottom of the tube), measurements of the sample in the tube were made so that any changes in void ratio during transportation or storage could be determined. Tubes were kept vertical during virtually all of the process, including sampling, handling, and transportation.

For deeper samples at Pinopolis West Dam, GEI used fixed-piston tube sampling. As with the test pit procedure, great care was taken to minimize sample disturbance. Special measures included bits with deflectors to discharge drill fluid upward instead of toward the sample location, carefully measured and maintained cutting edges on sampling tools, very tight tolerances on clearance ratios between tube and cutting edge, and the use of an independent frame to rigidly hold the actuating rods attached to the sampler. As with the test pit samples, special care was taken to avoid jarring or disturbing the tube samples during all phases of the operation, and precise measurements were made at various stages to estimate any changes in void ration during the process.

By following these procedures, GEI expressed confidence in the accuracy of void ratios (or in-place densities) determined for the foundation materials.

(Note: When carefully done by experienced personnel, these procedures probably yield high-quality density data. One concern with this process is that very detailed and precise procedures and measurements are required, which are both expensive and time consuming, as well as requiring experienced and trained personnel. Perhaps the major concern, however, would be that (as pointed out several times in this report) density is not the sole indicator of liquefaction resistance of a soil. Most of the *in situ* techniques account for the influence of the other factors affecting liquefaction potential by direct empirical correlation with field performance.)

Case Study of Rock Debris Avalanche Gravel Liquefied during 1993 Hokkaido-Nansei-Oki Earthquake (Kikusho *et al.*, 1995)

This paper describes the Japanese technique of freezing ground to obtain undisturbed soil samples. At this site, about 3 feet of material was excavated to get down to the suspect gravel layer. A main freezing tube was then carefully inserted about 3 feet into the gravel layer. A top plate about 4 feet in diameter, complete with coiling tubes, was placed on the surface of the layer, and a dead weight equal to the overburden removed was applied to the plate. Liquid nitrogen was then injected into the freezing pipe and coiling tubes, and the temperature change of the soil was monitored by thermocouples. Once the soils had frozen, soil surrounding the frozen mass was slowly excavated around the entire mass. Insulation was placed around the frozen mass as the side excavation progressed. When completely excavated, the frozen mass was removed by crane. Individual specimens were cut from the frozen material for laboratory testing, including in-place density. An adjustment was made for some sample consolidation as careful measurements during the process indicated the ground had settled slightly.

(Note: Similarly to the GEI process described above, this method does provide relatively accurate determination of in-place densities in potentially liquefiable soils. However, it carries the same two primary two concerns: (1) that the process is very time consuming and expensive, and (2) density is not the only factor governing the liquefaction resistance of a soil.)

Electrical Method—*Comparison of the SPT, CPT, SV and Electrical Methods of Evaluating Earthquake Induced Liquefaction Susceptibility in Ying Kou City During the Haicheng Earthquake* (Arulanandan *et al.*, 1986)

Six sites were examined in this Chinese city where the Haicheng earthquake hit on February 4, 1975. The earthquake was a magnitude 7.3 event with estimated maximum ground surface accelerations in the city of between 0.1 to 0.15g. Of the six sites, four had experienced liquefaction from the earthquake, one had apparently not, and one was judged to be on the brink of experiencing liquefaction (apparently very high excess pore pressure occurred, but dramatic signs were not apparent). Soils at these sites were relatively fine-grained, consisting of inorganic silts, silty or clayey fine sand, inorganic clays of low to medium plasticity, sandy clays, silty clays, and lean clays. Water contents ranged from 24 to 35 percent, liquid limits from 23 to 40, plasticity indices from 6 to 14, and fines contents from 40 to 90 percent.

The electrical method consisted of determining vertical and horizontal electrical conductivity of the soils, and of the pore fluid, then calculating several parameters that relate to porosity, particle shape, and anisotropy of soil deposits. These are used in an empirical relationship to predict liquefaction resistance.

The SPT method predicted liquefaction at all six sites. Weak layers at these sites had blow counts in the 3 to 5 range. The SPT was not judged to be a good test for the sensitive, high fines content soils present at these sites. CPT readings were converted to blow counts, and also predicted liquefaction at all six sites. Downhole shear-wave velocities in the weak layers were reportedly between 260 and 500 ft/s, which is unusually low. Using the methodology for predicting liquefaction from Stokoe (1984) in use at the time, no liquefaction was predicted at any site (although it was recognized that the velocities suggested liquefaction potential). Using the electrical method, liquefaction was predicted for the five sites that were believed to have experienced liquefaction. No liquefaction was predicted for the one site that did not show any signs of liquefaction.

The authors concluded that the electrical method appears promising for evaluating liquefaction potential at sites with sensitive soils.

(Note: The electrical resistivity of soils is sensitive to a large number of factors, including fine layering, anisotropy, and particle shapes. Site-specific correlations could be required. No shear strain at all would occur during resistivity testing, in contrast to very large strains in penetration testing. This test shares with shear-wave-velocity testing the issue of small (or no) strain in the test, in contrast to very large strain associated with liquefaction. To date, the database of case histories to validate the proposed relationship with liquefaction potential is quite small.)

Multi-Channel Analysis of Surface Waves—*Some contributions of in situ geophysical measurements to solving geotechnical engineering problems* (Stokoe *et al.*, 2004)

This recent paper by Stokoe *et al.* discusses some of the latest advances in shear-wave velocity measurement. One particular technique with promise is the multi-channel analysis of surface waves, or MASW. An additional refinement over SASW comes through analysis of data by two approaches rather than just one: the swept-frequency record approach and the frequency-wave number spectrum approach. Reclamation has had no experience with this technique to date, but it deserves consideration as it is further developed.

Summary of Findings

More than 30 years ago, in their groundbreaking work with developing a simplified method for evaluating liquefaction potential, Seed and Idriss listed five fundamental factors believed to influence the liquefaction susceptibility of a soil. These were soil type and structure, relative density or void ratio, initial confining pressure, intensity of ground shaking, and duration of ground shaking. The latter three are generally characterized by the cyclic stress ratio. This portion of any liquefaction evaluation is estimated from expected earthquake loadings at a site.

The second key part of a liquefaction evaluation consists of determining the pertinent soil properties. *In situ* methods are thus concerned with developing direct or indirect measurements of a soil's type, structure, and void ratio or relative density.

There are two basic approaches to measuring a soil's potential for liquefaction under a given earthquake; one involves the use of *in situ* techniques and the other laboratory testing. The *in situ* methods are used far more often, due to the difficulties and expense of collecting and testing sufficient numbers of undisturbed samples of the soils being evaluated. Although many of these difficulties can be overcome with sufficient time and money, typical practice reserves laboratory testing and in-place density measurements for large projects where refined analysis is required. In addition, a substantial database has been developed to reasonably well validate *in situ* techniques for evaluating liquefaction potential. (Note that this conclusion applies primarily to the sands, gravels, gravelly sands, and silty sands typically considered potentially liquefiable. Methods for slightly cohesive soils are still being in development, and eventually, preferred practice could involve laboratory testing.)

Although relative density (or relative compaction) is generally considered the most important soil property with respect to liquefaction potential, it is not the only consideration. Soil fabric or structure, the age of the deposit and/or presence of even slight amounts of cementation, the soil fabric or structure (a function of its mode of deposition), overconsolidation, and prior cyclic loading history may also have a significant influence. Therefore, the liquefaction potential of a soil should not be based solely on in-place density (or corresponding relative density or relative compaction). Penetration tests provide a general indication of density, but they are also influenced by some of the other factors (e.g., age and fines content) that affect liquefaction potential. As a result, they can provide better predictions of liquefaction potential than do density measurements on their own. Furthermore, penetration resistance, like liquefaction, is a large-strain phenomenon, and is therefore more directly linked to liquefaction behavior than small-strain or no-strain measurements, such as shear-wave velocity or electrical resistivity. Thus, *in situ* penetration tests should be less affected by minor cementation or aging effects.

Further difficulty with basing liquefaction potential on density arises from the inherent expense, time requirements, and difficulty of measuring the density, especially below the water table. Sampling of any soil more than a few feet deep entails difficulties with ensuring the soil is not disturbed by dewatering, or exploration equipment. Getting an undisturbed and reliable sample of a loose and cohesionless soil (the type usually most prone to liquefaction) can be very difficult. Even if the soil can be tested in place in a test pit or exploration shaft, measuring its density is still not easy. Sand-cone and large ring density tests are subject to errors, and it would be time consuming and expensive to perform enough tests to ensure that a soil deposit is adequately characterized.

Furthermore, there are problems with the tests to determine the minimum and maximum densities, which can lead to significant uncertainty in determining the relative density once the in-place density has been measured. (For this reason, Reclamation often uses percent compaction for embankment compaction control.)

These difficulties point out why the *in situ* techniques are generally correlated directly with liquefaction potential, rather than being used to estimate relative density, which would in turn be used for assessing liquefaction potential. They also point out why liquefaction evaluations are much more often performed with *in situ* methods than by laboratory testing.

Nonetheless, it is important to recognize that relative density is still a useful tool in liquefaction evaluations. There are some projects, such as when the potentially liquefiable soils may be shallow, the geology is not complex, and dewatering could be economically accomplished, where relative density data could be economically obtained in large enough numbers to ensure a reasonably accurate representation of the soil being evaluated. In some cases, perhaps in particularly coarse soils, deep shafts with density testing at specified intervals may be considered. In addition to providing density information, engineers and geologists can examine the soil directly to obtain insight into cementation, structure or fabric, or related characteristics. Density testing can also be compared to penetration data to help determine whether large particles may be generating uncertainties in the data collected from *in situ* procedures. Finally, relative density or relative compaction serves as a valuable tool in monitoring the placement of engineered fills and ensuring their resistance to liquefaction, whether for initial dam construction or as part of a dam modification.

The most widely used *in situ* techniques currently in use for liquefaction evaluations are the SPT, CPT, BPT, and measurement of shear-wave velocity. Each of these methods is generally regarded as an acceptable means of evaluating liquefaction potential, although each has its limitations. Other *in situ* methods are also available, but have far less history and are generally considered unreliable. Table 16 shows a comparison of these four methods, with advantages, disadvantages, and suggested applications.

Due to the previously outlined concerns with density determination, combined with the widespread use of SPT in soils characterization, the SPT emerged early on as the preferred *in situ* method of measuring a soil's resistance to liquefaction. It still stands today as the most widely used *in situ* technique. An abundant database of liquefaction and nonliquefaction case histories supports it. However, this method is not without its difficulties and potential for error. It is not a completely standardized test and is open to operational error or variability, and there is not even full consensus on certain aspects of data reduction. It can create disturbance in loose, cohesionless soils and is also subject to influence from gravel particles, which can lead to an overprediction of blow counts if not corrected for. In addition, the majority of the case histories used in the empirical

Table 16.—Comparison of *in situ* investigations techniques in assessing liquefaction potential

Tool	Suggested usage (applicable soils)	Advantages	Disadvantages	Comments
SPT	• Reliable in sandy soils • Reasonably reliable in low plasticity (low clay content) silty soils • Use with caution in soils with fine gravels; record penetration *vs* blow and adjust for "gravel interference;" compare with BPT and/or V_s. • Use with extreme caution in soils with more than 30 percent gravel or in medium/ coarse gravels.	• Largest empirical database for liquefaction evaluation • Most widely used; has industry confidence • Sample is obtained, which allows for direct measurement of gradation and fines content	• Procedure is not fully standardized (different means of dropping hammer, drill fluids, various means of advancing hole, etc.) • Process can create disturbance in loose, cohesionless soils, leading to misleading (over-conservative) blow counts • Presence of gravel can lead to misleading (unconservative) blow counts	• Method developed for Holocene clean sands, near-level ground, and depth less than 45 feet; other conditions introduce uncertainties. • Still the industry standard, particularly in soils with limited gravel • Should be used as either the main or supplemental technique in sandy and silty soils • Recording penetration *vs* blow is preferred practice so gravel corrections can be made if needed. • Drilling techniques should minimize disturbance and suction that can cause "flowing sand," caused by withdrawal of drill bit or sampler • Drilling fluid is required to balance pressure that can heave bottom of hole and produce false, low N-values
CPT	• Reliable in sandy soils • Reasonably reliable in low plasticity (low clay content) silty soils • Use with caution in soils with fine gravels • Not recommended in soils with greater than 30 percent gravel or medium/coarse gravels	• Penetration resistance measured at cone tip, eliminating concerns over energy differences or losses • Provides a continuous record of penetration resistance throughout depth of deposit (more detailed stratigraphy and confidence in seeing thin layers) • Minimum disturbance to soils • Can better assess thin strata than the SPT • Faster and less expensive than the SPT • More standardized than the SPT and less subject to operator variability • Empirical database is growing and now quite extensive • By using seismic cone, can also get shear-wave velocity with minimal additional cost	• No sample is obtained • Fines content determination and use in liquefaction evaluation is somewhat controversial • Presence of gravel can lead to significant uncertainties in q_c values • Measured tip resistance can be influenced by soils located several tip diameters below the cone tip (although this also applies to SPT testing as well) • With some CPT rigs, it is difficult to penetrate dense sands or sands at significant depth	• Method developed for Holocene clean sands, near-level ground, and depth less than 45 feet; other conditions introduce uncertainties • Method has developed rapidly and now considered almost on same level as SPT • Should be used as either the main or supplemental technique in sandy and silty soils • Use of seismic cone is recommended so that both CPT and V_s data can be obtained at same time • Additional techniques or separate sampling holes are a necessary part of any CPT program.

Tool	Suggested usage (applicable soils)	Advantages	Disadvantages	Comments
BPT	• Best suited for gravelly sites, although nongravelly materials are needed to develop/verify correlations with SPT.	• Because this procedure uses a larger penetrometer, it is subject to less gravel influence than the SPT or CPT • BPT soundings (as well as Becker sampling holes) are relatively fast and reasonably economical • Can obtain sample adjacent to BPT using Becker sampling (although sample is completely disaggregated and moisture content is altered)	• Procedure is not entirely standardized • Empirical database for liquefaction evaluation is not nearly as extensive as for SPT and CPT • Generally must be used to predict equivalent SPT blow counts, which can be problematic in coarse soils • Coarse gravels and cobbles interfere with penetration • Correlations for hammer energy and casing friction are still being debated • Hole can be significantly off vertical, which may raise uncertainty about actual depth to sample. • BPT does not provide sample, and Becker sampling yields a highly disturbed sample • Specialized equipment (requires contract) • Effect of fines is not completely known—generally assume it's the same as with SPT, but this has not been verified	• Recommend applying both Harder/Seed and Sy/Campanella correlations when reducing BPT data. If differences are appreciable, weighting of methods by judgment may be appropriate. • Harder/Seed correlation developed at sites with depths of less than 90 feet and with no dense overlying deposits—use with caution if site is different from this. Results are sensitive to rod friction. • Should be used in conjunction with gravel-adjusted SPT testing to help verify SPT-BPT correlation at site • Use of AP-1000 drill rig recommended, along with PDA for energy measurement, and pull-back tests
V_s	• Can be used in all soil types • Since more confidence exists with SPT and CPT, V_s is probably of most use in coarser soils (where SPT and CPT are unreliable) • Since V_s is typically used in response analyses (SHAKE, etc.), V_s measurements will often be obtained in any soils investigation program. In these cases, also evaluate liquefaction with this technique.	• Limits disturbance to soils • Can be used in coarse soils (unlike the SPT and CPT) • Sampling can be done in conjunction with drilling holes for V_s measurement • Some techniques (SASW) do not require any drilling • Tend to get an average V_s value between holes (or between probes), so individual cobbles and boulders have less influence	• Smaller empirical database than SPT and CPT • Correlation to cyclic resistance is less well defined than either the SPT or CPT • For crosshole or downhole V_s testing, grout penetrating coarse gravels and cobbles can cause over estimation of actual V_s. • Up to 3 boreholes needed for crosshole V_s • Because V_s is a nondestructive, micro-strain measurement, may not relate well to liquefaction, which is a destructive, large-strain phenomena • V_s can vary significantly with age of deposits. • V_s can be strongly influenced by aging effects and cementation or particle attractions, whereas liquefaction may not • Not as sensitive to changes in relative density as the SPT and CPT	• Method developed for Holocene clean sands, near-level ground, and depth less than 30 feet; other conditions introduce uncertainties. • Recommended to supplement BPT and gravel-adjusted SPT testing in gravelly soils • Generally not considered sufficiently robust to use as a stand-alone means of evaluating liquefaction potential, particularly if results are marginal. • Select appropriate drilling method (to help minimize disturbance) • Consider potential for "micro-cementation." V_s may not be appropriate in soils with cementation or high particle attractions.

relations involved certain site conditions such as level ground and fairly shallow limited depths. For sloping ground and high overburden pressure, one must apply adjustment factors based largely on theory and laboratory testing. (This last difficulty applies equally to other *in situ* methods.) Thus, the SPT must be used with due caution and recognition of the uncertainties.

The CPT has evolved into a widely used method for liquefaction evaluation, and is approaching the SPT in level of use and confidence. Its appeal stems largely from the speed and low cost with which large numbers of data can be collected, and from theoretical advantages in interpreting the mechanics of penetration. It has advantages over the SPT in that it does not create disturbance in loose soils, produces a continuous record of data with depth, and can thus provide more detailed stratigraphy. Like the SPT, however, the CPT's usefulness is limited in gravelly soils. The biggest disadvantage of the CPT is that the method does not provide samples of the soil being investigated, and must be supplemented by drilling and sampling. It is noteworthy to observe that the 1996-97 NCEER working group was unable to reach consensus on the selection of a CPT procedure for evaluating liquefaction potential. In particular, the fines content (and fines correction) to use in a CPT evaluation appear controversial. Additional research since 1997 supports fines adjustments that are somewhat different from either of the two considered by the NCEER group.

The BPT is becoming widely used in gravelly soils. However, it has more uncertainties than either the SPT or CPT. Rather than having a large database in which a BPT blow count can be compared directly to case histories of liquefaction and nonliquefaction, BPT blow counts must instead be converted to equivalent SPT blow counts. Particular issues in this conversion involve the determination of the amount of energy imparted and the effect of casing friction. As of yet, this technique is not considered to be as accurate a portrayal of liquefaction resistance as SPT or CPT because of the complicating factors, and the need for a two-stage correlation. However, a review of case history data has revealed a surprisingly good correlation between SPT and BPT, although a large amount of scatter in the correlation is often evident. In combination with short-interval SPT for correlation, it is regarded as one of the best methods to characterize liquefaction of gravelly soils. The BPT does not provide a sample; this disadvantage can be overcome to some extent by utilizing periodic Becker sampling holes adjacent to BPT holes. However, the quality of these samples is not great and does not allow for detailed logging.

The LPT has to date had rather limited application. Furthermore, there are a number of different LPTs in use, and the method and equipment are far from standardized. Although it has potential, it is viewed to still be in the development stage for liquefaction evaluation. However, large sample barrels have proven useful for recovering gravel deposits when SPT sampling is limited. The LPT has essentially all of the drawbacks that the SPT does, aside from being able to accommodate larger gravel, in addition to requiring a two-stage correlation like the BPT.

A final, commonly used *in situ* investigation technique that does not measure penetration resistance is shear-wave velocity, or V_s, measurement. A principal weakness of this method is that the database of liquefaction and nonliquefaction case histories, though growing, is still much smaller than for the SPT or CPT. In addition, aging of soils and even minor cementing can have a strong influence on V_s; thus the method is normally recommended for Holocene soils only (although even Holocene soils can exhibit aging effects that may make V_s results questionable). A review of case history data suggests that V_s measurements tend to show only a fair correlation with penetration techniques, whereas various penetrometers tend to show a relatively good correlation with each other. V_s is currently used mostly at gravelly sites and as a verification technique for other methods; this is viewed as a prudent use of this technique. At coarse gravel and cobble sites, use of crosshole or SASW shear-wave techniques can be valuable for comparison with penetrometer data. Also, shear-wave velocities are often needed for site response analysis in addition to liquefaction assessment.

General industry practice is to use more than one of these *in situ* investigation techniques for large projects or ones with potentially serious public-safety issues. There are a number of published papers or reports that compare the results of various techniques in predicting liquefaction potential. A review of this literature indicates that at the vast majority of sites tested, there is fair to good correlation between the various methods. In other words, triggering analyses based on relative density, V_s, or blow counts typically give the same prediction of liquefaction or nonliquefaction, although the precise level of shaking predicted to cause liquefaction may not be exactly the same. In addition, the various techniques usually display similar trends: low measurements (indicating more susceptibility to liquefaction) in the same general zones or layers within the soil profile.

These positive correlations are more often observed between different penetrometer techniques (such as between SPT and CPT or between SPT and BPT), than between penetrometers and V_s. Although there were a small number of cases where V_s was low relative to the blow count data, more often the V_s values were higher relative to the blow counts (That is, the V_s indicated higher resistance to liquefaction than did the penetration data).

The case histories suggest that all four basic *in situ* methods have shown reasonably good correlation to relative density, although the available database is not large. This good correlation provides general validation of the use of *in situ* techniques to evaluate liquefaction potential of soils.

Of all the cases in the literature reviewed for this study, only one instance (Moss and R. Seed, 2004) of *in situ* measurements in a soil subsequently experienced a significant earthquake (a "Class A" prediction). Most other publications reporting correlation techniques to performance case histories use testing at sites that have previously shown signs of liquefaction or nonliquefaction under an earthquake loading (a "Class 3" prediction). With this often overlapping database being utilized, it should not be surprising to see reasonably good correlation between the liquefaction evaluation procedures used for these various techniques.

Without question, there are uncertainties in the use of any *in situ* method to predict liquefaction potential, as there are with most geotechnical engineering evaluations. It should be recognized that evaluation of liquefaction potential, with any method, contains a fair amount of uncertainty. These uncertainties exist in the earthquake loading, how the ground motions or earthquake energy travels through the bedrock and overlying overburden, the degree of continuity or extent of the susceptible soils, how the soils do actually respond to the earthquake shaking, and several other factors. In addition to these uncertainties, there are uncertainties with the modeling of an embankment dam's performance in the event that liquefaction does occur. In light of these uncertainties in seismic analyses, practitioners and decision makers should always keep in mind the number of uncertainties inherent in the evaluation of liquefaction potential, and base conclusions and decisions on the engineering analyses tempered with good judgment and understanding of the consequences of liquefaction-induced ground movement.

Recommendations

At most sites, it is prudent to use more than one *in situ* technique to evaluate liquefaction potential. In light of uncertainties with all methods, multiple techniques can increase confidence in the conclusions if the methods predict the same general outcome, or highlight the level of uncertainty if they do not. Either way, a better decision can be made based on all other factors impacting such a decision (reliability of data, representativeness of data and samples, impacts/ consequences of liquefaction, costs of modification, etc.).

At least one of the techniques should include a means to provide samples of the soils being evaluated. Samples allow for visual classification and laboratory testing of physical properties without having to infer these properties from indirect *in situ* measurements such as CPT sleeve friction. This implies that CPT, BPT, and V_s measurement, none of which results in samples, should not be used as a sole means of evaluation at a site. They should be complemented by SPT or other sampling methods (borings or test pits, for example).

In addition to the *in situ* methods listed above, consider whether in-place density testing would be an economical and useful source of data for the specific conditions at the site where liquefaction potential is to be evaluated. For any dam-safety modification (or other construction) where suspected liquefiable materials will be exposed by excavation, include provisions in the specifications to permit numerous in-place density tests during the course of excavation. Where there are *in situ* test data for comparison, density measurements can be extremely valuable in advancing the understanding of *in situ* techniques for evaluation of liquefaction potential.

SPT and CPT are, in general, the most appropriate techniques for soils with limited gravel content. Measurement of V_s may be used for verification as well as development of shear-wave velocity data for input into response and deformation analyses. The CPT is encouraged, whether as the main tool or as a supplement to the SPT, whenever feasible. In addition to advantages listed earlier in this report, it can also provide shear-wave data by using a seismic cone with only very minor additional time and cost. If CPT is chosen as the primary means of evaluation, a few SPT tests should be added as well to provide samples to ensure that the estimated fines contents or fines corrections in the CPT evaluation reasonably well match actual sample characteristics.

At sites containing gravels, BPT and V_s are generally appropriate, and even SPT may provide useful information when corrected for gravel interference. The latter can provide a means of verifying that BPT interpretations (which might be gathered in much greater volume) are valid for the site in question. When conducting BPT explorations, it is recommended to include a number of Becker sampling holes adjacent to BPT holes to obtain some stratigraphic information and insight into material type. (One must recognize, of course, that those samples will be completely disaggregated and lose water.)

For foundations containing cobbles and boulders, both SPT and BPT are likely to result in unreliably high blow counts. If these very coarse soils have very little fines or fine sand and there is no overlying layer of less pervious material, excess pore pressure may be able to dissipate, preventing liquefaction. Exploratory drilling can be performed to try to locate finer (more vulnerable) layers to evaluate. In these types of deposits, surface geophysics such as SASW should also be considered. Suspension logging should also be considered. In addition, in-place density testing can be performed at the near surface.

In general, one should attempt to limit the use of these empirical investigative procedures to the conditions under which they were developed. For example:

- For both the SPT and CPT, triggering evaluations were based on Holocene age clean sands, level or gently sloping ground, and depths from 3 to 45 feet. Empirical or semi-empirical adjustments are required for other conditions, introducing additional uncertainty.

- The Harder and Seed correlation for estimating SPT N values from BPT blow counts was based on depths less than about 100 feet. It contains an implicit amount of rod friction, which, in effect, varies as a function of blow count only. At greater depths or when overlying materials may cause atypically large or small rod friction, the friction must be accounted for explicitly. Generally, this is done using the Sy and Campanella method, which includes energy measurement by PDA and friction measurements by wave-equation analysis and/or pullback. If, for example, mudded or predrilled holes are used to reduce the friction for easier driving, the reduced friction needs to be accounted for. If a nonstandard bit diameter is used, existing correlations are simply not applicable.

- The V_s procedure for predicting liquefaction potential was developed using data from relatively level ground, uncemented soils of Holocene age, average depths less than about 30 feet, and groundwater depths between 1.5 and 20 feet (with measurements taken below the groundwater level).

Care is needed when conducting investigations and interpreting data from conditions that are outside these limits. This occurs often with large embankment dams. As a result, there are a greater than usual number of uncertainties; interpretation of data and decision making depend heavily on judgment.

It is critical to ensure that a sufficient number of tests or borings are conducted that a reasonably representative sampling of the soils in question is obtained. This becomes even more important when a critical layer is thin (limiting the testing intervals from a single boring) or when the soils are particularly interfingered, lensed, or similarly heterogeneous. For obvious economical reasons, explorations and tests should target the areas deemed most vulnerable to liquefaction.

Because some case histories have indicated significant differences in estimating equivalent $(N_1)_{60}$ blow counts from BPTs by either the Harder and Seed or the Sy and Campanella method, it is prudent to analyze BPT data using both correlations if possible. The Sy and Campanella approach is theoretically more complete because of the measurement of energy transferred to the rods, and its explicit treatment of friction. These issues may be very important at high altitudes, at large depths, with coarse or dense overlying material, etc. The Harder and Seed method allows very rapid data reduction, and is thus of value for quick inspection of data for looser layers requiring further attention.

Inasmuch as Reclamation relies heavily on probabilistic risk analyses for dam safety decisions, the various correlations for estimating the probability of liquefaction as a function of CSR and $(N_1)_{60}$ (or Q_{c1}) are of particular interest. The recent work by Seed and colleagues (2003) appears to be the most comprehensive and up-to-date tool and is suggested for use. However, it is prudent to evaluate other probability correlations as well as a general check on reasonableness and sensitivity to choice of probability model. One must be aware of and account for differences in the way the input data were interpreted in developing the methods (e.g., different fines adjustments, or duration weighting factors).

There are new or existing techniques that are currently of limited use, yet show promise. Some of these include the LPT, MASW, and a large Vibroseis machine being tested by the University of Texas. None of these techniques could not be used as the sole means of evaluating liquefaction potential at a Reclamation dam site until more experience has been gained. However, use of those methods could advance the state of knowledge and provide long-term benefit, as well as benefit specific to the project.

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