Seismic Cone Penetration Testing for Evaluating Liquefaction Potential

by

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Abstract

Impressive progress has been made in the last 25 years in recognizing liquefaction hazards, understanding liquefaction phenomena and analyzing and evaluating the potential for liquefaction at a site. Recent findings related to the application of the seismic cone penetration test (SCPT) for the evaluation of liquefaction potential are presented and discussed. The seismic CPT provides independent measurements of penetration resistance, pore pressures and shear wave velocity in a fast, continuous and economic manner. The current methods available for evaluating liquefaction using penetration resistance are presented and discussed. The important effects of increasing fines content on the penetration resistance are also discussed. Recent developments in the application of shear wave velocity to evaluate liquefaction potential are discussed and a new method based on normalized shear wave velocity is proposed. Limited case history data are used to evaluate and support the proposed correlation. A worked example is presented to illustrate the usefulness of the SCPT for evaluating liquefaction potential at a site.

Key Words: Liquefaction, In-situ tests, Seismic

Introduction

Since the earthquakes of 1964 in Japan and Alaska there has been significant progress in recognizing liquefaction hazards, understanding liquefaction phenomena, and analyzing and evaluating the potential for liquefaction at a site. Currently available methodologies for the evaluation of liquefaction potential can be classified into two categories:

(a) analyses based on *laboratory* testing data, and

(b) analyses based on *in-situ* testing results, as shown in Table 1.

Table 1 also briefly outlines the principal advantages and limitations of each methodology.

Table 1

Summary of Main Laboratory and In-situ Tests Used to Evaluate Liquefaction Potential

Laboratory Tests	Advantages	Limitations
Undrained cyclic testing	Design Earthquake loading on "undisturbed" samples simulated in laboratory	Difficult to account for effects of sample disturbance
Steady State analysis	Steady State parameters determined by tests on "undisturbed" samples in laboratory	Difficult to account for effects of sample disturbance

In-situ Tests	Advantages	Limitations
Standard Penetration Test (SPT)	Large historical data base, soil samples recovered	Equipment variability, infrequent test interval, Variable energy levels
Cone Penetration Test (CPT)	Continuous readings, economic fast and standardized	Limited data base, no samples recovered
Shear Wave Velocity	Can be used at sites difficult to test by the other procedures	Limited data base, no samples recovered
Self-boring Pressuremeter (SBPT)	Test "undisturbed" soils in-situ	No data base, difficult to interpret test data
Flat Dilatometer Test (DMT)	Near continuous readings, economical and standardized	No data base

This paper briefly presents some recent findings related to the application of the seismic cone penetration test (SCPT) for evaluating liquefaction potential. The SCPT combines the piezocone penetration test (CPT) with measurement of shear wave velocity. A small geophone or accelerometer is placed inside a standard 10 cm^2 electric cone and seismic wave velocities are measured during pauses in cone penetration. Figure 1 shows a schematic representation of the SCPT. Full details of the test are described by Robertson et al., (1986). The SCPT is an ideal logging test that combines the excellent capabilities of the CPT with the specific measurement of the seismic velocities (compression P and shear S waves).

Before presenting some of the recent developments in the application of SCPT data to liquefaction evaluation it is worth defining what is meant by liquefaction of soils.

Liquefaction Phenomenon

Granular soils, such as sands, derive their strength from the intergranular effective stress. When these soils are saturated with water, the effective stress is the difference between the total stress (due to the total weight of overburden) and the water pressure in the soil pores. Hence, the strength and deformation properties of granular soils are dependent on the level of porewater pressure.

During cyclic loading induced by earthquake shaking, the grains of soil tend to move to form a denser arrangement. If the water in the pore spaces is unable to drain away to accommodate the compaction, the porewater pressure increases. This decreases the effective intergranular stress and the soil becomes weaker and more deformable. In very loose granular soils, the rise in pore water pressure can be extremely large due to collapse of the soil structure and there can be a very significant loss of strength. This phenomenon has traditionally been referred to as soil liquefaction.

However, in recent years, research has shown that the behavior of granular soils during cyclic loading is more complex. Vaid and Chern (1985) have explored the behaviour of sands at various densities in undrained cyclic loading and have defined three regimes of behaviour as shown on Figure 2. When the sand is contractive (Figure 2(a)), the sand strain softens after the peak deviator stress has been attained and the undrained strength reaches a minimum value which remains constant over a large range in strain. The minimum constant undrained strength is called the steady state or residual strength. This continued flow at constant resistance is called liquefaction or flow liquefaction.

Figure 2(b) shows the response of a sand that is initially contractive but then becomes dilative with increasing strain. This phenomenon is called limited liquefaction.

If the sand is dense, porewater pressures still develop during seismic shaking and may become large enough to eject sand and water and create sand volcanoes or boils. But since the sands are dense they do not undergo flow deformation. However, the porewater pressure does reduce the stiffness of the sand and the strength at small strains. Therefore deformations tend to increase with duration of loading (Figure 2(c)) and may become large enough in some cases to constitute failure. This phenomenon is called cyclic mobility. For the sands that are initially contractive but become dilative with increasing strain cyclic mobility correctly describes their response after the stage of limited liquefaction. The deformation patterns of these sands are shown together in Fig. 2(d).

The magnitude of potential deformations at a liquefied site with contractive sand depends on whether the static driving shear stresses (τ_{st}) are less than or greater than the residual strength (τ_{ss}). If greater, large scale flow deformation may occur. The extent of the deformations depends on the extent to which the driving stress exceed the residual strength.

If the residual strength is greater than the driving shear stresses, large scale deformations will not occur. In this case, the extent of the deformations depends on the duration and intensity of loading. This problem is similar to cyclic mobility. A flow chart showing the conceptual procedures for cyclic loading analyses for these different conditions are illustrated in Figure 3.

Empirical Assessment of Liquefaction Based on Penetration Resistance

Seed and his colleagues (Seed, 1979; Seed et al. 1983, and Seed et al. 1984) developed correlations relating the normalized SPT N-value and the average cyclic stress ratio (τ_{av}/σ_o') to cause liquefaction for an earthquake of magnitude M = 7.5, as shown on Figure. 4. Recent efforts by Seed et al (1983) were directed towards standardizing the SPT to a standard energy level of 60% of the free fall potential energy of the hammer. Hence, the correlation presented in Figure 4 shows the normalized SPT N value corrected to an energy level of 60%, $(N_1)_{60}$.

The curves presented in Figure 4 were based on observed response of predominantly level ground sites during and after earthquake loading for magnitudes around M = 7.5. Sites were considered to have "liquefied" based on observed surface features such as sand boils. To aid in the application of these correlations, Seed et al. (1984) proposed a set of curves that defined the approximate shear strain that could develop during earthquake loading. These curves suggest that for clean sand (mean grain size, D_{50} = 0.25 mm) with $(N_1)_{60} < 10 \sim 15$ liquefaction would produce essentially unlimited deformation (i.e. flow liquefaction). For clean sands with $(N_1)_{60} > 10 \sim 15$ 'liquefaction' would produce limited deformations or cyclic mobility (i.e. shear strains $\gamma <$ 20%).

There have been numerous studies on the correlation between CPT tip resistance and SPT resistance (Douglas et al., 1981; Seed et al. 1983; Robertson et al. 1983). Figure 5 shows that the ratio of cone resistance (q_c) to blow count (N) increases with increasing mean grain diameter. The correlation shown in Figure 5 contains some SPT data where energy measurements were made and corrected to an average level of 60% (Robertson et al., 1983). The correlation shown in Figure 5 therefore, represents an energy level of about 60%. Since, both the CPT and SPT penetration resistances are influenced by the effective overburden stress in essentially the same manner (Baldi et al. 1985) it is proposed to relate normalized cone penetration resistance (q_{c1}) directly with normalized SPT N-value at 60% energy $(N_1)_{60}$ using;

$$q_{c1} / (N_1)_{60} = 5 \qquad \text{for clean sands} \tag{1}$$

The ratio of 5 was selected to represent a conservative value to develop further correlations. The normalized cone penetration resistance q_{c1} is the cone penetration resistance (q_c) normalized for overburden pressure using;

$$q_{c1} = q_c \times \left(\frac{P_a}{\sigma_{vo}'}\right)^{0.5} \tag{2}$$

P_a = atmospheric pressure in the same units as σ_{vo}'
σ_{vo}' = effective overburden stress

Using the liquefaction correlation based on the SPT (Figure 4) it is possible to produce a similar correlation based on the CPT for M = 7.5, as shown in Figure 6. The curves in Figure 6 represent the correlation between normalized cone penetration resistance (q_{c1}) and the average cyclic stress ratio (τ/σ') to cause "liquefaction" for clean sands. The curve is essentially similar to those proposed by Robertson and Campanella (1985), Seed and de Alba (1986), Isihara (1985)and Shibata and Teparaksa (1988). However, they do have the advantage that they include the curves for estimating the amount of deformation.

The correlation shown in Figure 6 suggests that for clean sands with a normalized cone penetration resistance $q_{c1} < 50 \sim 75$ liquefaction would produce essentially unlimited deformation (i.e. flow liquefaction). For clean sands with $q_{c1} > 50 \sim 75$ liquefaction would produce limited deformations (i.e. shear strain $\gamma < 20\%$) or cyclic mobility.

The correlations shown in Figures 4 and 6 applies to earthquakes with a magnitude M = 7.5. The critical correlations for earthquakes of other magnitudes may be established by multiplying the critical cyclic stress ratios by correction factors as suggested by Seed (1979).

The data base for Figures 4 and 6 is limited to sites where liquefaction occurred under effective overburden pressures less than 150 kPa. For overburden pressures greater than this, it is necessary to make an appropriate reduction in the critical cyclic stress ratio (Seed et al., 1983).

The application of Figure 6 can be illustrated by an example. For a magnitude M = 7.5 earthquake, if at a certain depth the average cyclic stress ratio applied by the earthquake were 0.3 and the normalized cone penetration resistance at the same depth for a clean sand were 10 MPa (100 bars), than "liquefaction" would occur and the estimated shear strain would be approximately 10%. Hence, the implication is that flow deformation would not occur but deformations could be significant. If a layer of say 5 m was expected to experience the same shear strain during the earthquake, the accumulated horizontal deformation could be estimated to be approximately 0.5 m.

Identification of Contractive Sands

The empirical correlations based on the work of Seed and shown in Figures 4 and 6 would suggest that clean sands could experience flow liquefaction if triggered when their in-situ state is defined by the following normalized penetration resistance values;

$(N_1)_{60} \leq 10$ to 15

$q_{c1} \leq 50$ to 75

For flow liquefaction to occur the sands must be highly contractive during shear and the static driving shear stresses must exceed the residual (steady state) strength. Therefore, these normalized penetration resistance values can be used as a guide to identify potentially contractive sands, as required in the flow chart in Figure 3.

Recent work (Been and Jefferies, 1985; Been et al., 1986; Been et al., 1988; Sladen et al., 1985 and Sladen and Hewitt , 1989) has directed considerable attention to the problem of identifying contractive, potentially liquefiable clean sands based on CPT penetration resistance. The work by Been et al. (1986) was based on laboratory calibration chamber studies using the state parameter concept. Negative state parameter values imply dilative behaviour during shear. Hence, a state parameter (ψ) equal to zero implys the start of contractive behaviour. Been et al.(1986) showed that the cone penetration resistance could be normalized linearly with respect to the mean effective stress and correlated to the state parameter (ψ). The data presented by Been et al. (1986) suggest that contractive clean sands (i.e. $\psi = 0$) can be defined by the following range of penetration resistance

$$\frac{q_c - p_o}{p_o'} \leq 10 \text{ to } 45$$

where p_o = mean total stress

p_o' = mean effective stress.

The application of this normalization process requires the knowledge of the horizontal effective stress, usually in the form of K_o. Unfortunately, in most preliminary investigations K_o is unknown. However, if the objective is to identify the potential existence of very loose sands it is often sufficient to assume $K_o \approx 0.5$ as a first estimate. Also, for most onshore investigations involving the CPT the total overburden stress (p_o) is small relative to the penetration resistance (q_c), hence it is reasonable to use the much simpler normalization of q_c/σ_{vo}'. Therefore, based on the state parameter approach the following approximate normalized penetration resistance values can be used to identify loose, contractive sands;

$$q_c/\sigma_{vo}' \leq 7 \text{ to } 30$$

The extensive calibration chamber studies performed in Italy (Jamiolkowski et al.1989) suggest that this range is too low and that a normalized penetration resistance of about $q_c/\sigma_{vo}' = 40$ can be used to identify loose, contractive (i.e. non-diletive) sands. Robertson (1986) suggested that contractive sands could be identified from correlations between CPT penetration resistance and friction angle. Sands with a friction angle equal to their constant volume friction angle (ϕ'_{cv}) would be contractive. Hence, using the CPT friction angle correlation suggested by Robertson and Campanella (1983) the following normalized penetration resistance can be used to identify clean, silica sands that are contractive:

$$q_c/\sigma_v' < 40 \text{ to } 50$$

The work by Sladen and Hewitt (1989) involved the back analyses of several failures of hydraulically placed sand used for construction of artificial islands in the Beaufort Sea. Based on these observations Sladen and Hewitt (1989) suggest a relationship between CPT penetration resistance and effective overburden stress to define loose, contractive sands that can be approximated using:

$$q_c/(\sigma_{vo}')^{0.65} = 70$$

Figure 7 compares the relationships based on the state parameter or dilation approach with that proposed by Sladen and Hewitt (1989). Also included is the relationship inferred from the work by Seed, using Figure 6.

Figure 7 shows that the observations by Seed are remarkably similar to those of Sladen and Hewitt (1989) and that at shallow depth (low overburden stress) the relationship based on the state parameter approach appears to be less conservative. These relationships apply to clean sands with limited fines content, their application to sand with a high fines content remains questionable.

Fines Content

Seed et al. (1983) reviewed additional field data from sites with increasing amounts of fines content and developed the correlations shown in Figure 8. The field data in Figure 8 shows a strong dependency of liquefaction resistance on fines content. Surprisingly, there appears to be considerable confusion about the effects of fines content on resistance to cyclic loading. Troncoso (1990) compared the cyclic strength of tailings sands with different silt contents ranging from 0 to 30% at a constant void ratio and found that the cyclic strength decreased with increased fines content. However, Kuerbis and Vaid (1989) studied the effects of fines using samples deposited using a slurry technique in a manner that approximately replicated many field deposition conditions. They found for the sand tested that up to 20% fines could be accommodated within the sand skeleton and that samples of the same sand skeleton void ratio had the same cyclic strength for a non-plastic fines content less than 20%. The work by Kuerbis and Vaid (1989) suggests that provided the essentially non-plastic fines fill the sand skeleton voids the resistance to liquefaction can remain unchanged. Clearly there is still much research required to fully understand the effect that fines and their associated plasticity can have on the fabric and hence behaviour of sands.

From the viewpoint of penetration testing, when there is an increase in fines content there is often a substantial decrease in measured penetration resistance. This rapid decrease in penetration resistance is probably due to the combined effects of increased soil compressibility and decreased drainage during penetration. Since penetration resistance is a large strain measurement it is sensitive to changes in soil compressibility. Hence, the penetration resistance can show a marked decrease with increasing fines content. Also as the fines fill the sand skeleton voids there is a decrease in permeability which moves the penetration process from drained to undrained. This change in drainage is especially noticeable for cone penetration since the penetration rate is constant.

The determination of seismic shear wave velocities is a small strain measurement and generally appears to be insensitive to small additions of non-plastic fines content. This again is probably a feature of the fines filling the sand skeleton voids and it is the sand skeleton that controls the seismic shear wave velocity. Figure 9 presents a summary of the influence of fines content on penetration resistance, cyclic loading and shear wave velocity.

It has been common practice to correct the measured penetration resistance to an equivalent clean sand value based on the type of data presented in Figure 8. Such corrections have been suggested by Seed et al. (1984), Ishihara (1985), Tokimatsu and Yoshimi (1983), Olson and Farr (1986) and Shibata and Teparaksa (1988). However, these corrections can become very large especially when the measured penetration resistance is small. Therefore, penetration resistance becomes a very insensitive parameter to evaluate liquefaction resistance in soils with a large fines content. This can be illustrated by the data in Figure 8 where normalized SPT $(N_1)_{60}$ values of between 2 and 3 were recorded in soils with a fines content of 35% or greater. Hence, the correction to an equivalent clean sand penetration resistance would be about 8, i.e. the correction is 3 to 4 times larger than the measured value. Corrections of this magnitude clearly reflect the insensitivity of the measurement.

It is possible to estimate the fines content (FC) directly from CPT data. Figure 10 presents a recent soil behavior type classification chart based on normalized CPT data. Soils that fall in zone 6 are generally clean sands or silty sands with small amounts of fines (< 5%). Soils that fall in zone 5 are silty sands and sandy silts and generally have fines contents greater than about 15%.

Attempts have been made (Olsen, 1984; Olsen and Farr, 1986) to incorporate a correction for fines content directly into a CPT based liquefaction chart. However, this approach is highly sensitive to the accuracy of the CPT sleeve friction measurement (fs), which is known to be variable for cones of different design (Lunne et at. 1982).

Improvements to soil classification can be made using pore pressure measurements during the CPT. Penetration pore pressures as well as the rate of dissipation of pore pressures during pauses in the penetration can significantly improve soil classification (Robertson, 1990).

Based on data collected in the Richmond area of British Columbia, Woeller et al. (1989) suggested a correlation for low plastic soils (plasticity index <11%)between fines content and the time for 50% dissipation (t_{50}) of pore pressures during a pause in penetration as shown in Figure 11. The results in Figure 11 suggest that for $t_{50} >$ 50 sec the fines content is generally greater than 40%. For t_{50} between 10 sec and 50 sec the cone penetration process is partially drained and there appears to be no correlation between t_{50} and fines content. The increase in fines content makes the penetration process progressively move from a drained penetration to an undrained penetration, which encourages reduced penetration resistance.

Based on experience in China (Xie, 1979), the major variables that appear to influence the resistance to liquefaction for soils with fines are, the plasticity of the fines and the amount of clay size particles (i.e. percent finer than 0.005 mm). The Chinese (Wang, 1979) suggests the following criteria:

Percent finer than 0.005 mm	< 15%
Liquid Limit (LL)	< 35%
Water Content	> 0.9 Liquid Limit
Liquidity Index	> 0.75

Criteria such as these, that incorporates some measure of the plasticity of the fines appear to be more logical than the application of only fines content.

An example of the usefulness of the Chinese criterion has been described by Robertson et al (1983). A site in New Westminster, B.C. has a layer of uniform, essentially non-plastic silt . The silt was part of a hydraulically placed deposit. The mean grain size of the silt was $D_{50} = 0.025$ mm with a fines content of approximately 80%. However, the percent clay sizes (< 0.005 mm) was essentially zero, the plasticity index was only 8, the liquidity index was essentially 1.0 and the normalized cone penetration resistance (q_{ci}) was only 5. Based on the only undisturbed sample that could be obtained the cyclic stress ratio to cause liquefaction was approximately 0.09. This example illustrates that granular soils with a high fines content can have a very low resistance to liquefaction provided the fines are non-plastic and the amount of clay size particles is small. The penetration resistance in these soils is often very small and existing corrections based on fines content are very large.

Liquefaction Assessment From Shear Wave Velocity

Empirical methods have also been developed to evaluate liquefaction resistance directly from shear wave velocity (Bierschwale & Stokoe, 1984). Over the past 15 years, significant advances have been made in measuring shear wave velocities in the field. Accurate and detailed profiles can be determined with conventional crosshole and downhole seismic methods (Stokoe and Hoar, 1987; Wood, 1987). One of the most significant advances in recent years in the measurement of seismic wave velocities has been the development of the seismic cone penetration test (Campanella and Robertson, 1984; Robertson et al, 1986). Shear wave velocity, V_s is influenced by many of the variables that influence liquefaction, such as soil density, confinement, stress history and geologic age. Thus, V_s has promise as a field index in evaluation liquefaction susceptibility.

The major advantage of using shear wave velocity as an index of liquefaction resistance is that it can be measured in soils that are hard to sample (such as silts and sands) or penetrate (gravels).

Direct Shear Wave Velocity Correlations

The limiting shear wave velocities separating liquefied from non-liquefied sites for a given intensity and duration of shaking must be determined from field data. So far the data base is quite limited but it clearly shows that shear wave velocities can be a useful index of liquefaction potential. Data from sites in the Imperial Valley, California, which liquefied during the 1979 Imperial Valley, 1981 Westmorland and 1987 Superstition Hills earthquakes suggest that the limiting shear wave velocity separating liquefiable from non-liquefiable sites is about 140 m/s for earthquakes of local magnitude $M_L = 6.5$ generating peak ground accelerations of about 0.17g. Extensive liquefaction occurred in the Imperial Valley, California during the 1979 Imperial Valley and the 1981 Westmorland earthquakes (Youd and Wieczorek, 1984).

Shear wave velocity profiles determined at six sites which liquefied in either the 1979 Imperial Valley or 1981 Westmorland earthquakes are shown in Figure 12. The soils ranged from silty sands (SM) to sandy silts (ML). As can be seen from Figure 12, the soils that liquefied all have shear wave velocities less than 140 m/s (460 ft/sec). If only the 1981 Westmorland earthquake with a moment magnitude M = 5.6 is considered, then only site 1 through 4 liquefied and sands at these sites exhibit shear wave velocities less than 125 m/s (410 ft/sec) in the top 6 m (20 ft) of depth. The shear wave velocity of a silty sand at one site along Heber Road (site 6) which did not liquefy during the 1979 earthquake is also shown on Figure 12. This sand has a shear wave velocity greater than 150 m/s (500 ft/sec). Therefore for the given moment magnitude of the earthquake (M = 6.6) and distance from the fault, the limiting shear wave velocity to prevent liquefaction is about 140 m/s (460 ft/sec). A direct correlation between limiting shear wave velocities separating liquefiable from non-liquefiable sites based on the Imperial Valley data are shown on Figure 13 (Bierschwale and Stokoe , 1984). This data is based on calculated maximum surface accelerations on adjacent firm ground.

The Imperial Valley data of liquefaction and shear wave velocity represents a unique data base to develop direct correlations, such as that shown in Figure 13. However, it would appear that some improvements can be made over the existing observations.

Laboratory research (Hardin and Drenevch, 1970) has shown that shear wave velocity is a function of void ratio and effective confining stress. Hence, for a sand of constant void ratio (i.e. constant density) the shear wave velocity will increase with increasing depth. Hence, a correlation between V_s and cyclic stress ratio (τ/σ_v') to cause liquefaction should be based on shear wave velocities normalized with respect to effective overburden stress, similar to the manner penetration resistance is normalized with depth. Research (Hardin and Drnevich, 1972) has shown that the shear wave velocity is approximately a function of the effective overburden stress to the power 0.25. Therefore, a normalized shear wave velocity (V_{s1}) can be determined using the following expression:

$$V_{s1} = V_s \; (P_a / \sigma_v')^{0.25} \qquad (3)$$

Where

V_{s1} = normalized shear wave velocity

P_a = reference stress, typically 100 kPa (1 bar)

σ_v' = effective overburden stress in same units as P_a

V_s = measured shear wave velocity

The Wildlife site in the Imperial Valley was fully instrumented in 1982 following the 1981 Westmorland earthquake, in the hope of obtaining detailed quantitative data for a liquefied site during the next earthquake. The 1987 Superstition Hills earthquake ($M = 6.6$) in the Imperial Valley caused extensive sand boils, ground cracks and lateral movement associated with liquefaction at the Wildlife site (Holzer et al, 1988).

Four separate earthquakes during November 23-24, 1987 (Table 2) triggered the instrument array at the site: the first main shock ($M = 6.2$), a small aftershock, the second main shock ($M = 6.6$) and one of it's aftershocks. Only the second main shock, with peak horizontal accelerations of 0.21 and 0.17 g at the surface and downhole respectively, caused long-term excess porewater pressures at the array. Excess porewater pressures were not recorded during the first main shock which generated a peak surface acceleration of 0.13 g. It appears from preliminary studies of the data that porewater pressures did not develop during the larger main shock until the peak acceleration had exceeded 0.17 g. More detailed studies of this event are still in progress.

TABLE 2

Earthquakes That Triggered The Wildlife Liquefaction Array
(Holzer et al, 1988)

Event	Magnitude	Date (1987)	Time (Pacific Standard Time)	Peak Horizontal Surface Acceleration
1	6.2 (M)	23 Nov	17:54	0.13g
2	4.0 (M_L)	23 Nov	22:23	<0.05g
3	6.6 (M)	24 Nov	05:16	0.21g
4	4.8 (M_L)	24 Nov	05:35	0.02g

The data seems to suggest that, for an earthquake of moment magnitude M = 6.6 generating peak accelerations of 0.17g, potentially liquefiable sites would require a minimum shear wave velocity of about V_S = 140 m/s to prevent the generation of pore pressures and liquefaction. This represents a normalized shear wave velocity in the range of 120 m/s to 150 m/s over the depth of 3 m to 6 m.

It is interesting to note the continuing susceptibility of the Wildlife site to successive earthquakes. Despite suspected liquefaction of the entire silty sand layer, it was not possible to identify major physical changes within the deposit. In April 1988, cone penetration tests were conducted by the US Geological Survey (USGS) near pre-earthquake (1982) test locations to search for physical changes. Neither the detailed stratigraphy nor the inferred porosity of the silty sand changed significantly between 1982 and the time of the post-earthquake studies in 1988, a result consistent with the small volume of material deposited on the surface by sand boils. Thus, it is believed the deposit continues to be susceptible to liquefaction following the 1979 Imperial Valley and the 1981 Westmorland earthquake. Indeed it is interesting that the 1981 earthquake ($M = 5.6$) could reliquefy the site after it had liquefied previously during the 1979 earthquake which had a magnitude $M = 6.6$. Field soils that develop large strains due to liquefaction do not usually develop increased resistance to subsequent liquefaction.

Figure 14 presents a proposed correlation between normalized shear wave velocity (V_{S1}) and τ/σ' to cause liquefaction based on the observation of liquefaction at Imperial Valley. The data from the Wildlife Site in the Imperial Valley represents a unique set of information since it represents the point at which liquefaction just occurred and therefore should represent the boundary between liquefaction and no liquefaction in any correlations. The 1987 Imperial Valley earthquake was for a $M = 6.6$ and can be converted to a $M = 7.5$ using the values suggested by Seed, (1979). The shape of the proposed correlation shown in Figure 14 was based on the additional Imperial Valley data presented by Bierschwale & Stokoe (1984) and shown in Figure 13 but where the shear wave velocity is corrected for overburden pressure.

To evaluate the correlation shown in Figure 14, additional published data has been included. Although the additional data are somewhat limited they show surprising agreement with the Wildlife data.

It is interesting to note that fines content appears to have little or no effect on the correlation between normalized shear wave velocity and τ/σ' to cause liquefaction. Whereas, when the same Imperial Valley data is processed in terms of penetration resistance ($(N_1)_{60}$ and q_{c1}) fines content has a significant influence. Based on the experience with the SPT (Figure 4) and the proposed correlation using normalized shear wave velocity, it may be possible to identify potentially contractive sands and silty sands using shear wave velocity. It would appear that a normalized shear wave velocity of less than about 120 m/s may represent potentially contractive sands.

CASE HISTORY

The Geological Survey of Canada is involved in a multi-stage geophysical and geotechnical research program in the Frasre Delta of British Columbia. The seismic piezocone has been used in the study to investigate the liquefaction potential of selected sites using the procedures outlined above (Finn, Woeller and Robertson, 1989a; Finn et al., 1990; Finn, Robertson and Woeller, 1990). Figure 15 shows the soil profile at one of these sites in terms of CPT cone bearing (penetration resistance), sleeve friction, friction ratio and penetration pore pressure. The friction ratio is the ratio of sleeve friction to cone bearing expressed in percent.

The seismic CPT data were obtained using a standard 10 sq cm electric cone with a 60° apex cone angle. Pore pressures were measured immediately behind the cone tip using a 5 mm thick porous plastic element saturated with glycerin. Data were collected every 5 cm and stored in a data acquisition system. Seismic shear wave velocity measurements were made every 1 m during brief pauses in the cone penetration. Full details of the test procedures are given by Robertson et al.(1985).

The profile shown in Figure 15 is reasonably representative of many areas in the Fraser River Delta. The profile consists of about 4 m of a clayey silt overlying a sand and silty sand to a depth of about 16 m. The sand deposit is highly variable both in density and grain size. The sand from approximately 12 m to 16 m appears to be quite clean, whereas, the sand from 4 m to 12 m is quite silty. Occasional silt lenses exist at depths of 5.8 m, 7.3 m, 9.2 m, 11.3 m, 12.3 m, 13.0 m, 14.0 m and 15.4 m. The soil from 16 m to 21 m appears to be a silt with interbedded sand lenses. Below 21 m is an extensive deposit of essentially normally consolidated clayey silt. Below 26 m the clayey silt is interbedded with sand layers.

Figure 16 shows the measured shear wave velocity profile using the seismic CPT obtained during the same sounding shown in Figure 15. The potential for liquefaction can be assessed by comparing the cyclic stress ratio induced by the design earthquakes with the critical cyclic stress ratio determined from CPT penetration resistance or shear wave velocity (Figures 6 and 14).

Figure 17 shows the average cyclic stress ratio profiles for earthquakes with a maximum surface acceleration of 0.17 g and 0.22 g compared with the critical cyclic stress ratios derived from Figure's 6 and 14 using the measure CPT penetration resistance and shear wave velocity profiles shown in Figure's 15 and 16. A correction for fines content has been made for the CPT penetration resistance according to the recommendations of Robertson and Campanella (1983) and Seed et al. (1984).

Liquefaction is expected to occur whenever the cyclic stress ratio of the earthquake exceeds the critical stress ratio defined by the in-situ test data (q_{c1} or V_{s1}). Based on the CPT data the soil above 4 m and below 21 m was considered non-liquefiable. The normalized shear wave velocity (V_{s1}) independantly predicts essentially the same result. The two test methods predict essentially similar "liquefaction" response in that sediments may "liquefy" to a depth of 21 m except for a few stiffer layers.

The CPT penetration resistance (q_{c1}) provides greater detail of soil variability then the shear wave velocity (V_{s1}), since q_{c1} is collected every 5 cm compared to every 100 cm for V_{s1}. The q_{c1} responds well to dense zones such as that between 14 m to 16 m, whereas the V_{s1} responds in a more subdued manner with less detail. In softer more fine grained zones q_{c1} decreases rapidly as the process becomes undrained (see

Figure 15) and penetration resistance becomes a less sensitive parameter. On the other hand, V_{s1} appears to be less influenced by fines content and provides a somewhat more reliable measure of liquefaction resistance in fine grained soils.

In highly interbedded soils, q_{c1} may not attain its true value in thin dense layers due to the proximity of adjacent softer layers. The shear wave velocity will tend to average the soil response over the 100 cm interval. Therefore, in complex soil profiles like that shown in Figure 15 the overall interpretation for liquefaction potential can be greatly assisted by the combined information of q_c and V_s.

SUMMARY

Although there is, at present, a limited data base to evaluate liquefaction potential from cone penetration resistance, a reasonable estimate of liquefaction potential can be made for most clean sand deposits using correlations such as that shown in Figure 6. The identification of very loose contractive sands can be accomplished using the cone profile suggested by Sladen and Hewitt (1989) and shown in Figure 7. The application of this criteria to sands at depth ($\sigma_{vo}' > 200$ kPa) and for sands with appreciable fines content is uncertain. For cone penetration into silty sands a correction is required based on % fines content. An estimate of % fines content can be made using CPT data and recent soil behavior type classification charts (Figure 10) and correlations with the rate of pore pressure dissipation (Figure 11). However, for soils with a large amount of fines (>35%) the cone penetration process rapidly becomes undrained and the penetration resistance becomes an insensitive parameter to evaluate liquefaction resistance. For these soils it is important to obtain samples to determine the Atterberg Limits and the percent clay size (% finer than 0.005mm), as suggested in the Chinese criteria (Wang, 1979). This, however, does not imply the CPT is not a valuable in-situ test. The CPT provides a fast, economic, continuous measure of soil variability and a reliable, profile of penetration resistance. The additional CPT measurements (sleeve friction, penetration pore pressures and rate of pore pressure dissipation) can be used to make a reasonable estimate of soil type and % fines content. Then, based on this extensive data, samples can be taken at **selected** locations to verify important details. Since most CPT work in Canada is performed using drill rigs, the combined operation of CPT and limited, selective sampling can be highly cost effective. The logic of using SPT data alone with all its associated uncertainty regarding procedures and energy levels is questionable.

The recent development of the seismic piezocone (SCPTU) offers the added advantage that the independent measurement of shear wave velocity can also be used to evaluate liquefaction. This can be very beneficial in silty sands since the shear wave velocity appears to be independent of fines content. A new correlation has been proposed that uses normalized shear wave velocity (Figure 14) to estimate the critical cyclic stress ratio to cause liquefaction. Based on limited available data from sites that have been known to have liquefied this new correlation shows promise. It is hoped that additional data can be collected and processed in the form of normalized shear wave velocity to further evaluate the proposed correlation.

It may also be possible to identify potentially contractive silty sands using normalized shear wave velocity. Based on the proposed correlation shown in Figure 14, a normalized shear wave velocity of less than about 120m/s may be used to define clean sands that are potentially contractive behaviour during shear.

If, based on the SCPTU, liquefaction is considered likely, then an estimate of the amount of resulting deformation is often required. A crude estimate of deformation can be made using the empirical limiting shear strain curves shown in Figures 4 and 6. For a more detailed evaluation it is necessary to determine the residual shear strength (τ_{SS}). Robertson (1990) suggested that there appears to be no unique correlation between penetration resistance and τ_{SS} for all sands, but that the correlation proposed by Seed (1984) probably represents a conservative lower bound.

An important issue that still requires further research and understanding is the influence of fines content and plasticity of fines on soil response due to montomic and cyclic loading as well as in-situ testing results.

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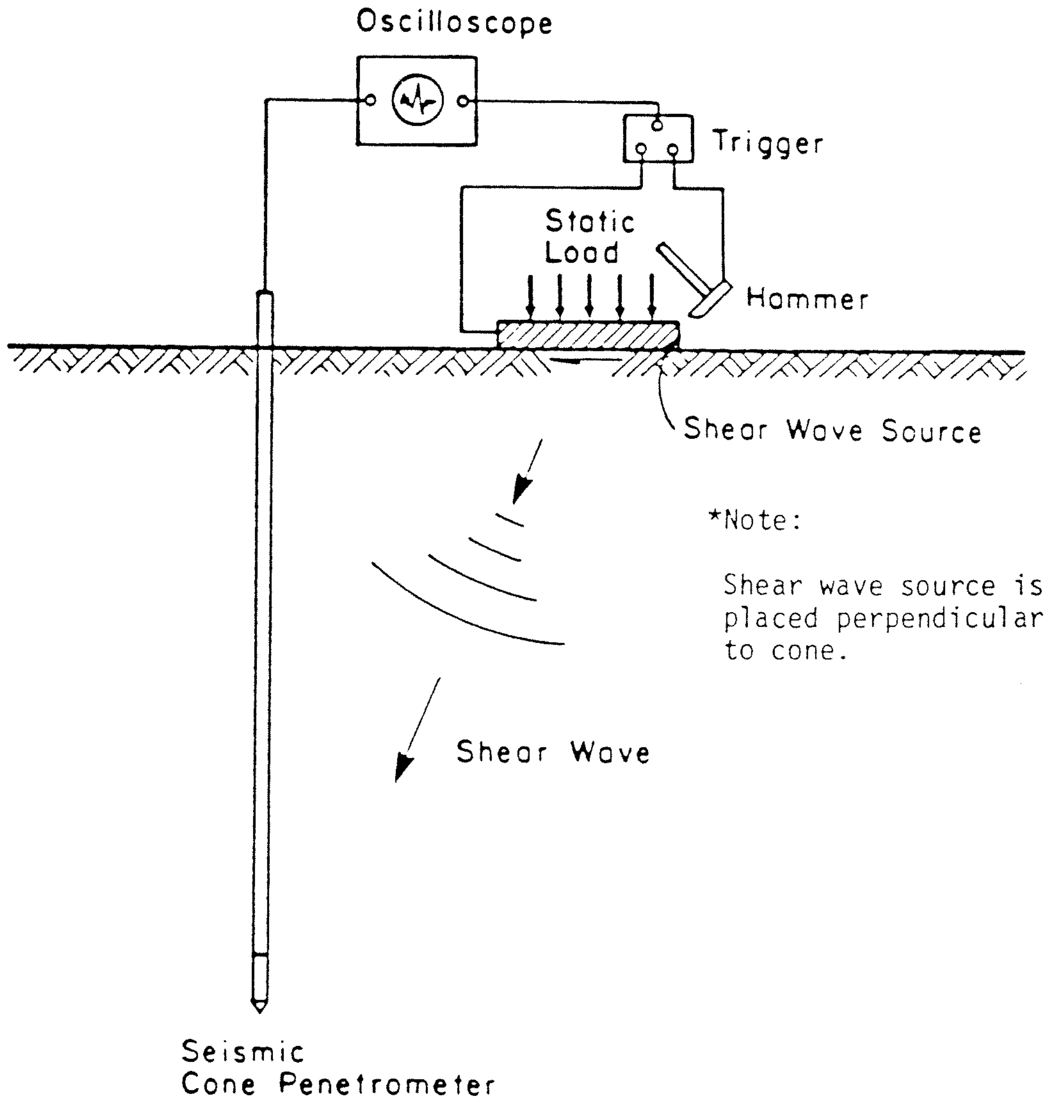


Figure 1. Schematic of Seismic Cone Peneration Test

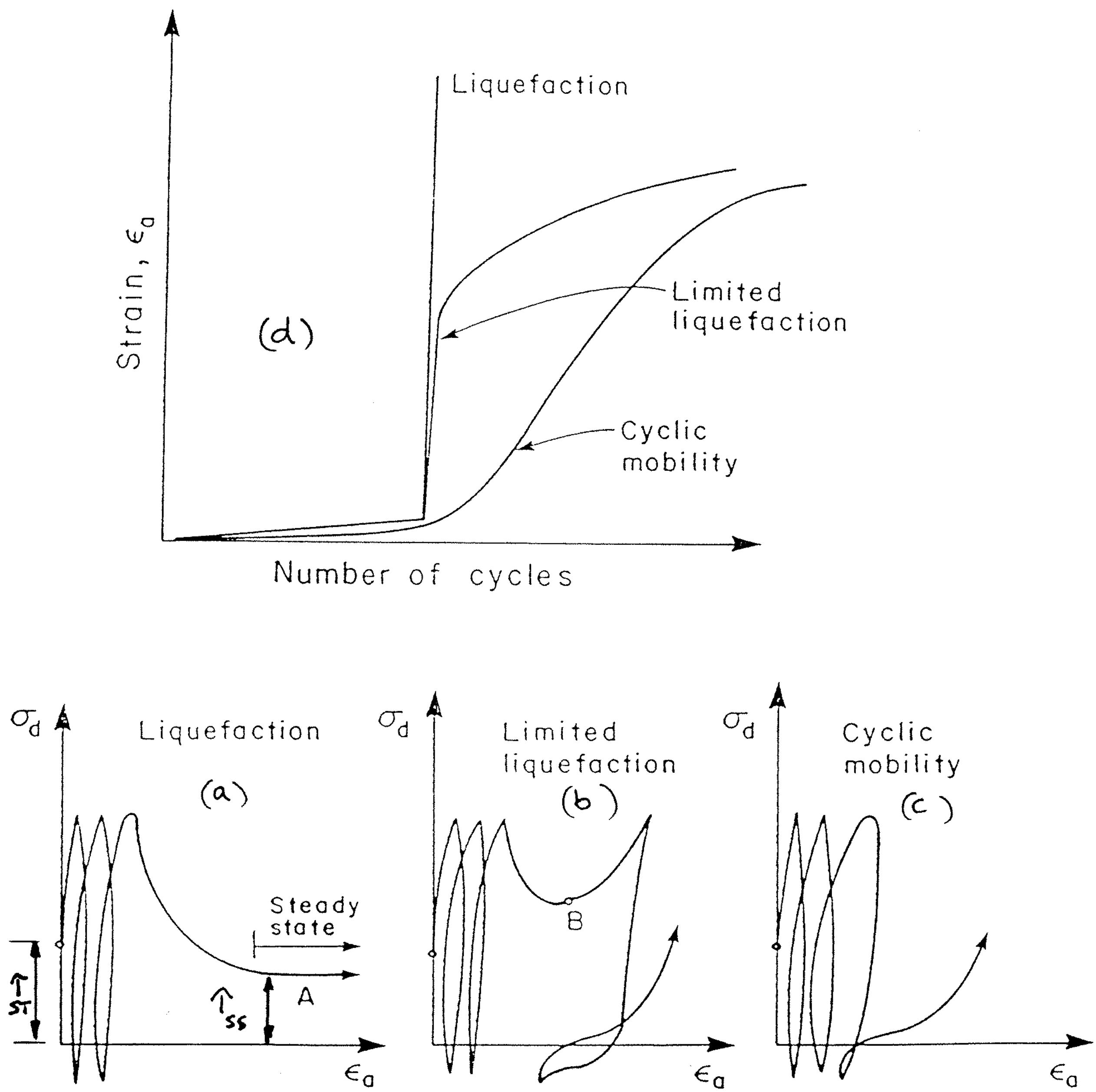


Figure 2. Definition of Liquefaction (After Vaid and Chern, 1985)

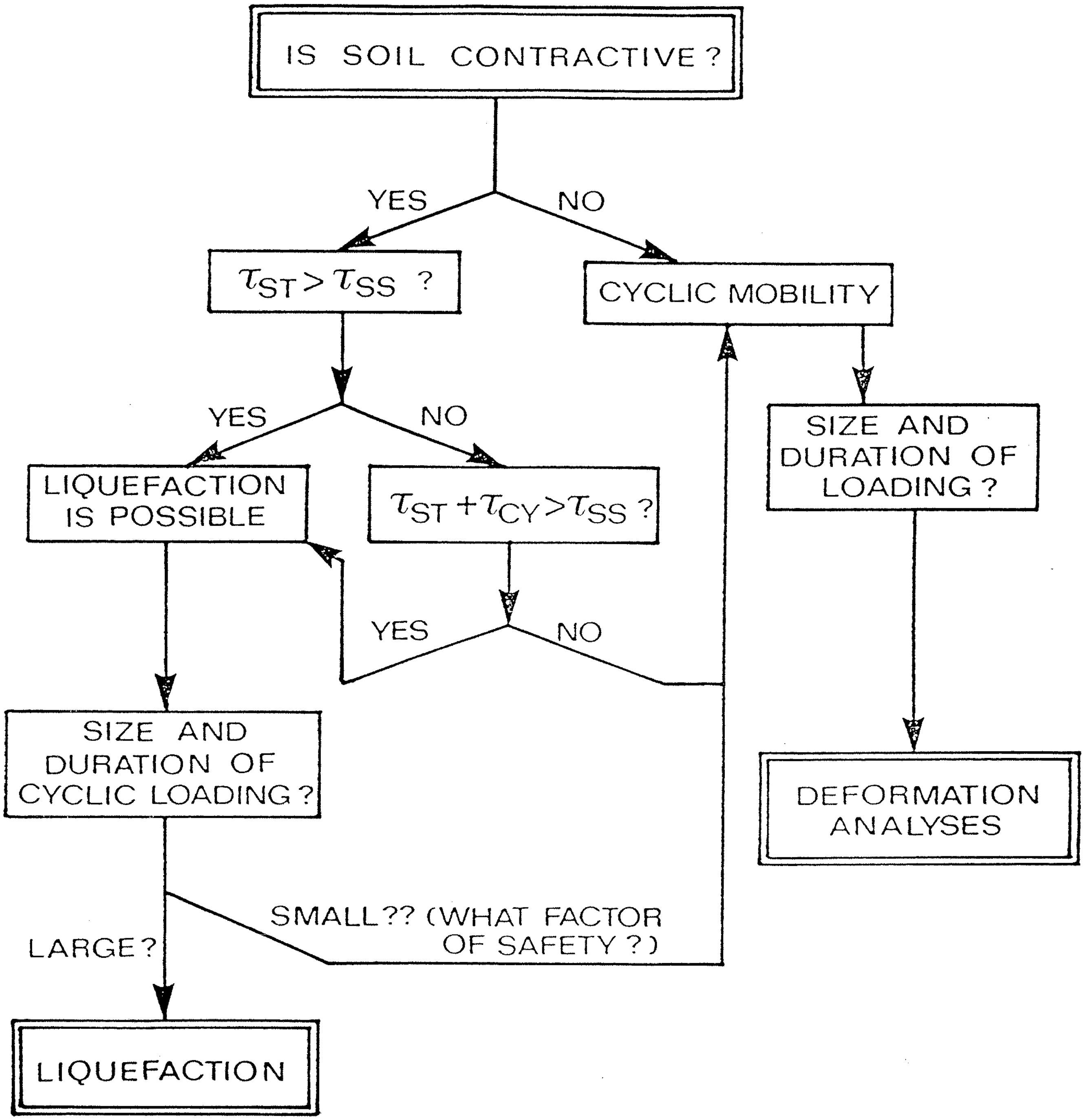


Figure 3. Schematic flow chart for liquefaction analysis

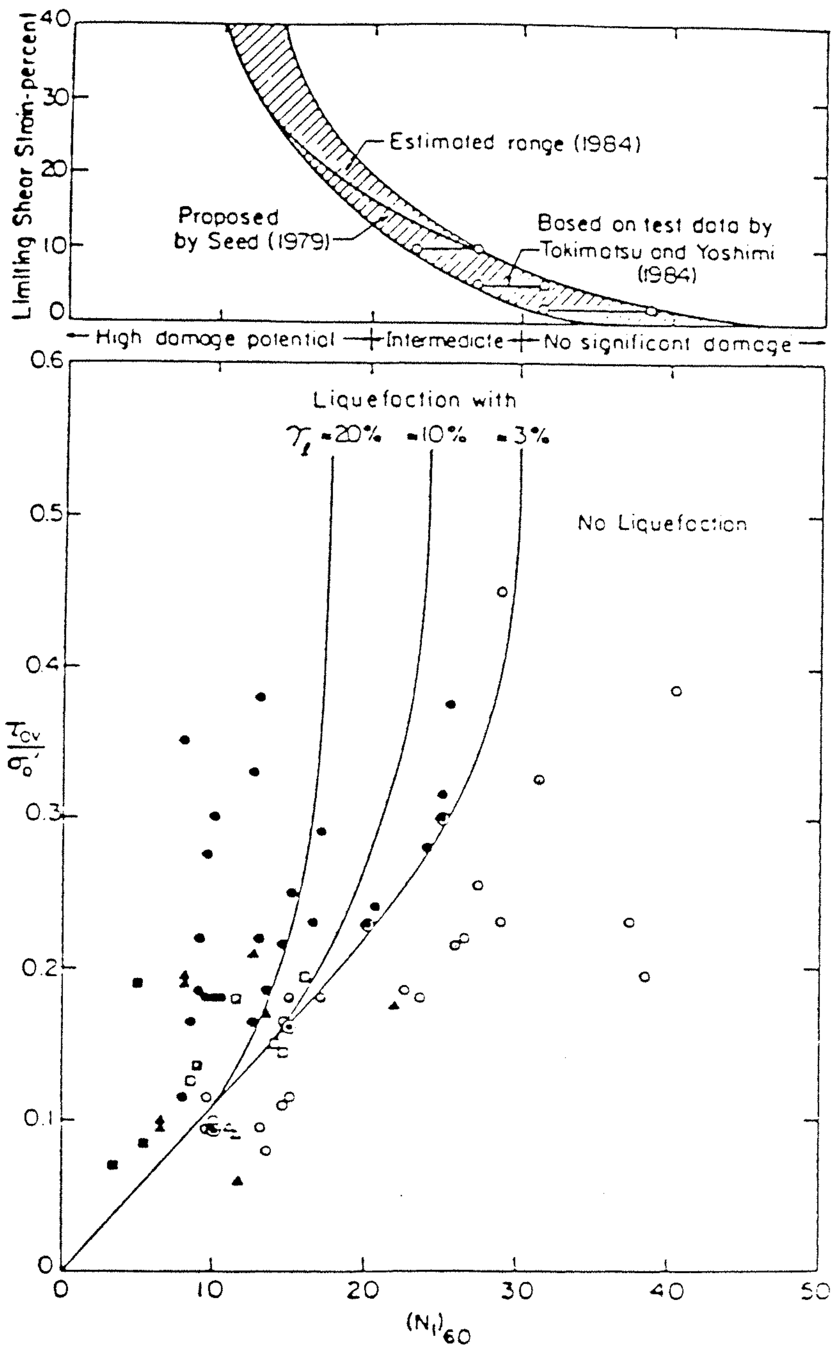
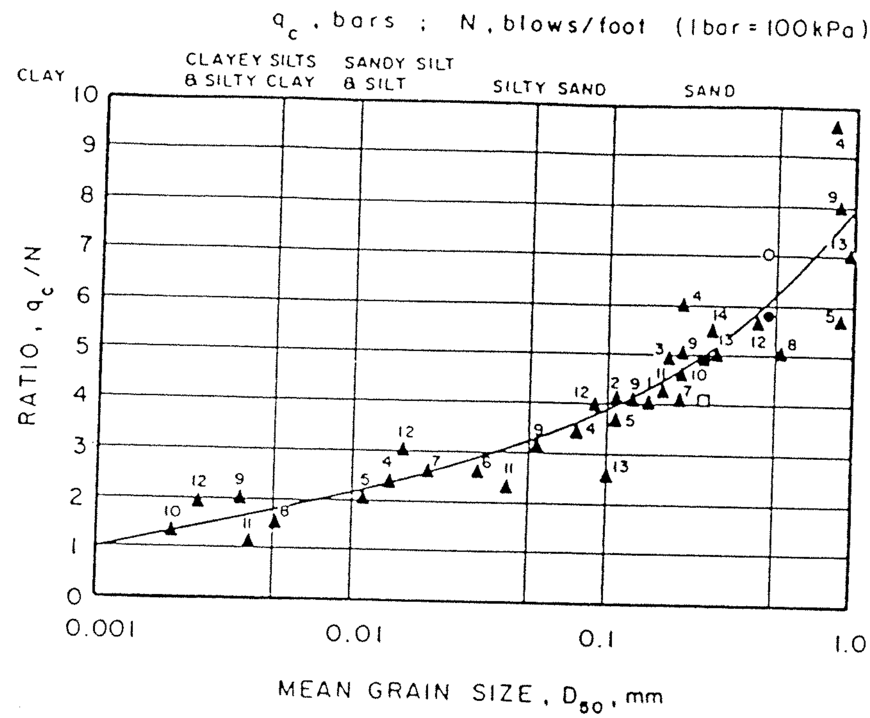


Figure 4. Evaluation of deformation due to earthquake loading using the SPT (After Seed et al., 1984)

BASED ON ENERGY RATIO OF 60% (N_{60})



1. Meyerhof (1956)
2. Meigh and Nixon (1961)
3. Rodin (1961)
4. De Alencar Velloso (1959)
5. Schmertmann (1970)
6. Sutherland (1974)
7. Thornburn (1970)
8. Campanella et al. (1979)
9. Nixon (1982)
10. Kruizinga (1982)
11. Douglas (1982)
12. Muromachi & Kobayashi (1982)
13. Goel (1982)
14. Ishihara and Koga (1981)

TILBURY ISLAND SITE: □ SPT N , ER_i = 47% ; ■ SPT N_c , ER_i = 55%

UBC SITE, McDonald's Farm: ○ SPT N , $ER_i \approx$ 65% ; ● SPT N_c , ER_i = 55%

Figure 5. Variation of q_c / N with mean grain size
(After Robertson et al., 1983)

CORRELATION FOR "CLEAN SANDS"

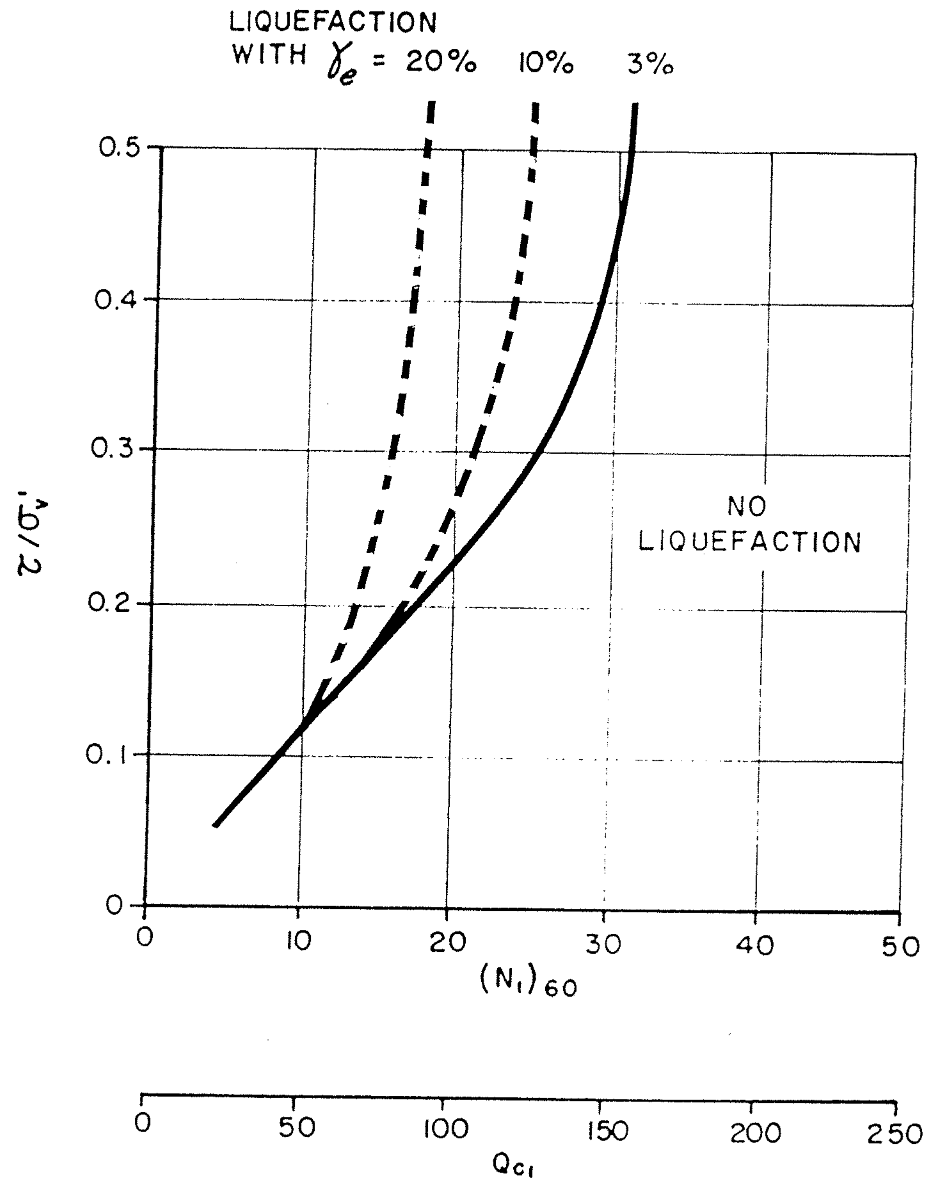


Figure 6. Evaluation of liquefaction potential from the CPT
(Modified from Seed et al., 1983)

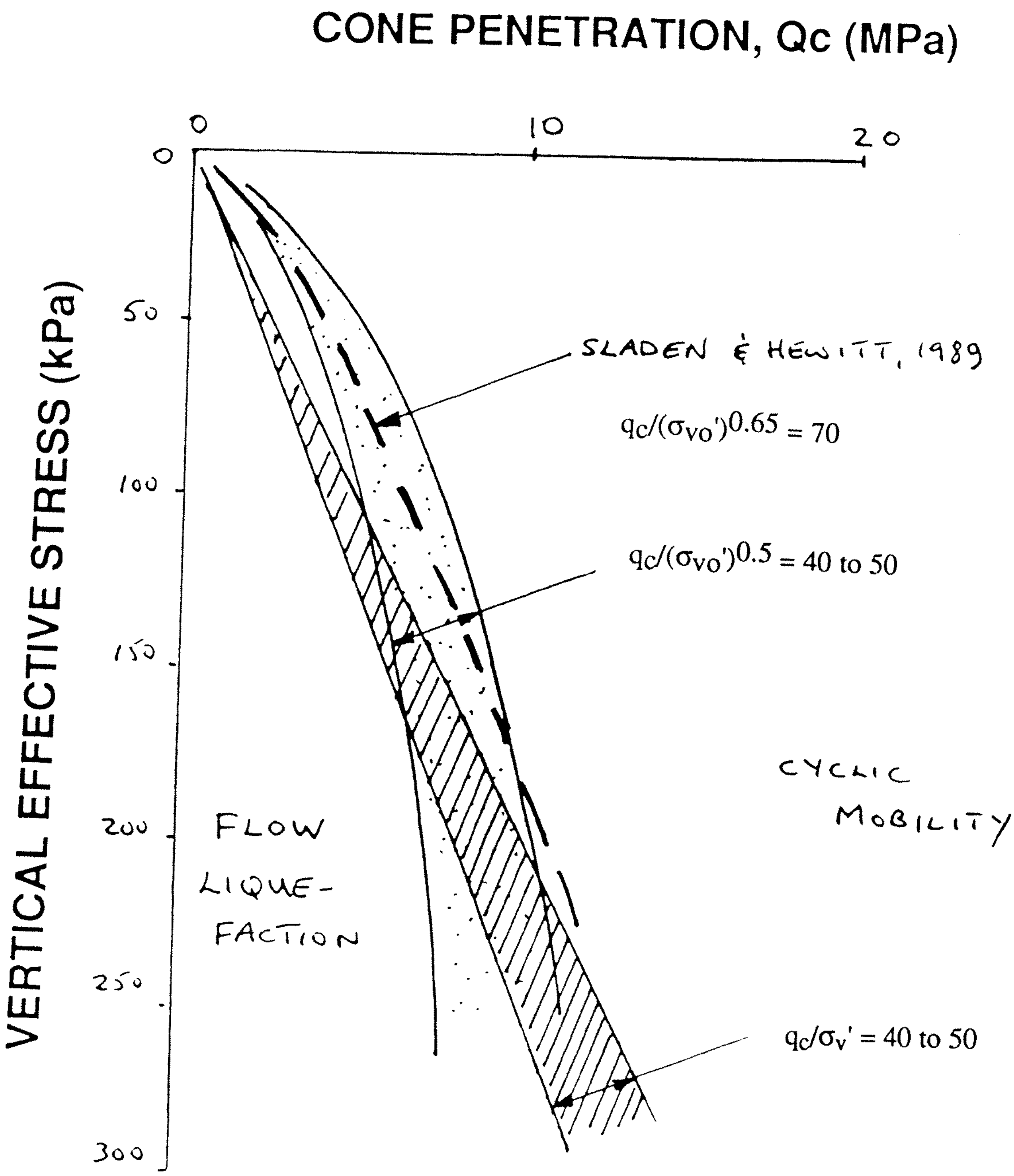


Figure 7. Comparision of CPT penetration profiles to define contractive behaviour for clean sand

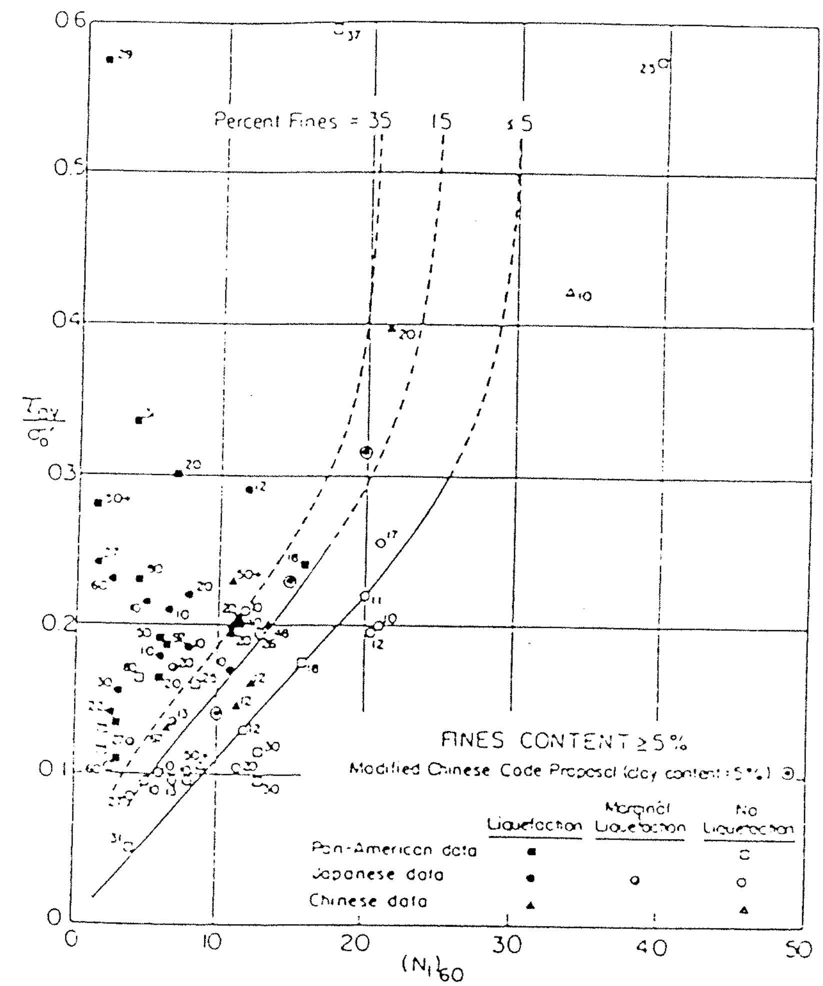


Figure 8. Influence of fines content on the evaluation of liquefaction potential from the SPT (After Seed et al., 1984)

INFLUENCE OF FINES CONTENT

<u>PENETRATION RESISTANCE</u> - *LARGE STRAINS*

- INCREASED COMPRESSIBILITY, HENCE DECREASED PENETRATION RESISTANCE
- FUNCTION OF % FINES, MINERALOGY AND FABRIC

<u>RESPONSE TO CYCLIC LOADING</u> - *SMALL STRAINS*

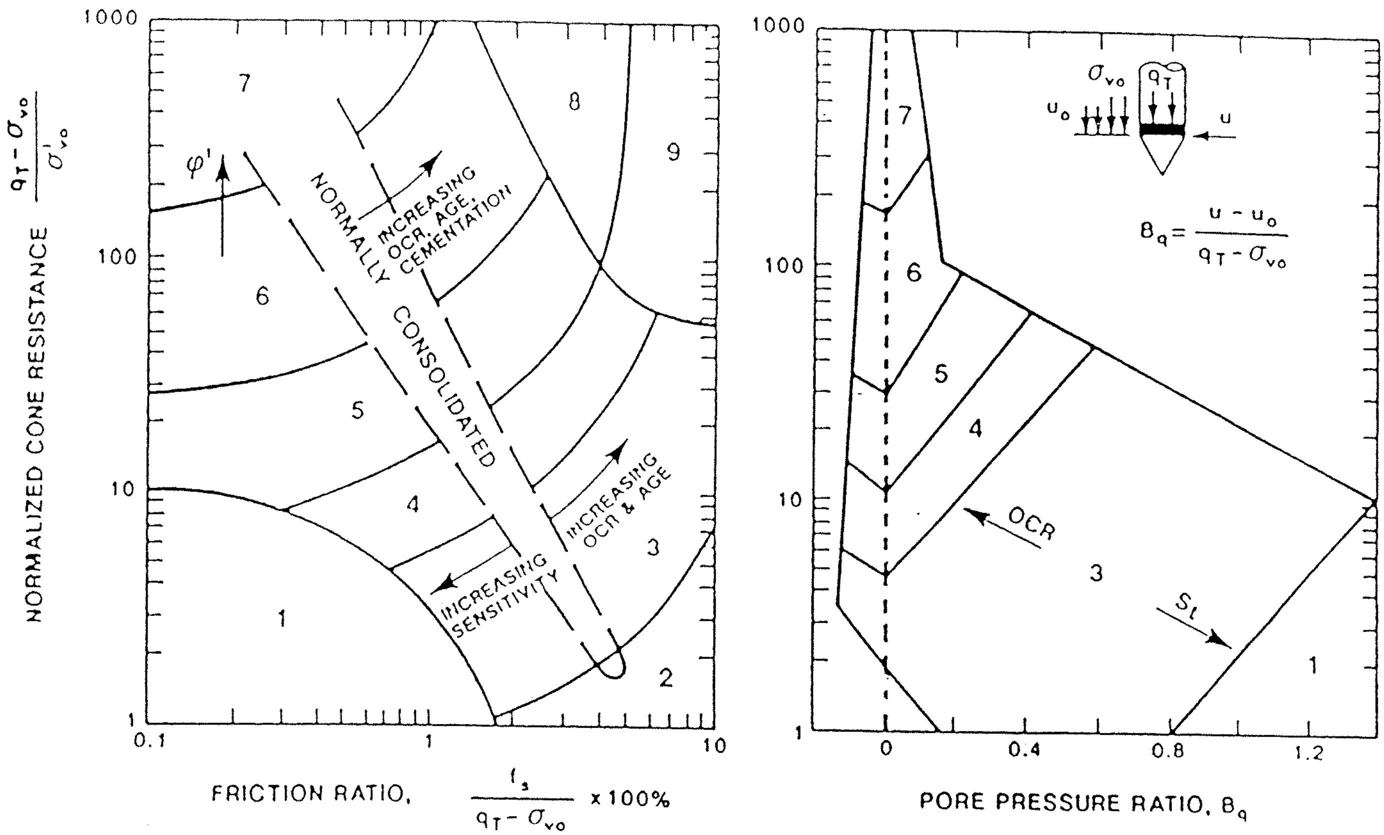
- NO CHANGE OR SLIGHTLY INCREASED / DECREASED RESISTANCE

<u>SEISMIC SHEAR WAVE</u> - *SMALL STRAINS*

- NO CHANGE OR SLIGHTLY INCREASED / DECREASED VELOCITY

Figure 9. Schematic outline of the influence of fines content

q_c, f_s and σ_{vo}' in bars or tsf



1. SENSITIVE FINE GRAINED
2. ORGANIC SOILS - PEATS
3. CLAYS - CLAY TO SILTY 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 SAND
8. VERY STIFF SAND TO CLAYEY* SAND
9. VERY STIFF FINE GRAINED*

(*) HEAVILY OVERCONSOLIDATED OR CEMENTED

Figure 10. Soil behaviour type classification charts for CPT
(After Robertson 1990)

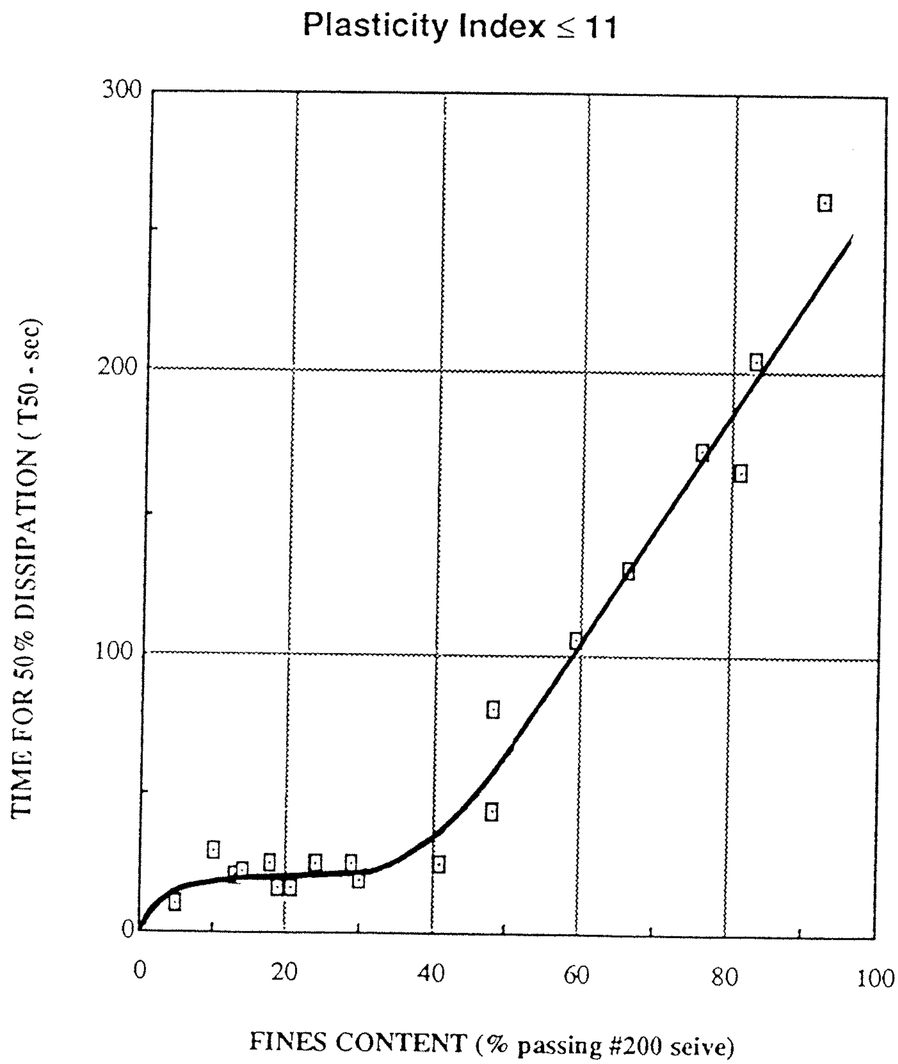


Figure 11. Correlation between fines content and 50% dissipation time for 10 cm^2 CPT

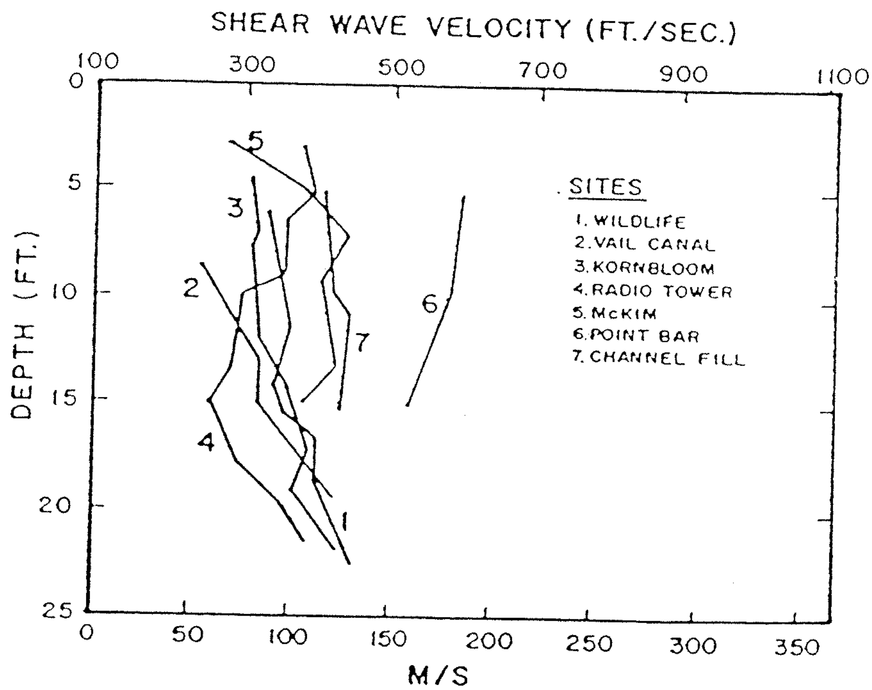


Figure 12. Profiles of shear wave velocities of sandy soils in Imperial Valley (After Stokoe, 1988)

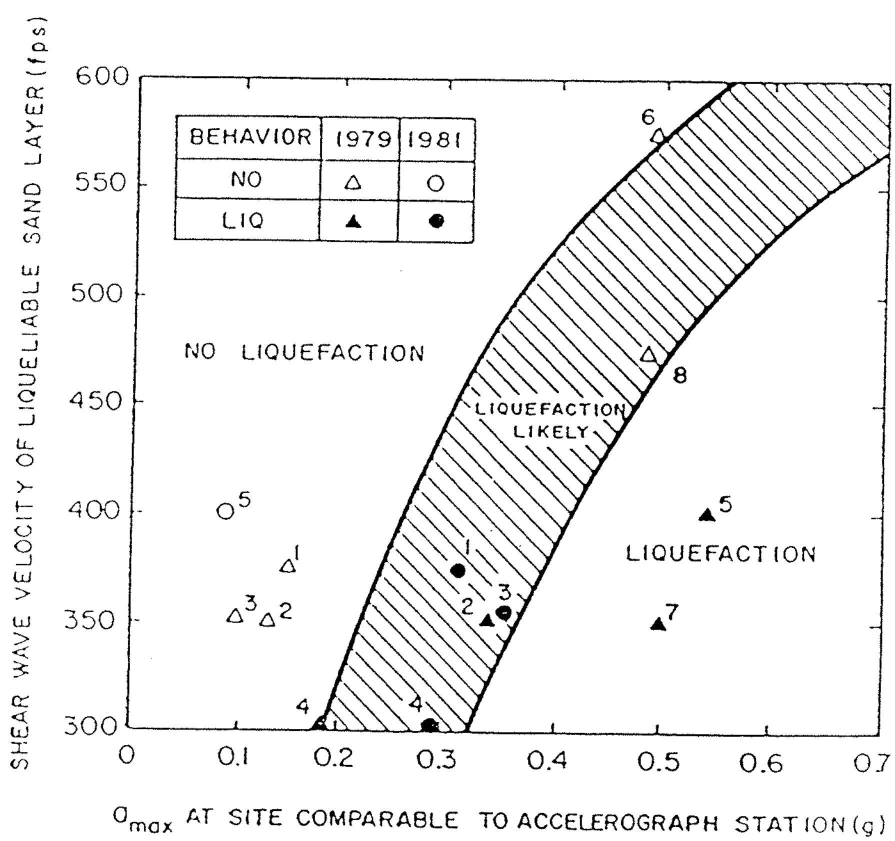
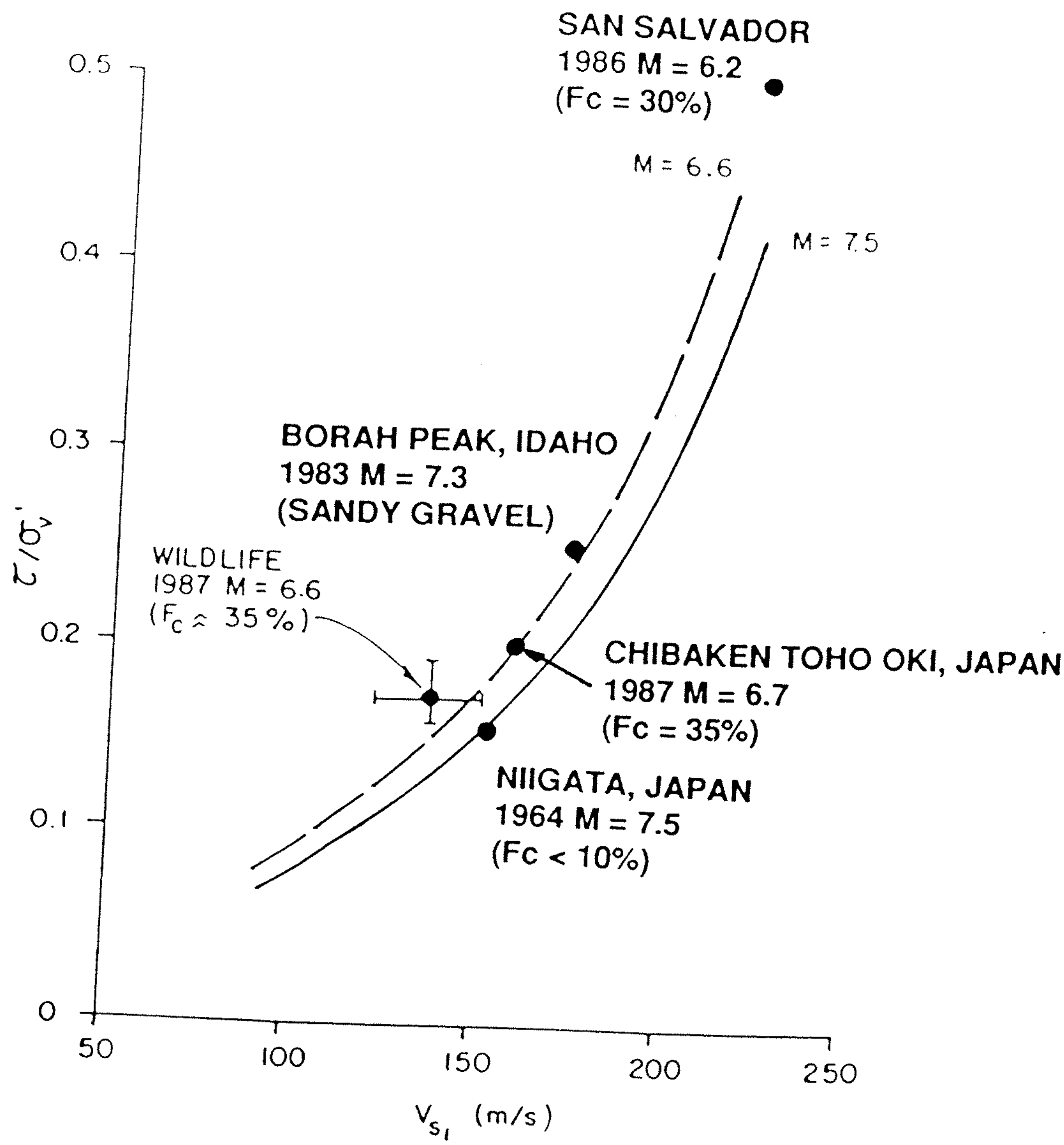


Figure 13. Correlation between shear wave velocity and liquefaction potential of sands based on Imperial Valley data (After Bierschwale and Stokoe, 1984)



$$V_{s_1} = \frac{V_s}{\left(\sigma'_{v_o}\right)^{0.25}} \left(P_a\right)^{0.25}$$

V_s in m/s, σ_{vo}' in bars or tsf

Figure 14. Proposed correlation between normalized shear wave velocity and cyclic stress ratio to cause liquefaction

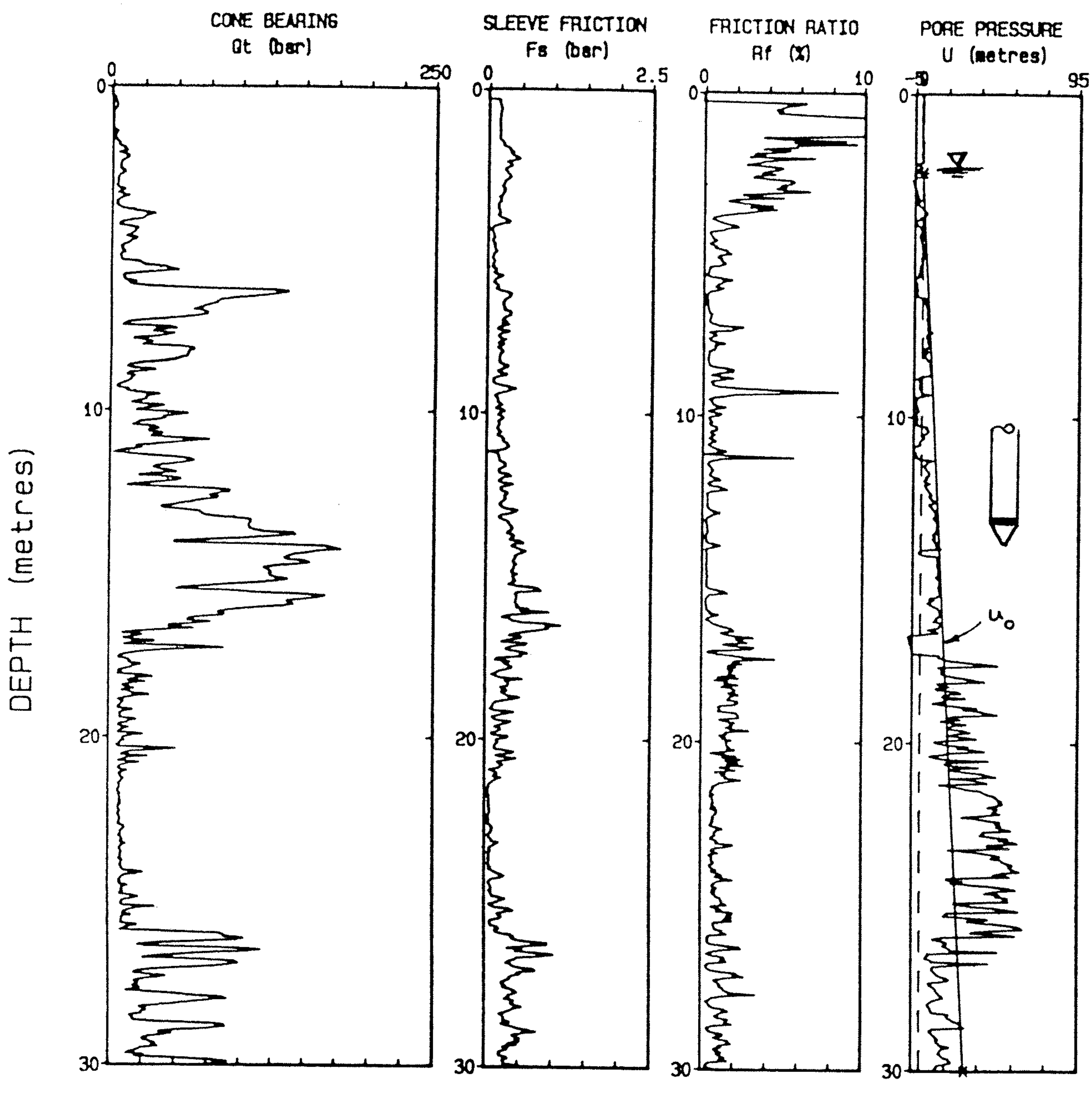


Figure 15. CPTU profile for site in the Fraser River Delta Site (After Finn et al., 1990)

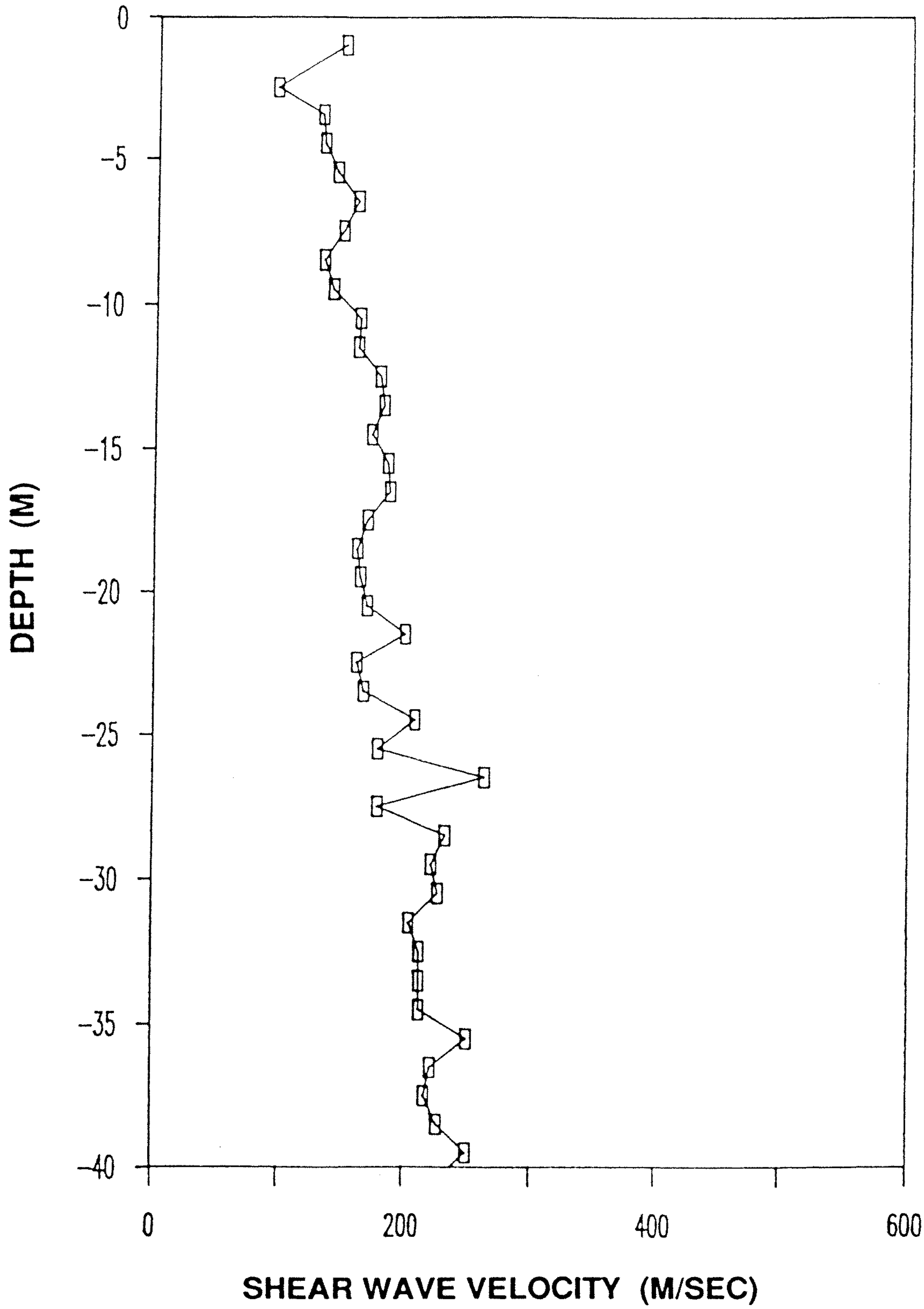


Figure 16. Shear wave velocity profile from seismic CPTU at site in the Fraser River Delta Site

(After Finn et al., 1990)

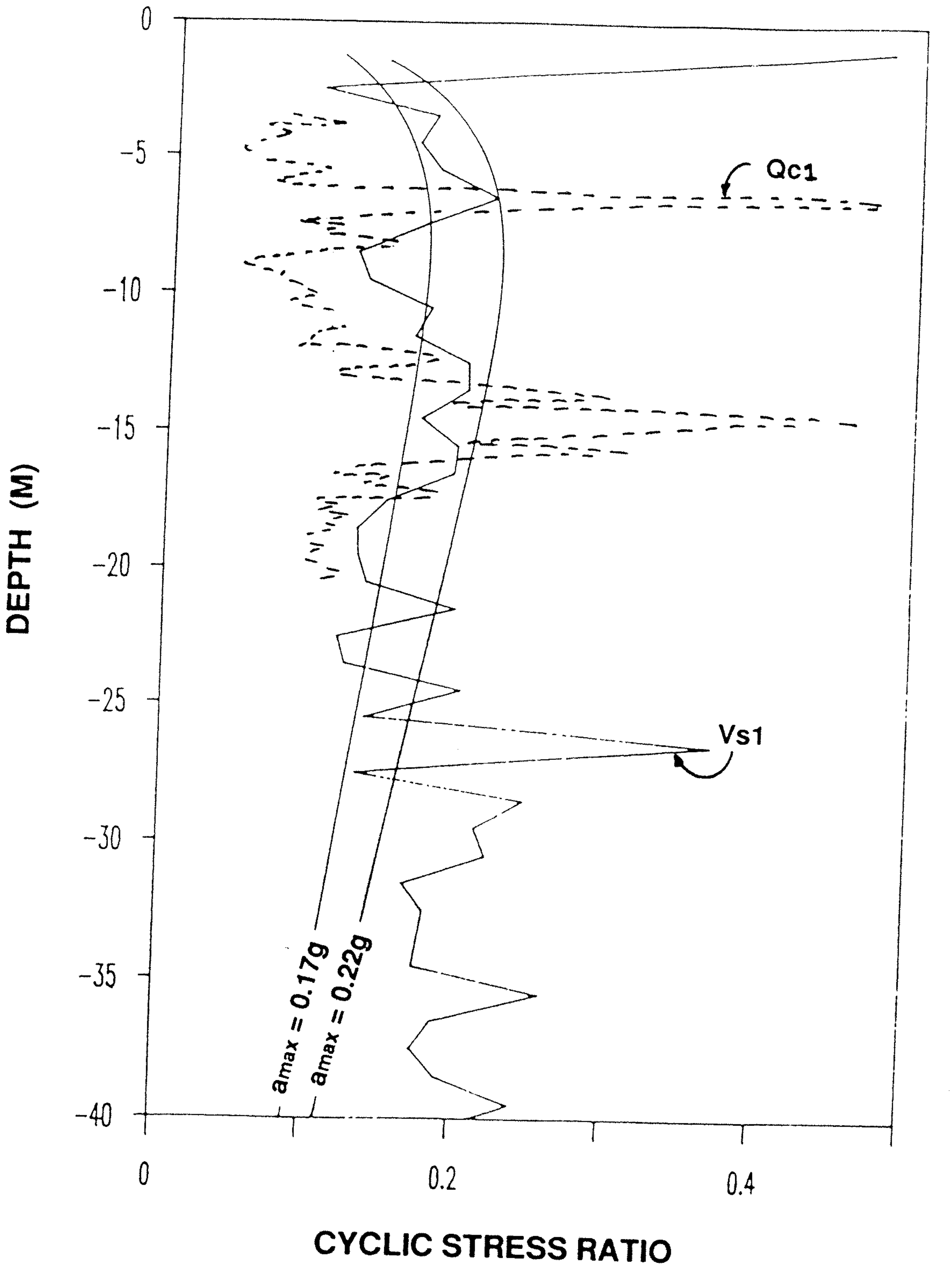


Figure 17. Comparison of predicted response to earthquake loading for site in Fraser River Delta using CPT penetration resistance and shear wave velocity method.

(After Finn et al., 1990)