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Estimation of minimum undrained shear strength for flow liquefaction using the CPT

P.K. Robertson
University of Alberta, Edmonton, Alb., Canada

ABSTRACT: Sensitive clays, metastable silts and very loose sands can strain soften following earthquake loading resulting in possible instability. This instability is often referred to as flow liquefaction. The minimum undrained shear strength is an important parameter in a stability analysis for soils than can strain soften during undrained shear. However, the estimation of this minimum undrained shear strength is often difficult, especially for sandy soils. Research has shown that the undrained shear strength of soils is usually a function of direction of loading, with compression loading often stronger than simple shear and triaxial extension. For many conditions in practice simple shear often represents the average direction of loading. A method is proposed to estimate the minimum undrained shear strength of soils in simple shear using the CPT.

1 INTRODUCTION

Sensitive clays, metastable silts and very loose sands can strain soften following earthquake loading resulting in possible instability. This instability is often referred to as flow liquefaction. Several case histories exist where slopes have failed and flowed due to the strain softening response of the soils. In general, flow liquefaction failures are not common; however, when they occur, they can take place rapidly with little warning and are often catastrophic. Hence, design against flow liquefaction should be carried out cautiously.

If a soil is strain softening in undrained shear, flow liquefaction is possible if the soil can be triggered to collapse and if the gravitational shear stresses are larger than the resulting minimum undrained shear strength. The trigger can be either monotonic or cyclic. Whether a slope or soil structure will fail and slide will depend on the relative amount of strain softening soil to strain hardening soil within the structure, the brittleness of the strain softening soil, the geometry of the ground and drainage conditions.

The minimum undrained shear strength ($s_{u(min)}$) is defined as the minimum strength after undrained strain softening occurs and can be an important parameter in any stability analysis for soils that can strain soften during undrained shear. In this study, the undrained shear strength of interest will be referred to as the minimum strength following strain softening, so as not to confuse the value with the residual strength of some clays associated with particle rearrangement following very large strain. In some clays, the undrained shear strength can drop to very low values, due to slippage between clay platelets. In this study, the focus is primarily on sandy and silty soils in which strain softening can occur due to the loose arrangement of particles resulting in a structural collapse of the grain structure and resulting high pore pressures leading to strain softening and low undrained shear strength. This response in sandy soils is referred to as flow liquefaction.

The objective of this paper is to describe a new screening technique to estimate the minimum undrained shear strength using the Cone Penetration Test (CPT). The new technique builds upon existing CPT methods for clean sands and extends them to silty sands, silts and some clays.

2 MINIMUM UNDRAINED SHEAR STRENGTH

2.1 *Direction of loading*

Research has shown that the undrained shear strength of most soils is a function of direction of loading (e.g. Bjerrum, 1972). In general, the undrained shear strength in triaxial compression (s_{uTC}) is larger than that in simple shear (s_{uSS}) which is larger than that in triaxial extension (s_{uTE}); i.e. $s_{uTC} > s_{uSS} > s_{uTE}$. The appropriate value of the undrained shear strength for a given project will be a function of the geometry and resulting direction of loading. Case histories indicate that the undrained shear strength in simple shear loading can often represent the average undrained shear strength for most projects (Bjerrum, 1972; Yoshimine et al., 1998). Hence, the undrained shear strength in simple shear loading is often the key parameter, although all projects should be evaluated based on their actual geometry.

2.2 *Stress normalization*

For normally consolidated clays, the undrained shear strength increases approximately linearly with increasing vertical effective stress and hence, it is common to define the undrained strength in terms of an undrained strength ratio, s_u/σ'_{vo}. For normally consolidated clays, this undrained strength ratio is approximately constant, depending on soil plasticity, direction of loading and soil density. For simple shear loading, the undrained strength ratio for normally consolidated clay is typically between $s_u/\sigma'_{vo} = 0.2$ and 0.3.

For granular soils, Stark and Mesri (1992) suggested linking the Standard Penetration Test (SPT) $(N_1)_{60}$ with the minimum undrained strength ratio, $s_{u(min)}/\sigma'_{vo}$. Yoshimine et al.. (1998) presented a review of laboratory test results on Toyoura sand and showed that, for loose Toyoura sand at a constant relative density over a low stress range ($\sigma'_{vo} < 300$ kPa), the minimum undrained strength ratio in simple shear loading is approximately constant. Hence, it appears appropriate to link a measure of relative density in loose sand with a constant value of undrained strength ratio. However, this link may not hold for denser sands and at higher stress levels. Hence, caution should be used when applying this type of relationship when granular soils are under a vertical effective stress greater than about 300 kPa.

2.3 *Brittleness*

The possibility of instability in undrained shear is also linked to the brittleness or sensitivity of the soil. Recent laboratory testing on sands has shown a link between the brittleness and the minimum undrained shear strength ratio (Yoshimine et al. 1998). When the minimum undrained strength ratio decreases below about 0.3 the brittleness increases. When the undrained strength ratio is less than about 0.1 the brittleness is usually high.

3 EXISTING METHODS FOR ESTIMATING UNDRAINED SHEAR STRENGTH

There are many methods for estimating the minimum undrained shear strength of soils from in-situ tests. The existing methods tend to be limited to either clay or sand, but are rarely for both soil types. The following is a brief review of the methods available.

The field vane test is often used to measure both the peak and minimum undrained shear strength in clay soils. Wroth (1984) showed that the field vane undrained shear strength is close to that in simple shear. The CPT has been used to estimate the peak and minimum undrained shear strength of clay soils through empirical correlations. Typically, the peak undrained shear strength is estimated using:

$$s_{u(peak)} = \frac{(q_t - \sigma_{vo})}{N_{kt}} \tag{1}$$

where:

q_t is the total cone penetration resistance corrected for unequal end area effects

σ_{vo} is the total overburden stress

N_{kt} is an empirical cone factor.

Typically the cone factor, N_{kt}, that links the CPT to the field vane or simple shear undrained shear strength is between 10 and 20, with an average of about 15 (Lunne et al., 1997).

The minimum (residual) undrained shear strength ($s_{u(min)}$) in clays is often assumed to be equal to the CPT sleeve friction, f_s, since the clay is almost fully remolded as it passes the friction sleeve. Hence, clay sensitivity can be estimated from the CPT using:

$$\text{Sensitivity, } S_t = \frac{s_{u(peak)}}{s_{u(min)}} = \frac{(q_t - \sigma_{vo})}{f_s \cdot N_{kt}} \tag{2}$$

The normalized friction ratio suggested by Robertson (1990) is defined as:

$$F = [\frac{f_s}{(q_t - \sigma_{vo})}] \times 100 \text{ in percent} \tag{3}$$

Hence, clay sensitivity can be estimated from the CPT using:

$$S_t = \frac{100}{F.N_{kt}} \qquad (4)$$

With N_{kt} typically between 10 to 20, this becomes:

$$S_t = \frac{10}{F} \text{ to } \frac{5}{F} \qquad (5)$$

In clean sands, the minimum undrained shear strength is often estimated using empirical correlations with penetration resistance from the Standard Penetration Test (SPT). The most commonly used correlations are those proposed by Seed and Harder (1990) and Stark and Mesri (1992). These correlations were based on case histories where instability occurred and the average minimum undrained shear strength was back calculated. A recent re-evaluation of these case histories (Wride et al., 1998) has questioned the validity of the proposed correlations, especially for $(N_1)_{60} > 10$.

Yoshimine et al. (1998) suggested correlations between the minimum undrained shear strength values in triaxial compression, simple shear and triaxial extension and the normalized cone penetration resistance, q_{t1}, for clean sands. The normalized cone resistance was defined as:

$$q_{t1} = \left(\frac{q_t}{P_{a1}}\right)\left(\frac{P_{a2}}{\sigma'_{vo}}\right)^n \qquad (6)$$

where:

P_{a1} is atmospheric pressure in the same units as q_t

P_{a2} is atmospheric pressure in the same units as the vertical effective stress, σ'_{vo}

n is a stress exponent, typically equal to 0.5 for clean sands.

The correlations were initially based on high quality laboratory results on both reconstituted and undisturbed samples. However, the correlations were evaluated using three case histories of flow liquefaction failures in essentially clean sands where CPT data were available. Yoshimine et al. (1998) found that the case histories agreed well with the laboratory data in simple shear, as shown in Figure 1. The data from Yoshimine et al. (1998) suggests that the critical normalized cone resistance in clean sand above which strain softening in simple shear is unlikely is about 60. The data used to compile Figure 1 were based primarily on sub-rounded to sub-angular quartz sands for which the vertical effective stress was less than about 300 kPa. Ishihara (1993) proposed a similar relationship between normalized CPT and minimum undrained shear strength based on other case histories in sands.

4 PROPOSED CPT SCREENING METHOD

A new method to estimate the minimum undrained shear strength will be described that builds upon existing methods. If a CPT based method is to be applied over a wide range of soil types, the data must be normalized to correct for effective overburden stress in such a way to fit most soils. In the following sections, first a new method of stress normalization is proposed and then the new CPT-based method of estimating minimum undrained shear strength is described.

4.1 Stress normalization of CPT data

There has been much discussion in the literature about the correct normalization of CPT penetration resistance. Wroth (1984) suggested a linear normalization for the interpretation of undrained shear strength in clays, as follows:

$$Q = \frac{(q_t - \sigma_{vo})}{\sigma'_{vo}} \qquad (7)$$

Extensive field experience and theoretical work (Lunne et al., 1997) has shown that this normalization works very well in clay soils to link cone resistance to undrained shear strength ratio, s_u / σ'_{vo} (see Equation 1).

Baldi et al. (1982), suggested a non-linear normalization to link relative density to CPT penetration resistance using the normalized resistance given in Equation (6) using a stress exponent n = 0.5. In most cases, the cone resistance, q_t, is much larger than the total overburden stress, σ_{vo}, and hence, $(q_t - \sigma_{vo})$ is approximately equal to q_t. However, to be consistent, it is recommended to use the following general relationship:

$$Q = \left(\frac{q_t - \sigma_{vo}}{P_{a1}}\right)\left(\frac{P_{a2}}{\sigma'_{vo}}\right)^n \qquad (8)$$

Where n = stress exponent

P_{a1} is a reference pressure in the same units as q_t and σ_{vo}

P_{a2} is a reference pressure in the same units as σ'_{vo}

As stated above, Wroth (1984) showed that a linear normalization (n = 1.0) should be used for clays.

The linear normalization for clays is effective because clays typically have a steep critical state line in void ratio – log mean normal effective stress space (e – log p'), where the slope of the critical state line (λ) is often around 0.4. For sands, the critical (steady) state line is typically flatter in e – log p' space with a slope (λ) often around 0.04 or less. For clean rounded quartz sands, the critical (steady) state line can become almost flat in e-log p' space over the low stress range (i.e. λ close to zero). Ishihara (1993) showed that the steady state line for Toyoura sand curves from being essentially flat at low stresses to very steep at very high stresses. Hence, at low stresses, the steady state line is essentially at a constant value of void ratio. At high stresses, grain crushing occurs and the steady state line becomes similar to that of some clays.

When the slope of the critical (steady) state line is flat in e – log p' space, the state line becomes a constant value of void ratio (e) and, hence, is analogous to constant relative density. Hence, when the state line becomes very flat, the stress normalization should approach that used for relative density; i.e. n = 0.5. Therefore, a stress exponent of n = 0.5 should be appropriate for clean quartz sands in the low stress range (σ'_{vo} < 200 kPa). In the high stress range where the state line becomes very steep the stress exponent should approach that used for clay, i.e. n = 1.0.

Recently, Robertson and Wride (1998) suggested a simple technique to apply a variable normalization, using a soil behavior index (I_c) to perform variable stress normalization, where:

$$I_c = \left[(3.47 - \log Q)^2 + (\log F + 1.22)^2\right]^{0.5} \qquad (9)$$

The CPT soil behaviour type chart by Robertson (1990), uses a normalized cone penetration resistance (Q) based on a simple linear stress exponent of n = 1.0. The procedure using n = 1.0 was recommended by Robertson and Wride (1998) for soil classification in clay type soils when I_c > 2.6. However, in sandy soils when $I_c \leq 2.6$, Robertson and Wride (1998) recommended that data being plotted on the chart be modified by using n = 0.5.

The simplified normalization suggested by Robertson and Wride (1998) is easy to apply but produces a somewhat discontinuous variation of the stress exponent, n. To produce a smoother variation of the stress exponent the following modified method is recommended.

Assume an initial stress exponent n = 1.0 and calculate Q and F and then I_c. Then:

If $I_c < 1.64$ $\quad$ n = 0.5
If $I_c > 3.30$ $\quad$ n = 1.0
If $1.64 < I_c < 3.30$ $\quad$ $n = (I_c - 1.64)\,0.3 + 0.5$ (10)

Iterate until the change in the stress exponent, Δn < 0.01. When the in-situ vertical effective stress (σ'_{vo}) exceeds 300 kPa assume n = 1.0 for all soils.

In clean sands, the normalized cone resistance, Q, suggested by Robertson and Wride (1998) is essentially the same as the normalized cone resistance, q_{t1}, used by Yoshimine et al. (1997), since typically $q_t >> \sigma_{vo}$ in clean sands. A variable normalization, using a stress exponent (n) as a function of I_c, allows a transition from clean sands at low stresses (n = 0.5) to clays (n = 1.0) using CPT data.

The method described by Equation 10 is recommended for stress normalization of CPT results and is used in developing the new CPT-based method of estimating minimum undrained shear strength described in the next section.

4.2 Representative Values

When evaluating the potential for either cyclic or flow liquefaction, there is little guidance given on what value of penetration resistance can be taken as representative of the deposit. In the SPT based method for cyclic liquefaction suggested by Seed et al. (1985) and updated by the NCEER Workshop (1997), generally the average SPT $(N_1)_{60}$ value was taken from the case histories to develop the method. Similarly, Seed and Harder (1990) and Stark and Mesri (1992) generally used average values from case histories to develop the relationship between $(N_1)_{60}$ and minimum undrained shear strength. Fear and McRoberts (1995) argued that the minimum value of $(N_1)_{60}$ would be more appropriate. In general, if all values of the measured penetration resistance are used with a relationship that was based on average values, the resulting design will generally be somewhat conservative. A disadvantage of defining a criteria based on minimum values is the uncertainty that the measured values represent the minimum. In practice, a lower bound relationship is often applied to all measured penetration resistance values. Recently Popescue et al. (1997) suggested that the 20-percentile value would be appropriate as the representative value for liquefaction. The 20-percentile value is defined as the value at which 20 percent of the measured values are smaller (i.e. 80 percent are larger). In the authors' opinion, the 20-percentile value is likely the more representative value for any given deposit for the evaluation of liquefaction potential.

4.3 Minimum undrained shear strength: Cohesive soils

For insensitive normally consolidated clays, the peak undrained shear strength ratio can be estimated from the CPT using:

$$\frac{s_{u(peak)}}{\sigma'_v} = \frac{(q_t - \sigma_{vo})}{N_{kt} \cdot \sigma'_{vo}} = \frac{Q}{N_{kt}} \qquad (11)$$

As outlined earlier, for insensitive, normally consolidated clays, the peak undrained shear strength ratio in simple shear is between 0.2 to 0.3. Hence, assuming N_{kt} between 10 and 20, the normalized cone penetration resistance, Q, in normally consolidated insensitive clays is around 2 to 6. Combining an average $s_{u(peak)}/\sigma'_v = 0.25$ and an average N_{kt} of 15 gives Q = 3.75. The normalized friction ratio, F, in insensitive normally consolidated clays is typically between 5 and 10 percent (see Equation 5). As the sensitivity (S_t) of the clay increases, the sleeve friction decreases and hence, friction ratio decreases. Using the CPT soil behaviour type chart suggested by Robertson (1990) and the link between minimum undrained shear strength and CPT sleeve friction (Equations 2 and 5), it is possible to plot contours of minimum undrained shear strength ratio in simple shear, $s_{u(min)ss}/\sigma'_{vo}$, on the soil behaviour chart in Zones 2, 3 and 4, as shown in Figure 2. These contours are approximate, but provide a guide to a possible relationship between minimum undrained shear strength ratio in simple shear and CPT data for normally consolidated clays. In overconsolidated clays, the normalized cone resistance and friction ratio tend to increase with increasing OCR, which can offset any tendency for friction ratio to decrease with increasing soil sensitivity.

4.4 Minimum undrained shear strength: Cohesionless soils

Using the relationship for clean sands suggested by Yoshimine et al. (1998), it is possible to identify the approximate location of contours of minimum undrained shear strength ($s_{u(min)ss}/\sigma'_{vo}$) in simple shear for clean sands (Zone 6) on the CPT soil behaviour chart, as shown in Figure 2. It is clear from Figure 2 that any contours of undrained shear strength ratio should vary from those for the clean sand to those for clay.

It is also possible to develop complete contours of minimum undrained shear strength for each direction of loading based on the results from Yoshimine et al. (1998). However, in this study, the focus is on the value of the minimum undrained shear strength in simple shear. The following describes how to extend the relationship suggested by Yoshimine et al. (1998) for clean sands to silty sands, silts and possibly clays.

Robertson and Wride (1998) suggested a method for correcting normalized cone resistance to an equivalent clean sand value, $(Q)_{cs}$, using a correction factor K_c, where K_c is a function of grain characteristics estimated using the soil behavior type index, I_c.

$$(Q)_{cs} = K_c\,(Q) \qquad (12)$$

where

if $I_c \leq 1.64$ $\quad K_c = 1.0$

if $I_c > 1.64$ $\quad K_c = -0.403\ I_c^4 + 5.581\ I_c^3 - 21.63\ I_c^2 + 33.75\ I_c - 17.88$

Robertson and Wride (1998) suggested that CPT data that plot in the region defined by $1.64 < I_c < 2.36$ and $F < 0.5\%$ should be assumed to indicate a loose clean sand and hence, K_c should be set equal to 1.0.

The correction factor, K_c, is approximate since the CPT responds to many factors, such as soil plasticity, fines content, mineralogy, soil sensitivity, age and stress history. However, in general, these same factors influence both the resistance to cyclic loading (Robertson and Wride, 1997) and the undrained shear strength ratio in a similar manner.

By combining the relationship between normalized cone resistance and minimum undrained shear strength in simple shear for clean sands, suggested by Yoshimine et al. (1998), with the correction factor, K_c, it is possible to develop contours of undrained strength ratio on the CPT soil behaviour type chart. The resulting contours are shown in Figure 3. The resulting contours fit the general location of values for sands and clays shown in Figure 2.

The resulting contours shown in Figure 3 are approximate and apply primarily to young, normally consolidated, uncemented soils. Sandy soils with angular grains and aged soils would likely have higher strengths. Soils that have a minimum undrained shear strength ratio in simple shear of around 0.30 or higher are generally not brittle. Soils that have a minimum undrained shear strength ratio of around 0.10 or less are often highly brittle (Yoshimine et al. 1998). Hence, the contour of $s_{u(min)}/\sigma'_{vo} = 0.10$ represents the approximate boundary between soils that can show significant

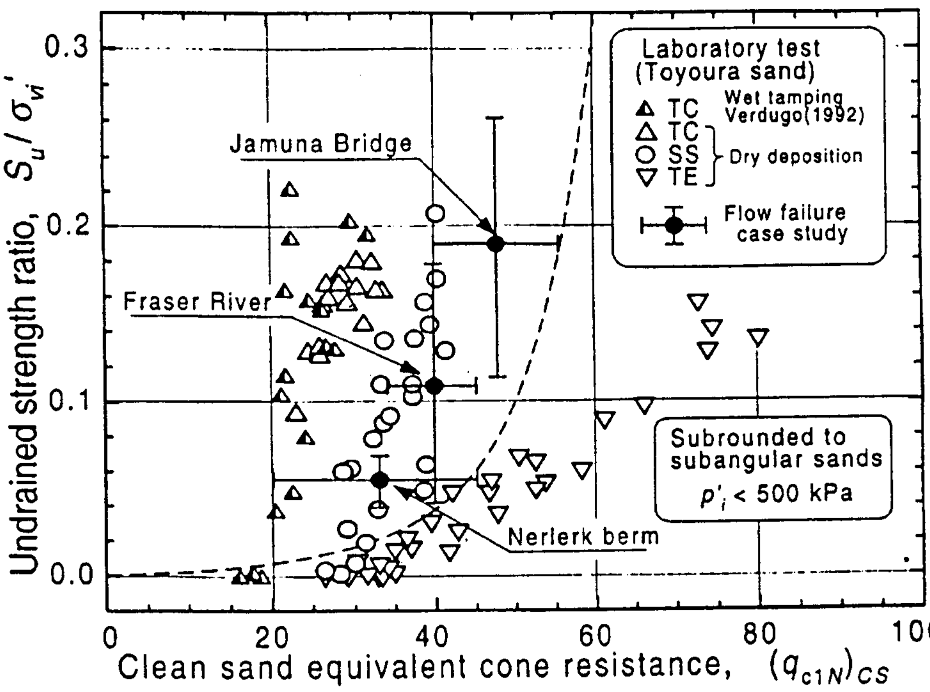


Figure 1. Minimum undrained shear strength ratio for clean sand as a function of CPT cone resistance (After Yoshimine et al;. 1998)

strain softening in undrained simple shear and soils that are general not strain softening.

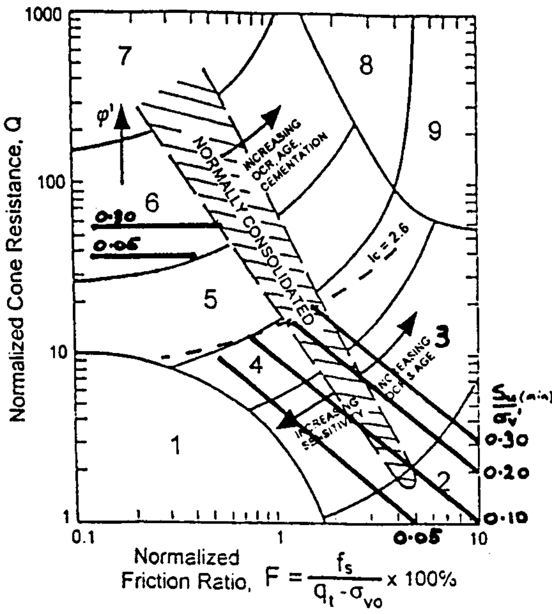
5 SUMMARY

A proposed screening method has been described to estimate the minimum undrained shear strength of soils from the CPT. The minimum undrained shear strength is defined as the minimum shear strength following undrained strain softening. Research has clearly shown that the undrained shear strength of soils is usually a function of direction of loading, with undrained shear strengths in compression loading often being higher than those in simple shear and triaxial extension. The resulting average minimum undrained shear strength is therefore a function of the slope geometry. Although all projects should be evaluated based on their actual geometry, often the average undrained shear strength is close to that in simple shear loading.

The proposed screening technique uses a variable normalization of CPT data based on soil type and in-situ vertical effective stress. The proposed normalization is modified from the method proposed by Robertson and Wride (1997) and makes use of a soil behaviour index (I_c). The proposed screening method builds upon the technique proposed by Yoshimine et al. (1998) in which normalized cone resistance was linked to the minimum undrained shear strength ratio for clean sands. The relationship proposed by Yoshimine et al. (1998) was based on laboratory test results on rounded sands as well as case histories of flow liquefaction failures in essentially clean sands. Yoshimine et al., (1998) suggested that the undrained shear strength in simple shear is often a reasonable average value for most slope geometries in sands, which was consistent with the observations made by Bjerrum (1972) for slopes and embankments in clays.

By combining the results from Yoshimine et al. (1998) and the CPT based approach suggested by Robertson and Wride (1997) contours of minimum undrained shear strength ratio on the CPT soil behaviour chart by Robertson (1990) were developed. The resulting contours (shown in Figure 3) are approximate and apply primarily to young, normally consolidated, uncemented soils.

Sands that have angular grains may have a minimum undrained strength ratio higher than predicted using the suggested CPT chart. Aged soils (age > 1,000 years) may also be somewhat stronger. For high risk projects, the proposed CPT method provides a useful screening technique to identify potentially critical zones. For low risk projects, the



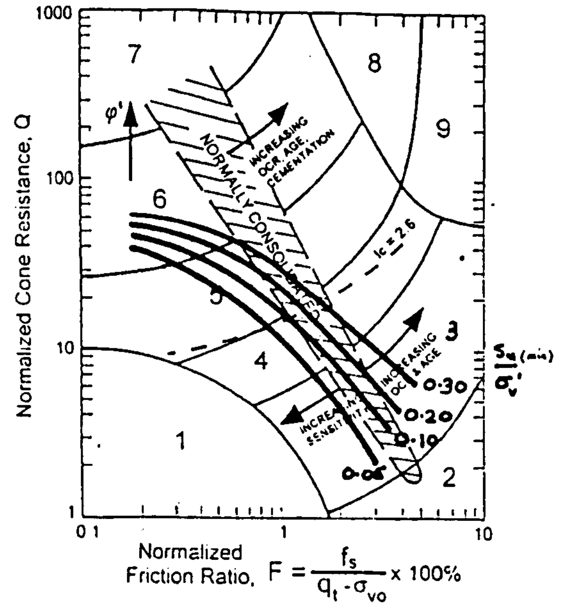
1. Sensitive, fine grained
2. Organic Soils – peats ($I_c > 3.6$)
3. Clays – silty clay to clay ($2.95 < I_c < 2.95$)
4. Silt Mixtures – clayey silt to silty clay ($2.60 < I_c < 2.95$)
5. Sand Mixtures – silty sand to sandy silt ($2.05 < I_c < 2.60$)
6. Sands – clean sand to silty sand ($1.31 < I_c < 2.05$)
7. Gravelly sand to dense sand ($I_c < 1.31$)
8. Very stiff sand to clayey sand*
9. Very stiff, fine grained*

- Heavily overconsolidated or cemented

Note: Soil behaviour type index (I_c) is given by $I_c = [(3.47 - \log Q)^2 + (\log F + 1.22)^2]^{0.5}$

Figure 2. Approximate contours of minimum unstrained shear strength ratio in simple shear for clays and sands shown on the normalized CPT soil behaviour type chart proposed by Robertson (1990).

proposed CPT method will generally provide a conservative estimate of the minimum undrained shear strength ratio in simple shear loading. The proposed relationship conservatively estimates the minimum undrained shear strength ratio in simple shear for a soil structure which contains extensive amounts of loose soils with impeded drainage, such as thick deposits of loose interbedded sands and silts. In soil structures where drainage and consolidation of the liquefied layer can occur during and immediately after the earthquake, higher values of undrained shear strength will likely exist. Such conditions may exist in a thin deposit with free drainage to the ground surface or a deposit interbedded with extensive pervious gravel layers.



1. Sensitive, fine grained *
2. Organic Soils – peats ($I_c > 3.6$)
3. Clays – silty clay to clay ($2.95 < I_c < 2.95$)
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5. Sand Mixtures – silty sand to sandy silt ($2.05 < I_c < 2.60$)
6. Sands – clean sand to silty sand ($1.31 < I_c < 2.05$)
7. Gravelly sand to dense sand ($I_c < 1.31$)
8. Very stiff sand to clayey sand*
9. Very stiff, fine grained*

- Heavily overconsolidated or cemented

Note: Soil behaviour type index (I_c) is given by $I_c = [(3.47 - \log Q)^2 + (\log F + 1.22)^2]^{0.5}$

Figure 3. Recommended contours for estimating minimum undrained shear strength ratio in simple shear using the CPT.

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