# Seismic Design for Liquefaction
# Summary

## Introduction

This summary was produce for the Greater Vancouver Task Force in late 2006 for seismic geotechnical design. It represents a summary of seismic design for liquefaction. The ideas and suggestions are made by Dr. Peter Robertson based on research and experience from recent consulting assignments. In is not possible to address all issues in detail and to provide full supporting evidence in the space available. The sequence has been presented in an attempt to rationalize a complex topic.

## Project Risk

The level of site investigation and analysis should be governed by the risk of the project. Low risk projects can use traditional site investigation techniques combined with a simplified design approach using conservative criteria. High risk projects generally apply simplified design approaches for initial screening followed by more advanced techniques, where appropriate.

The risk of a project is a function of the hazards, consequences and the potential for cost savings (Robertson, 1994). For seismic design, the risk is primarily the earthquake hazard combined with the consequences of failure. In regions of medium to high seismic loading the project risk is controlled primarily by the consequences of failure which can result in safety, environmental and economic considerations. For many situations 'failure' is defined in terms of intolerable deformations.

An example of a low risk project might be the construction of a light-weight, flexible warehouse structure that will contain/store low risk materials/products in an industrial area (where there have been other similar structures) where the consequences of failure are generally low and where the potential for significant cost savings are also low. An example of a high risk project might be either a major critical structure (e.g. hospital, nuclear power plant, LNG tanks) or a large earth dam or embankment where failure would have major economic, environmental and safety consequences. Sometimes a relatively small (low risk) project may result in significant economic seismic design considerations which may require further analysis, if warranted.

The sequence for seismic design is slightly different for either level ground or steeply sloping ground.

# Level Ground Sites

A level ground site is defined as:

1. Level ground
2. Gently sloping ground (slope angle < 5 degrees)
3. Level or gently sloping ground with a nearby steep slope or 'free-face' (i.e. nearby slope with a height less than 5m)

Buildings are often constructed on level or gently sloping ground and are generally investigated using a level ground approach, since well designed structures often impose small average shear stress to ensure static stability.

Sequence to evaluate liquefaction

1. Evaluate susceptibility to seismic loading
2. Evaluate triggering of seismic liquefaction
3. Evaluate post-earthquake deformations.

### *1. Evaluate susceptibility to seismic loading*

The response of soil to seismic loading varies with soil type and state (void ratio, effective confining stress, stress history, etc.). Boulanger and Idriss (2004) correctly distinguished between *sand-like* and *clay-like* behavior. The following criteria can be used to identify soil behavior:

**Sand-like behavior**
Sand-like soils are susceptible to cyclic liquefaction when their behavior is characterized by Plasticity Index (PI) < 12 and Liquid Limit (LL) < 37 and natural water content ($w_c$) > 0.8 (LL). More emphasis should be placed on PI. (The PI and LL is measured on the portion passing the No. 40 sieve). See attached Figure 4 from Seed et al., 2003.

- Low risk project: Assume sand-like soils are susceptible to cyclic liquefaction unless previous local experience shows otherwise.
- High risk project: Either assume sand-like soils are susceptible to liquefaction or obtain high quality samples and evaluate susceptibility based on appropriate laboratory testing, unless previous local experience exists.

**Clay-like behavior**
Clay-like soils are generally not susceptible to cyclic liquefaction when their behavior is characterized by PI > 12 but they can experience cyclic failure.

- Low risk project: Assume clay-like soils are not susceptible to cyclic liquefaction unless previous local experience shows otherwise. Check for cyclic deformation/failure.
- High risk project: Obtain high quality samples and evaluate susceptibility to either cyclic liquefaction and/or cyclic failure based on appropriate laboratory testing, unless previous local experience exists.

These criteria are generally conservative. Boulanger and Idriss (2004) suggested that sand-like behavior is limited to PI < 7. Use the above (more conservative) criteria, unless local experience in the same geology unit shows that a lower PI is more appropriate. The intent is to classify borderline materials as sand-like to be conservative.

## *2. Evaluate triggering of seismic liquefaction*

### *Sand-like materials*

Apply the simplified (NCEER) approach as described by Youd et al (2001) using generally conservative assumptions. The simplified approach to evaluate the triggering of seismic liquefaction involves comparing the cyclic stress ratio (CSR) caused by the design earthquake with the cyclic resistance ratio (CRR) of the soil. Youd et al (2001) provided guidelines for various in-situ techniques, SPT, CPT, $V_s$ and BPT to estimate the CRR in predominately sand-like soils. The simplified approach should be used for low to medium risk projects and for preliminary screening for high risk projects. For low risk projects, where the simplified approach is the only method applied, conservative criteria should be used.

*CPT-based method*

The CPT is the preferred in-situ test due to the increased repeatability and the continuous nature of the profile. Youd et al (2001) provided a summary of the Robertson and Wride (1998 – R&W) CPT-based approach.

The R&W CPT-based approach uses the soil behavior type index $I_c$ to define soil behavior, using the CPT tip and sleeve resistance values. A correction based on soil behavior using $I_c$ is used to calculate a *clean sand equivalent* cone resistance. The CPT $I_c$ is used to correct measured cone resistance values to account for the variations in grain characteristics, such as fines content, plasticity, fabric, etc. Since sand-like soils tend to be heterogeneous with rapid variations in grain characteristics, the continuous CPT results allow corrections for each CPT data point (typically ever 5cm (~2 inches)) and hence capture the variability. Corrections based only on fines content (FC) are generally less reliable since they are unable to capture the important influence on soil behavior from the plasticity of the fines as well as the rapid variation in grain size and fines characteristics. It is well recognized that soils with a small amount of fines but containing a very active clay mineral (i.e. highly plastic fines) can behave more like clay than sand, even though the fines content is low. Since the CPT responds to soil behavior, the corrections (based on $I_c$) attempt to capture all aspects that influence behavior.

For low risk projects and for preliminary screening in high risk projects, R&W suggested that when $I_c < 2.6$ the soil would have sand-like behavior and would be susceptible to liquefaction depending on the size and duration of seismic loading and, when $I_c > 2.6$ the soil would have clay-like behavior and would likely not be susceptible to liquefaction. Youd et al (2001) recommend that soils should be sampled where $I_c > 2.4$ to verify the behavior type and susceptibility to liquefaction.

When $I_c$ > 2.4 selected (disturbed) soil samples (for grain size distribution, Atterberg limits and water content) should be obtained and tested to confirm susceptibility to cyclic liquefaction using the criteria in the previous section. Selective soil sampling based on $I_c$ should be carried out adjacent to some CPT soundings. Disturbed samples can be obtained using either direct push samplers (using CPT equipment) or conventional drilling/sampling techniques close to the CPT sounding. If SPT's are carried out to obtain samples, the borehole should be at least 20 borehole diameters from the CPT sounding to avoid disturbance on the SPT N-values. Perform the CPT first to provide a continuous stratigraphic profile and to minimize disturbance. The SPT and CPT results should be compared using the full CPT profile to ensure soil variability is captured in the comparison.

Although the CRR curves are based on average values from case histories, the CPT-based approach should be applied to all CPT data, (often measured ever 5 cm (~2 inches)), to maintain the detailed stratigraphy and to ensure conservatism. The CPT-based approach captures low (minimum) cone values in soil layers and in transition zones at soil boundaries (e.g. transition from clay to sand and from sand to clay). It is not recommended to apply either average or some percentile value for CPT data, since details in the soil stratigraphy may be lost, although it is common the smooth the CPT profile using values averaged every 3 or 5 data points.

If pore pressure measurements are included with the CPT (i.e. CPTu), the additional information can be used to aid in soil behavior type and susceptibility to liquefaction (Robertson, 1990), e.g. high CPT penetration pore pressures and slow pore pressure dissipation indicates a more clay-like behavior.

*SPT-based Method*

The SPT method is less preferred due to its poor repeatability and discontinuous nature of the test (typically every 1.5m or 5 feet). Also corrections to an equivalent clean sand resistance are based only on fines content which can not capture all aspects of soil behavior. Users should avoid using the SPT with a rope and cathead hammer system, since these tend to produce low energies with resulting unconservative (high) N values, unless energy measurements and corrections are made. Users should also avoid using a hollow-stem auger system for drilling the borehole in loose sand-like soils below the water table, unless carried out under very careful supervision. For high risk projects the energy of the SPT hammer/anvil system should be measured and N values corrected using the measured energies. SPT carried out in shallow, very loose sand-like soils $(N_1)_{60}$ < 10 can be influenced by the weight of the rods and hammer, which tend to produce overly conservative low N values (Niven et al 2005).

*Shear wave velocity ($V_s$) Method*

Shear wave velocity can be used either as a preliminary screening method or as a supplement to either the CPT or SPT method. When performing CPT, seismic CPT (SCPT) should be considered so that the both shear wave velocity ($V_s$) and cone data can be obtained in the same profile and used to provide independent evaluations of liquefaction potential. If both agree, there is greater confidence in the interpretation. If both do not agree, the more conservative can be used for low risk projects and further investigation should be carried out for high risk projects. High shear wave velocities compared to cone penetration resistance may indicate slight cementation or more clay-like behavior.

Youd et al (2001) recommend that no correction should be made for high overburden stress (i.e. assume $K_\sigma = 1.0$). The simplified approach described by Youd et al (2001) is based on case histories where there is no evidence of liquefaction at depths greater than 20m (60 feet). For low risk projects it is reasonable to assume that there will be no liquefaction below 20m. For high risk projects appropriate laboratory testing on high quality samples to evaluate the influence of high overburden stresses should be considered, where appropriate.

***Clay-like materials***

Because of the cohesive nature of clay-like materials, they tend to develop smaller pore pressures under undrained cyclic loading than sand-like materials. Hence, clay-like materials do not reach zero effective stress with resulting large deformations under cyclic loading. Hence, clay-like materials are not susceptible to cyclic liquefaction. However, when the cyclic stress ratio (CSR) is large relative to the undrained shear strength ratio ($S_u/\sigma'_{vc}$) of clay-like materials, cyclic deformations can develop. However, post-earthquake volumetric strains tend to be small. Boulanger and Idriss (20024) used the term 'cyclic failure' to define this build-up of deformations under cyclic loading in clay-like soils. Boulanger and Idriss (2004) showed that the CRR for cyclic failure (deformations) in clay-like materials is controlled by the undrained shear strength ratio, which is controlled by the stress history (OCR). Boulanger and Idriss (2004) recommended the following expressions for $CRR_{M=7.5}$ (for a magnitude 7.5 earthquake) in natural deposits of clay-like soils:

$$CRR_{M=7.5} = 0.8\ (S_u/\sigma'_{vc})\ K_\alpha \qquad (1)$$

and

$$CRR_{M=7.5} = 0.18\ (OCR)^{0.8}\ K_\alpha \qquad (2)$$

Where:

$S_u/\sigma'_{vc}$ is the undrained shear strength ratio for the appropriate direction of loading (typically direct simple shear).

$K_\alpha$ is a correction factor to account to static shear stress. For well designed structures where the factor of safety for static loading is large, $K_\alpha$ is generally close to 0.9.

For seismic loading where CSR < 0.6, cyclic failure (deformation) is possible only in normally to lightly overconsolidated (OCR < 4) clay-like soils.

Boulanger and Idriss (2004) recommended three approaches to determine CRR for clay-like materials, which are essentially:

1. Estimate using empirical methods based on stress history (equation 2 above)

2. Measure Su using in-situ tests (equation 1 above)
3. Measure CRR on high quality samples using appropriate cyclic laboratory testing.

The CPT can be used to estimate both undrained shear strength ratio ($S_u/\sigma'_{vc}$) and stress history (OCR). The CPT has the advantage that the results are repeatable and provide a detailed continuous profile of OCR and hence CRR. Contours of $CRR_{M=7.5}$ (for $K_\alpha = 1.0$) in the clay-like region of the CPT SBT chart are shown in Figure 2. Figure 2 also shows the contours of $CRR_{M=7.5}$ in the sand-like region using the R&W method. If the R&W method (based on clean sand equivalent values) is used in the clay-like region (where $I_c > 2.6$) the estimated values of $CRR_{M=7.5}$ are generally conservative (i.e. smaller than those derived from OCR for clay-like soils). Details are contained in the attached Appendix.

For low risk projects, the $CRR_{M=7.5}$ for cyclic failure in clay-like soils can be estimated using conservative correlations from the CPT (see Figure 2). For medium risk projects, field vane tests (FVT) can also be used to provide site specific correlations with the CPT. For high risk projects high quality undisturbed samples should be obtained and appropriate cyclic laboratory testing performed. Since sampling and laboratory testing can be slow and expensive, sample locations should be based on preliminary screening using the CPT.

## *2. Evaluation of post-earthquake deformations*

### *Vertical settlements*

For low to medium risk projects and for preliminary estimates for high risk projects, post earthquake settlements can be estimated using various empirical methods to estimate post-earthquake volumetric strains (Zhang et al., 2002). The method by Zhang et al (2002) has the advantage that it is based on CPT results and can provide a detailed vertical profile of volumetric strains at each CPT location. The CPT-based approach is generally conservative since it is applied to all CPT data often using either commercially available software or in-house spreadsheet software. The CPT-based approach captures low (minimum) cone values in soil layers and in transition zones at soil boundaries. These low cone values in transition zones often result in accumulated volumetric strains that tend to increase the estimated settlement. Engineering judgment can be used to remove excessive conservatism in highly interbedded deposits where there are frequent transition zones at soil boundaries.

Engineering judgment is required to evaluate the consequences of the calculated vertical settlements taking into account soil variability, depth of the liquefied layers and project details (see Zhang et al., 2002). For high risk projects, selected high quality sampling and appropriate laboratory testing may be necessary in critical zones identified by the simplified approach.

### *Lateral deformations*

For low to medium risk projects and for preliminary evaluation for high risk projects, post earthquake lateral deformation (spreading) can be estimated using various empirical methods (Youd et al, 2002 and Zhang et al, 2004). The method by Zhang et al (2004) has the advantage that it is based on CPT results and can provide a detailed vertical profile of strains at each CPT location. The CPT-based approach is generally conservative since it should be applied to all CPT data and captures low (minimum) cone values in soil layers and in transition zones at soil boundaries. These low cone values in transition zones often result in accumulated shear strains that tend to increase the estimated lateral deformations. Engineering judgment can be used to remove excessive conservatism in highly interbedded deposits where there are frequent transition zones at soil boundaries.

Engineering judgment is required to evaluate the consequences of the calculated lateral displacements taking into account, soil variability, site geometry, depth of the liquefied layers and project details. In general, assume that any liquefied layer located at a depth more than twice the depth of the free-face will have little influence on the lateral deformations. For high risk projects, selected high quality sampling and appropriate laboratory testing may be necessary in critical zones identified by the simplified approach.

When the calculated lateral deformations using the above empirical methods are very large (i.e. shear strains of more than 30%) the soil should also be evaluated for susceptibility for strength loss (see next section on sloping ground) and the overall stability against a flow slide evaluated.

## Steeply Sloping Ground Sites

A sloping ground site is defined as:

1. Steeply sloping ground (slope angle > 5 degrees)
2. Earth embankments (e.g. dams, tailings structures)

Sequence to evaluate liquefaction

1. Evaluate susceptibility for strength loss
2. Evaluate stability using post-earthquake shear strengths
3. Evaluate if earthquake will trigger strength loss
4. Evaluate deformations.

### *1. Evaluate Susceptibility for Strength loss*

Use in-situ testing, preferably CPT, plus disturbed samples (for grain size distribution, Atterberg limit and water content) to identify materials that are susceptible to strength loss due to earthquake shaking, i.e. loose, sand-like and sensitive clay-like materials. Use conservative evaluation techniques, since experience has shown that when strength loss occurs, instability can be rapid with little warning and deformations can be very large.

a. Loose, sand-like materials (susceptible to strength loss)
    i. Either fines content < 20% or fines content > 35% and Plasticity Index (PI) < 12 and water content ($w_c$) > 0.80 Liquid Limit (LL)
    ii. CPT ($q_{c1N}$) < 70 and SPT $(N_1)_{60}$ < 14. These values will be conservative for soils with high fines content. (Robertson suggests that these values be 'clean sand equivalent', i.e. CPT ($q_{c1Ncs}$) < 70 and SPT $(N_1)_{60cs}$ < 14, but currently there is no published evidence to fully support the use of 'clean sand' values.)

b. Sensitive Clay-like materials (test for susceptibility, function of sensitivity and strain to failure)
    i. Fines content > 35%, and water content ($w_c$) > 0.85 LL
    ii. CPT Zone C (see CPT chart, Figure 2) and/or FVT, where sensitivity, $S_t > 5$
    iii. Shear strain to failure less than 2%

c. Clay-like materials (not susceptible to sudden strength loss)
    i. Fines content > 20% and PI > 12, and water content ($w_c$) < 0.80 Liquid Limit (LL)
    ii. CPT Zone B (see CPT chart, Figure 2)

If layers/zones of low permeability exist that could inhibit pore water redistribution after seismic loading and promote void redistribution, increase conservatism when evaluating susceptibility for strength loss.

## 2. *Evaluate stability using post-earthquake shear strengths*

a. Initial stability analysis assuming strength loss is triggered and using conservative estimates of minimum (liquefied/residual/steady state) undrained shear strength, $S_{ur}$, based on either empirical correlations with in-situ tests or field measured values:

    i. Use $S_{ur}/\sigma'_v$ or $S_{ur}$ versus CPT $q_{c1N}$ and/or SPT $(N_1)_{60}$ in loose sand-like materials (assume $S_{ur}/\sigma'_v \sim 0.05$, if no data available)
    ii. Use remolded undrained shear strength, $S_{ur}$, for sensitive clay-like materials measured from either CPT ($f_s$) or FVT
    iii. Use 80% of peak undrained shear strength, $S_{up}$, measured using either CPT or FVT in clay-like materials (or drained strength, whichever is lower)

If Factor of Safety (FS) > 1.2, assume stability is acceptable and check deformations

If FS < 1.2, evaluate material behavior and triggering in more detail

- For low risk structures, redesign or modify
- For moderate and high risk structures, carry out more detailed investigation

b. If project risk is moderate to high risk and FS < 1.2, evaluate post-earthquake shear strength in more detail:

   i. Additional in-situ testing, such as, SCPT, FVT, geophysical logging, and,
   ii. High quality undisturbed samples and appropriate laboratory testing.

   - If FS > 1.2, assume stability is acceptable and check deformations
   - If FS < 1.2, check triggering

If layers/zones of low permeability exist that could inhibit pore water redistribution after seismic loading and promote void redistribution, increase conservatism when evaluating post earthquake shear strengths. For high risk projects, the potential for void redistribution can be further evaluated using complex effective stress numerical models.

### *3. Evaluate if earthquake will trigger strength loss*

When FS < 1.0, using best estimates of post-earthquake shear strength parameters, assume that strength loss will be triggered.

When 1.0 < FS < 1.2 using best estimates of post-earthquake shear strength parameters, check if earthquake will trigger strength loss by applying either of the following approaches:

a. Pore-pressure approach, using CRR (Youd et al. 2001)
b. Strain-based approach (Castro, 2003)
c. Yield-strength approach (Sladen , 1985, Olsen and Stark, 2003)

All approaches should be based on improved knowledge of materials based on combined results from in-situ tests and appropriate laboratory testing on high quality samples. When soils are susceptible to strength loss and slopes are steep, a trigger mechanism should always be assumed to be present (Silvis and de Groot, 1995). Hence, for high risk structures caution and conservatism should be exercised.

If one or more zones are not expected to trigger strength loss, re-evaluate stability using higher shear strength in these zones.

- If FS > 1.2, assume stability is acceptable and check deformations
- If FS < 1.2 assume unsafe and redesign or modify

4. ***Evaluate seismic deformations***

If embankment is considered stable, evaluate seismic deformations based on size and duration of earthquake shaking.

a. Preliminary screening
    i. If no cyclic liquefaction in sand-like soils and/or cyclic failure in clay/like soils are identified and earthquake is small, assume deformations are small.
b. Pseudo-static analysis
    i. If earthquake is not large, M < 8, and,
    ii. If no significant zones indicate either strength loss or cyclic liquefaction, and,
    iii. Small deformations (less than 3 feet) are not significant to the performance of the embankment

    Use limit equilibrium stability analyses using pseudo-static seismic coefficient of 0.5 of the PGA and 80% of peak undrained strength for clay-like and sand-like materials (but not to exceed 80% of drained strength).

    - If FS > 1.0, deformations likely less than 1m (3 feet)

c. Newmark-type analyses (no seismic liquefaction/cyclic mobility)
    Perform a Newmark-type analysis if no zones of material indicate cyclic liquefaction or cyclic failure.

d. Numerical modeling (seismic liquefaction/cyclic mobility)
    Perform appropriate numerical analyses (finite element/finite difference) that incorporate special provisions for pore pressure build-up.

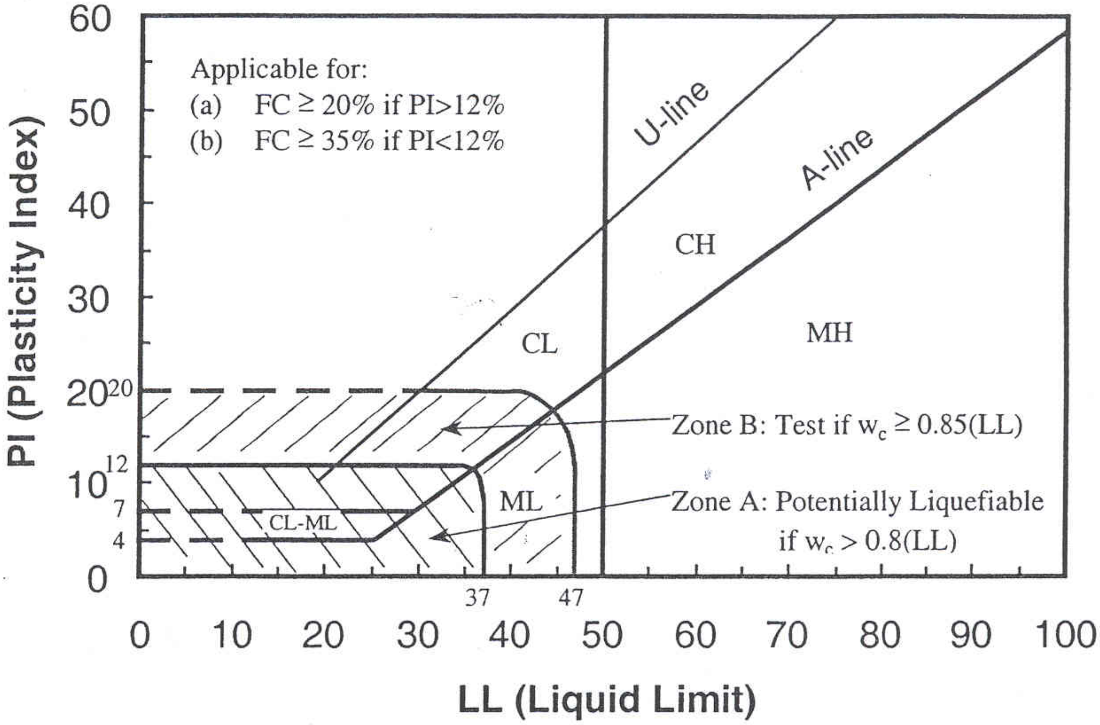



**Fig 4: Recommendations Regarding Assessment of "Liquefiable" Soil Types**

(After Seed et al., 2003)

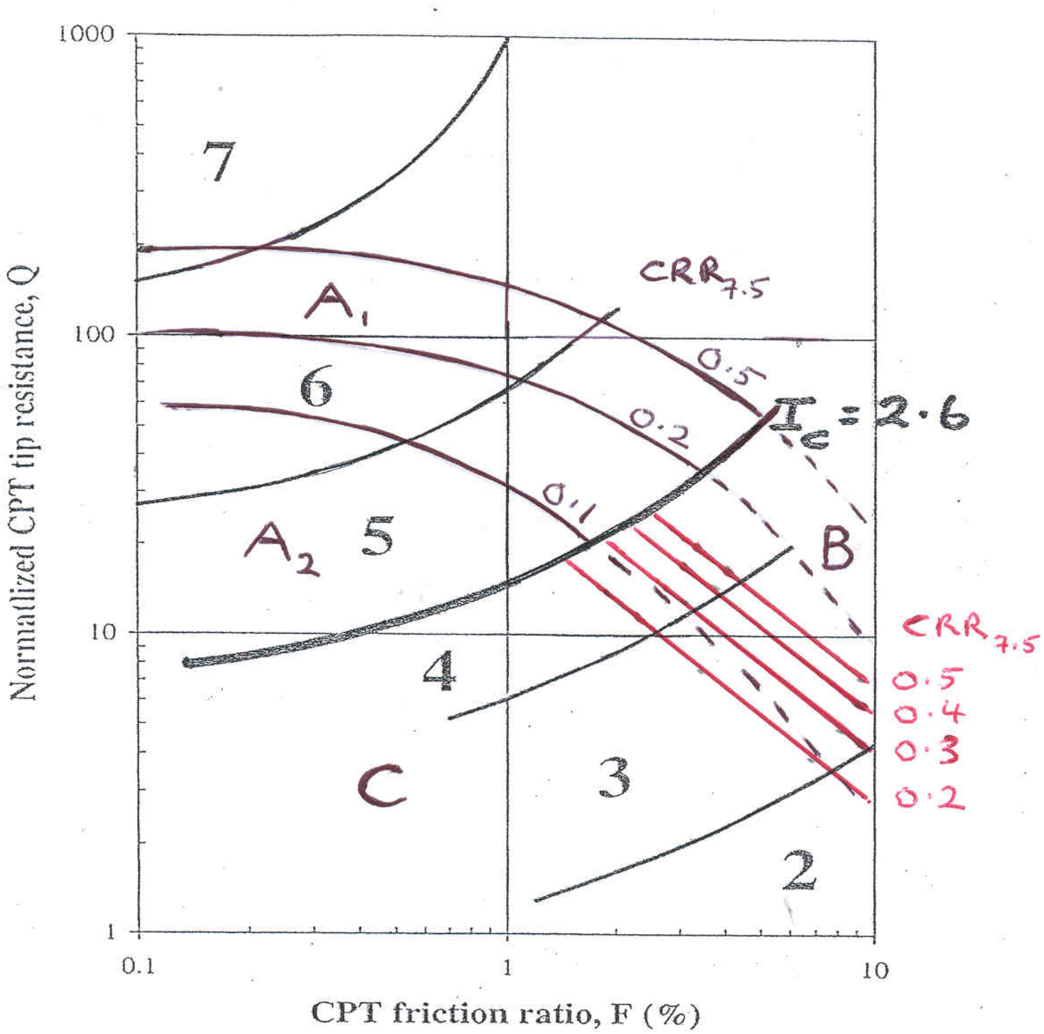
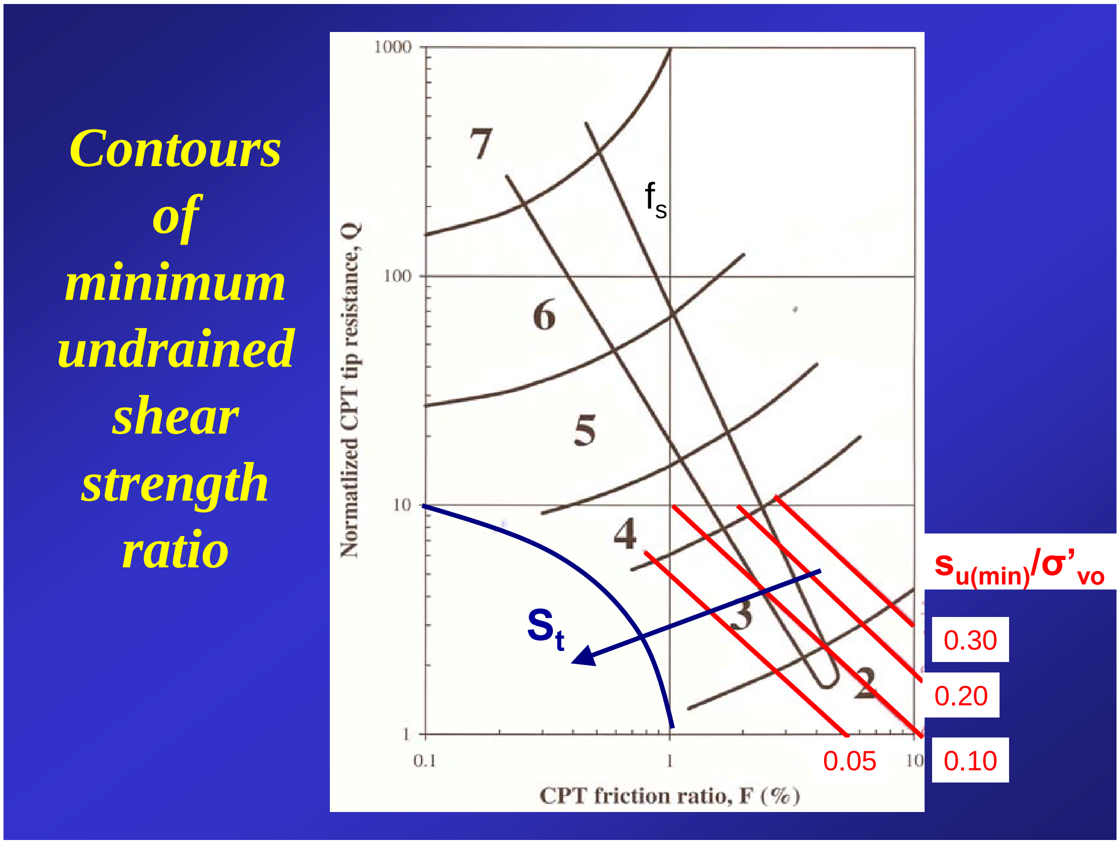



**Figure 2. Cyclic Resistance Ratio (CRR)$_{M = 7.5}$ using CPT**
(based on Robertson & Wride, 1998 and Boulanger and Idriss, 2004**)**

| | |
|---|---|
| Zone $A_1$ | Cyclic liquefaction likely depending on size and duration of cyclic loading |
| Zone $A_2$ | Cyclic liquefaction and strength loss likely depending on loading and ground geometry |
| Zone B | Liquefaction and post-earthquake strength loss unlikely, check cyclic failure |
| Zone C | Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry |

## APPENDIX: Undrained shear strength ratio from CPT




Normalized CPT parameters in clay, (where n = 1): (Robertson, 1990)

$Q = (q_t - \sigma_v)/\sigma_v'$ and $F = 100\ f_s / (q_t - \sigma_v)$ %

$S_{uPeak} = (q_t - \sigma_v) / N_{kt}$

$S_{uRemolded} = f_s$ (i.e. remolded undrained shear strength = CPT sleeve friction, Lunne et al, 1997)

Therefore:

$S_{uRemolded}/\sigma_v' = fs/\sigma_v' = (F . Q) / 100$ (see figure above for contours)

For overconsolidated clays

$(S_u/\sigma_v')_{OC} = (S_u/\sigma_v')_{NC}\ (OCR)^{0.8}$

Assuming that the average $(S_u/\sigma_v')_{NC} = 0.22$ the contours also represent OCR (for high values of $(S_u/\sigma_v')$ and Sensitivity for low values of $(S_u/\sigma_v')$, since $S_t = S_{uPeak} / S_{uRemolded}$