TASK FORCE REPORT

GEOTECHNICAL DESIGN GUIDELINES

FOR BUILDINGS ON LIQUEFIABLE SITES

IN ACCORDANCE WITH NBC 2005

for

GREATER VANCOUVER REGION

MAY 8, 2007

DISCLAIMER

This report reflects the general consensus of the Task Force Members on the "best practice" for geotechnical design of buildings on liquefiable sites in Greater Vancouver. The opinions and recommendations in this Report are those of the Task Force Members and not necessarily of their organizations. Use of this document is the sole responsibility of the user, and designers must use their own judgment in interpreting and applying the recommendations in this Report. Be aware that "best practice" will change with time as more research and observations become available.

Preamble

In 1991, a task force consisting of a group of local geotechnical and structural engineers produced a report entitled Earthquake Design in the Fraser Delta (Task Force Report 1991). The report was intended to provide general design guidelines for engineers involved in the seismic design of foundations for buildings in the Fraser Delta where liquefaction is a concern. At that time, the building code in effect was the National Building Code of Canada (1990) [referred to hereinafter as NBC 1990], and the seismic hazard stipulated by this code remained essentially unchanged until 2005. However, in 2005 the national building code changed to a new version, National Building Code of Canada (2005) [referred to hereinafter as NBC 2005], which includes a substantial increase in the return period of ground motions required for design. The seismic hazard in NBC 1990 was based on a probability of exceedance of 10% in 50 years (the 475 year ground motions, referred to hereinafter as A475), while NBC 2005 is based on an exceedance probability of 2% in 50 years (a 2475 year ground motions, referred to hereinafter as A2475). Furthermore, the seismic design philosophy has changed to collapse prevention from what used to be moderate damage and life safety. NBC 2005 considers the explicit use of over-strength factors for structural design, so that the lateral force levels required for the seismic design of structures has not changed appreciably. However, the larger intensity of ground motions poses problems for geotechnical engineers in assessing the potential for soil liquefaction, analysis and design of the foundations and the resulting movements, and if needed, remedial measures. Included in this report is a discussion on the structural deformation limits prescribed in NBC 2005, and how deformations caused by liquefaction might be assessed.

The purpose of this report is to provide revised general guidelines for geotechnical and structural engineers taking into consideration the longer return period ground motions and the change in seismic design philosophy. Since 1991, there have been considerable advances in the methods used to assess soil liquefaction, as well as analysis techniques that can better predict movements associated with liquefied sites. Furthermore, there have been a number of earthquakes in the past 15 years that have caused widespread soil liquefaction and foundation damage, and observations from these events have led to better analysis and design procedures for dealing with soil liquefaction. However, there are many judgmental factors in assessing soil liquefaction, and its implications on safety, and there is a need for some consensus on these issues to agree on an accepted state of practice when using the new NBC 2005. This Task Force Report reflects a consensus of the task force members on recommended design philosophies and methodologies to be followed in the seismic design of foundations.

NBC 2005 presents the seismic hazard in terms of a probabilistic-based uniform hazard spectrum, replacing the probabilistic estimates of peak ground velocity (PGV) and peak ground acceleration (PGA) in earlier codes. In addition, NBC 2005 explicitly considers ground motions from the potential Cascadia subduction earthquake located off the west coast of Vancouver Island. While the amplitude of peak ground motions resulting from such an earthquake are expected to be smaller than from local crustal earthquakes, the duration of shaking will be greater which has implications for liquefaction assessment.

This report presents guidelines for the analysis and design of building foundations in Greater Vancouver, where soil liquefaction is a concern. The concepts and guidelines presented herein may be extended to other geographic areas with appropriate modifications of the seismic hazard.

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1 Introduction

Seismic liquefaction refers to a sudden loss in shear stiffness and strength of soil due to cyclic loading effects of an earthquake. The loss arises from a tendency for soil to contract under cyclic loading, and if such contraction is prevented or curtailed by the presence of water in the pores, it leads to a rise in pore water pressure and a resulting drop in effective stress. If the effective stress drops to zero (100% pore water pressure rise), the shear strength and stiffness also drop to zero and the soil behaves as a heavy liquid. However, unless the soil is very loose, it will likely dilate and regain some shear stiffness and strength as it strains. The post-liquefaction shear strength is commonly referred to as the residual shear strength and may be 1 to 10 times lower than the static shear strength.

If the residual shear strength is sufficient, it will prevent a bearing capacity failure for level ground conditions, but large settlements may occur due to distortion during the earthquake and dissipation of excess pore water pressures after shaking. For sloping ground conditions, if the residual strength is sufficient, it will prevent a flow slide, but displacements commonly referred to as lateral spreading, could occur along with settlements. These ground or foundation movements may be excessive for functionality of the structure.

Determination of appropriate site-specific engineering properties for the soil conditions at a site is a key aspect in evaluating the liquefaction response. These are generally quantified based on in-situ penetration measurements and shear strength tests, as well as laboratory index and direct simple shear tests on representative soil samples. Understanding the geological processes leading to the formation of the soil deposit is important, and can provide insight to the engineering properties and their variability. Post-glacial or Holocene soils of fluvial and alluvial depositional origin and loose man-made fill materials are highly vulnerable to soil liquefaction and form the class of soils for which the design guidelines given in this report are applicable. An example of the effects of liquefaction on buildings in Japan during the 1964 Niigata earthquake is shown in Fig. 1.1. Pre-glacial soils encountered in the Greater Vancouver region are generally dense and would have a low vulnerability to liquefaction under the seismic loading conditions described in this report.

There is some evidence that liquefaction resistance may improve with aging and over-consolidation (preloading). However, these effects are difficult to quantify and are usually not directly addressed in design procedures; however, such effects should be reflected in terms of increased penetration resistance and/or shear strength.

1.1 Assessment of Liquefaction

Liquefaction assessment involves addressing the following concerns:

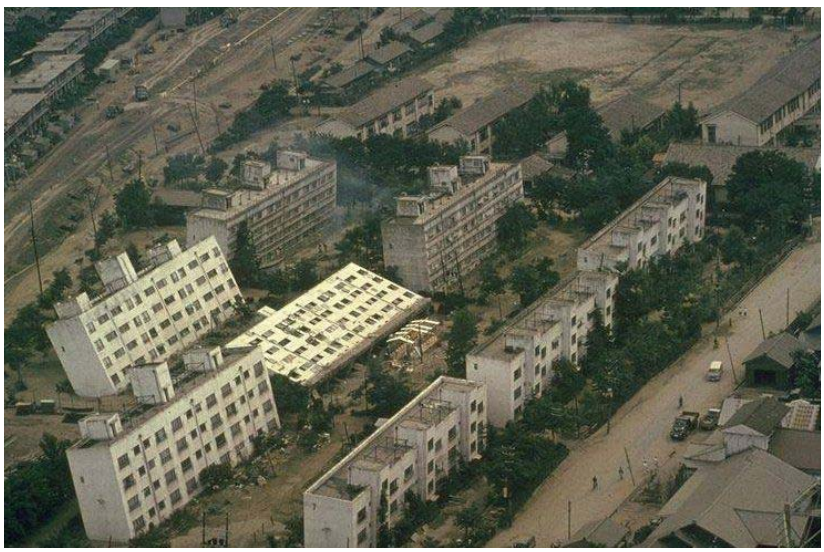
- Will liquefaction be triggered in significant zones of the soil foundation for the design earthquake, and if so,

- Could a bearing failure or flow slide occur, and if not,
- Are the displacements tolerable?

These effects can be assessed from simplified or detailed analysis procedures.

Simplified analysis of liquefaction triggering involves comparing the Cyclic Stress Ratio (CSR) caused by the design earthquake with the Cyclic Resistance Ratio (CRR) that the soil possesses due to its density.

The CSR depends on the design earthquake (Section 2) and soil and groundwater conditions (as described in Section 4). The CRR is discussed in Section 3.



http://www.ce.washington.edu/~liquefaction/selectpiclique/nigata64/tiltedbuilding.jpg

Figure 1.1 An example of the effects of liquefaction on buildings in Japan during the 1964 Niigata earthquake.

1.2 Bearing failure and/or Flow slide

If triggering of liquefaction is predicted in zones within the foundation soils, the possibility of a bearing failure or flow slide can be estimated from a bearing capacity or static limit equilibrium analysis using residual strengths in those zones predicted to liquefy. Residual strength is discussed in detail in Section 3.

1.3 Displacement

If some triggering of liquefaction is predicted but not a bearing failure or flow slide, both lateral and vertical displacements need to be estimated. These displacements depend on post-liquefaction stress-strain response of soils discussed in Section 3 of this report, and can be estimated from empirical as well as numerical analyses procedures described in Section 4.

1.4 Tolerable Displacements and Remedial Methods

Tolerable structural displacements from foundation movements are discussed in Section 5. Remedial measures to structures and/or to reduce predicted displacements to tolerable levels are described in Section 6.

2 Seismic Hazard, Choice of Earthquake Magnitude, and Ground Motion Records

This section provides guidelines for identifying the seismic hazard at a site, the magnitude of the earthquake to be used in the liquefaction assessment, and suggestions on earthquake ground motion records to be used. In the discussions presented herein, seismic hazard for Vancouver has been used as an example; the data for other location can be obtained from NBC 2005 on a site-specific basis.

2.1 Hazard

The design seismic hazard is specified in terms of probabilistic ground motions having a 2 percent chance of being exceeded in 50 years (i.e. A2475 event), and deterministic ground motions corresponding to the Cascadia subduction earthquake offshore of Vancouver Island. Both hazards should be considered in any design.

In NBC 2005, the applicable ground motions are described in terms of firm-ground (Site Class C) response spectra. The A2475 and subduction hazard firm-ground response spectra for Vancouver are shown in Fig. 2.1. This figure shows more data points than are found in the Climatic Data table of the Code, and the additional data are taken from Geological Survey of Canada (GSC) Open File 4459 (2003) which is the base document for the code seismic hazard. In particular, Open File 4459 gives more data for periods shorter than 0.2 seconds, but the data is only available for the major cities in Canada.

The hazard corresponding to A475 is also available in GSC Open File 4459 or at http://earthquakescanada.nrcan.gc.ca/hazard/interpolator/index_e.php. For Vancouver, the A475 spectrum is closely matched by multiplying the A2475 spectrum by a factor of 0.53, and is also shown in Fig. 2.1.

2.2 Earthquake Magnitude for Use in Liquefaction Assessment

When assessing the potential for liquefaction from the A2475 hazard it is necessary to select a magnitude for the ground motion. To date, the maximum recorded crustal earthquake for the Vancouver region is M7.3. The probabilistic analysis performed by the Geological Survey of Canada (GSC) to establish the seismic hazard, assumes an upper-bound magnitude of M7.7 as being possible. These magnitudes represent upper limits but are by themselves not a rational method of choosing what magnitude should be used in assessing liquefaction.

Deaggregation of the probabilistic seismic hazard gives a histogram of the magnitude and distance of the seismic events that contribute to the hazard. Figure 2.2 shows the deaggregation plot for the A2475 $S_a(2.0s)$ for Vancouver as given by Halchuk et al. (2007). Table 2.1 gives the mode, mean, median, and the mean plus one standard deviation estimates of the magnitudes for the A2475 ground motions for peak ground acceleration

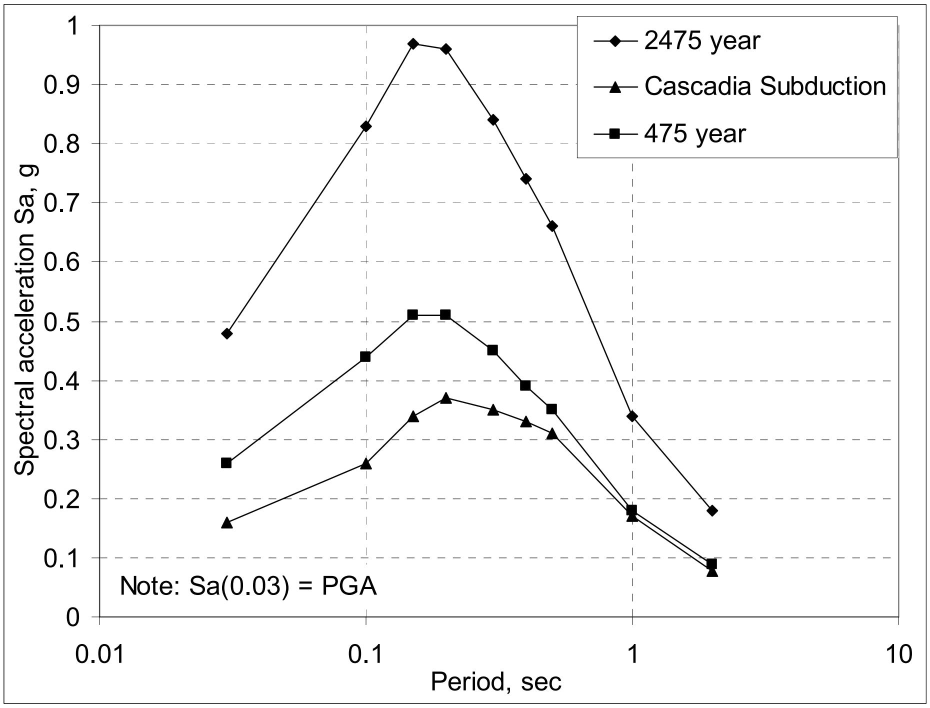


Figure 2.1 Vancouver; A475, A2475, and Cascadia Subduction response spectra.

(PGA), Sa(1.0s) and Sa(2.0s) spectral values for Vancouver. The mode magnitude represents the magnitude of the bin that has the highest contribution to the hazard, but statistically it is not meaningful.

Finn and Wightman (2006) have calculated the probabilistic hazard PGA for Vancouver where they have weighted the attenuation relations for the different magnitudes contributing to the hazard by the inverse of the magnitude scaling factor (MSF) used in liquefaction triggering assessment. The MSF is equal to 1.0 for M7.5 and increases for smaller magnitudes. They term this the weighted magnitude method. Their results show that the A2475 PGA is 0.32g with the weighted attenuation relations, as opposed to 0.46g if the attenuation relations are not modified. This implies that if M7.5 was chosen as the site magnitude to use in liquefaction assessment, then only 0.32g should be used as the firm ground PGA rather than 0.46g. Alternately, if M6.5 (MSF=1.44) is used for the site and the weighting factors changed accordingly, the A2475 PGA would be 0.46g, the same as the code A2475 PGA hazard for Vancouver. These studies suggest that code PGA and M6.5 should be used for liquefaction assessment when using the A2475 hazard. This is not recommended as discussed below.

The A2475 PGA deaggregation for Vancouver has a mean magnitude of 6.32. Thus the 6.5 magnitude calculated for liquefaction assessment with the PGA hazard of 0.46g is 0.2 units greater than the mean magnitude.

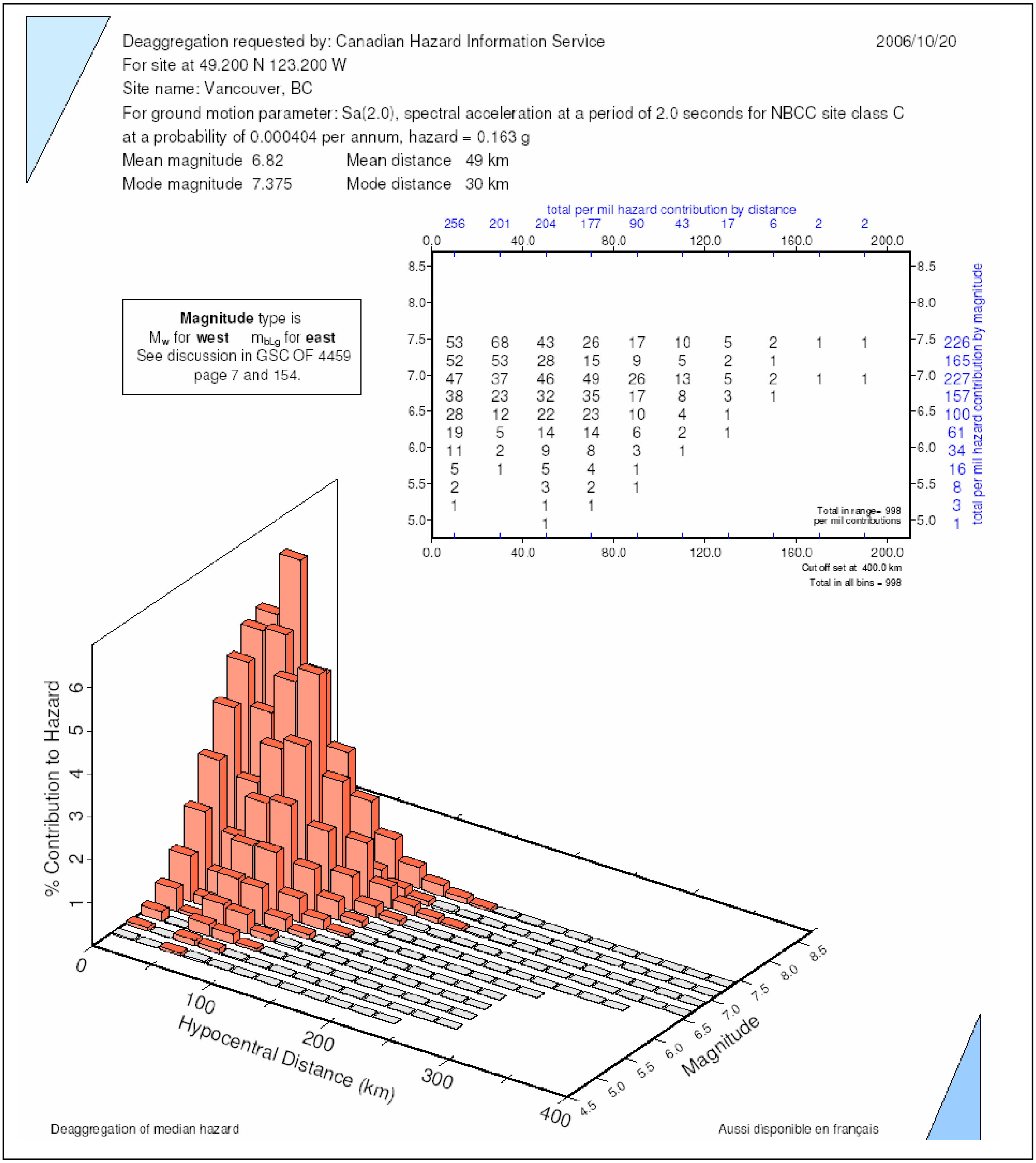


Figure 2.2 Deaggregation of Sa(2.0) for Vancouver for the A2475 hazard.

Studies on soft soil sites, using either SHAKE or FLAC, show that the shear stress, or shear stress ratio, near the surface is highly dependent on the spectral values of the input motion around the period of the soft soil layer, and not on the PGA. Thus it would seem more appropriate to use the deaggregation of the $S_a(1)$ or $S_a(2)$ spectra in determining the magnitude to use for liquefaction studies. The deaggregation mean magnitude values for $S_a(1)$ and $S_a(2)$ for Vancouver are M6.72 and M6.82 respectively. Assuming the 0.2 unit adjustment factor relating mean magnitude to liquefaction magnitude is the same for the spectral values as for the PGA values, a magnitude about M6.9 or M7.0 should be used for assessment of liquefaction when using the A2475 hazard. M7.0 is recommended.

Table 2.1: Deaggregation Magnitudes for Vancouver, A2475 Hazard

Measure of Earthquake Magnitude	PGA	S_a(1.0s)	S_a(2.0s)
Mode	7.125	6.875	7.375
Mean	6.32	6.72	6.82
Median	6.31	6.67	6.76
Mean + 1 std dev	6.88	7.15	7.20

The Cascadia subduction hazard considered by the GSC is an M8.2 event located some 140 km from Vancouver. Thus a magnitude of M8.2 should be assigned for assessment of soil liquefaction when the subduction hazard is considered. Although the spectral hazard for the A2475 hazard is much higher than the subduction hazard, the duration of the higher magnitude subduction earthquake may result in a more critical condition for liquefaction. Limited non-linear studies have shown that for weak soils in high seismic regions there is not much difference in the shear stress ratio between say the A2475 and A475 ground motions because of the limiting soil strength. In such cases the longer duration or higher number of cycles of the ground motion may become a more important factor and warrant further consideration.

2.3 PGA for Site Classes D and E

If the simplified Seed-Idriss approach is used to assess liquefaction triggering, the peak ground acceleration (PGA) at the surface is required. For Site Class C this is given in NBC 2005, but for softer sites such as D and E, which are more susceptible to liquefaction, it is not provided. NBC 2005 provides scale factors F_a and F_v to modify the spectral hazard for different site classes. The F_a factor modifies the low period part of the spectrum, but the code states that the design spectrum for periods shorter than 0.2 seconds should be constant at the $S_a(0.2)$ value. While this appears very conservative and implies the PGA is the same as $S_a(0.2)$, it is appropriate for structural design as the displacements, and ductility demand, are larger than expected if the usual design rules are followed for very short period structures. However, for liquefaction assessment this would be unreasonable, and a more realistic estimate of the PGA must be made.

If it is assumed that the F_a factor modifies the PGA in the same way as the short period spectral hazard, then the PGA for site classes D and E can be easily determined. For Vancouver, for site class D, F_a=1.1 and so PGA=0.46g*1.1=0.51g. For site class E, F_a=0.95 and thus PGA=0.44g. Note that the F_a value is dependent on the hazard level and for sites with lower hazard, F_a will be larger.

In classifying sites, NBC 2005 considers only the average soil properties in the top 30 metres, and for sites where the depth to firm ground is equal to or less than 30 metres, the above approach is thought to be reasonable. It should be noted that the "average" soil properties required in the NBC 2005 is not the arithmetic average but the travel time average (i.e. $V_{30} = 30/[\sum(h/V_s)]$ where h and V_s represent the thickness and shear wave velocity of individual layers between the ground surface and 30m depth) (Finn & Wightman 2003). Where the soft to stiff soil deposits extend deeper than 30 metres, the above approach is thought to be conservative.

2.4 Selection of Earthquake Records

The intent at the start of this task force was to develop a suite of records that would be appropriate for use in the Vancouver area for liquefaction assessment. The GSC started the task of assembling suites of appropriate earthquake records for the A2475 and A475 probabilistic hazards, and for the Cascadia subduction hazard, but at this time the project is on hold. Selecting earthquake records that match the firm ground response spectrum, and have the appropriate duration and/or number of cycles, is difficult for a region such as Vancouver that lacks records from past events with strong shaking. Given below are some general guidelines for choosing records, and a short description of some work that is ongoing to assess the suitability of records. It is anticipated that at a future date an addendum to this report will contain recommended records.

Some general guidelines for choosing suitable records are as follows:

- select records that have a spectrum close to the design spectrum;
- select records that have durations consistent with the earthquake magnitude and that have the appropriate number of cycles of strong shaking. For example, the NCEER assessment criteria (Youd et al. 2001) assumes that a M7 earthquake record has 10 significant full cycles greater than 0.65 PGA.
- scale the records so that they match the design spectrum in the period range of interest, or spectrally match the records to the design spectrum.

Scaling records should not be done on the basis of peak ground acceleration or peak ground velocity, but should entail scaling so that the spectrum is matched over the period range where the soil will respond to the ground motion. This usually entails scaling such that the average of the spectrum over the period range of interest is greater than the average of the design spectrum over the same range. The period range of interest is suggested as being from the smallest period that would produce significant response to 1.5 times the longest period of the soil column. It is recommended that if a minimum of three records are used the envelope of the response be used, whereas if seven or more records are used the average of the response can be used. If the surface ground motions or spectra are to be used for structural analysis, consideration should be given to the structural periods of interest when scaling the records.

The alternative to scaling records is to modify the records such that the spectrum of the modified record closely matches the design spectrum. The closer the spectrum of the original record is to the design spectrum means less change to the record, and better retention of the characteristics of the original record, but the selection of records is not as critical using this approach as it is with scaling of the records. At least two methods are available for modifying records, a frequency based system such as used in the program SYNTH (1985), and a time based system as used in the program RSPMatch (2005). Figure 2-3 shows the spectrum of an original record and the spectrum of the modified record using SYNTH, and how it matches with the design spectrum. The advantage of using modified records is that the scatter in the response amongst records is not as great as that from simple scaling of the records, and so fewer records are needed to get a reliable result. If orthogonal sets of records are perhaps required for 3-D structural analysis, see ASCE 7-05 for further discussion on scaling orthogonal pairs.

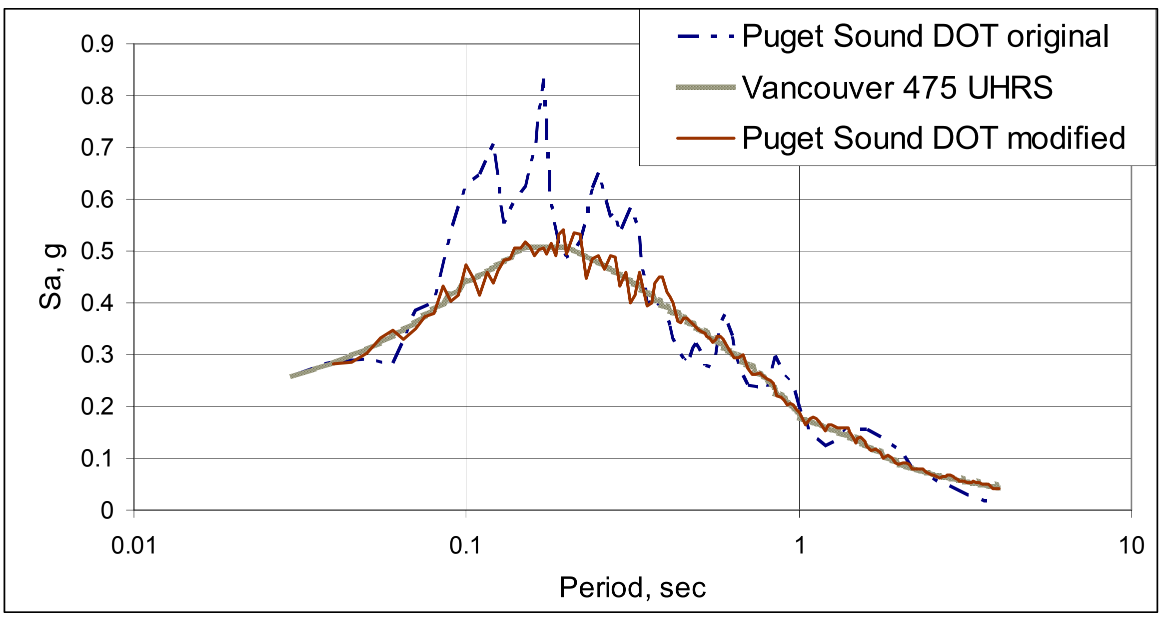


Figure 2.3 Spectra of Puget Sound DOT original record (scaled to 0.21g PGA), and same record modified to match the Vancouver A475 UHRS.

There is not a well developed method of selecting records with the appropriate duration or number of cycles. There are measures of duration and intensity, such as the Arias Intensity (Arias 1970), but it is not clear that they are appropriate for liquefaction studies, either in assessing the potential for liquefaction or movement if liquefaction should occur. The Newmark sliding block method has been used to assess slide movements, and so may be useful in rating records for duration or number of cycles of strong shaking. Some preliminary work has been done using the Newmark method, and involves applying the record at the base of a sliding block, where the sliding resistance has been specified as some percentage of the value needed to prevent sliding at the design PGA of the site, and then computing the resulting displacements. This procedure would need to be standardized so that the target displacement would be related to the earthquake magnitude, and a suitable suite of records then assembled that produce this target displacement. By implication these records should then have the requisite number of cycles of strong shaking for liquefaction assessment. It is planned that this work will comprise an appendix to this report at a later date.

2.5 Characteristic Earthquake Distance

The deaggregation results give the mean distance of the earthquakes contributing to the hazard. For Vancouver the mean distance for the A2475 spectral hazard is about 50 km. However, if a M7.0 earthquake, which is greater than the mean magnitude, is assumed at 50 km distance, the spectral hazard from such a scenario will be smaller than the spectrum produced by the probabilistic assessment unless the attenuation relation used is the mean plus several standard deviations.

If a distance is required in the assessment of liquefaction-induced ground displacements it is recommended that the magnitude-PGA-distance relation used in Youd et al. (2002) be used.

3 Liquefaction Resistance and Post-liquefaction Response

3.1 Introduction

Seismic loading subjects elements of the foundation soil to oscillating (or cyclic) shear stresses, typically denoted by the symbol τ_{cyc}. Laboratory and field experience indicate that it is the cyclic stress ratio (CSR), where CSR = τ_{cyc}/σ'_{vo}, and σ'_{vo} is the vertical effective stress prior to seismic loading that induces liquefaction and the development of large strains. The CSR that triggers liquefaction to occur in a specific number of cycles (usually 15) represents the capacity of the soil to resist liquefaction, and is called the Cyclic Resistance Ratio (CRR). CRR depends mainly on the soil type and density or state, and it can be obtained directly from tests on undisturbed samples of soil, or indirectly from field experience and in-situ testing at sites recently subjected to seismic loading.

When CRR is obtained using laboratory testing, the common approach is to subject specimens of soil to cyclic shear loading using triaxial or direct simple shear devices. In laboratory cyclic shear testing, an equivalent uniform amplitude cyclic stress ratio is generally applied and the number of cycles to cause liquefaction (100% pore pressure rise or a specified strain) is recorded. Cyclic direct simple shear (DSS) tests are considered to be representative of field conditions during earthquake loading. While other advanced apparatus such as hollow cylinder torsional (HCT) shear device may be considered to simulate these loadings, they are less attractive because of their lack of availability, experimental complexities, and the associated high costs.

3.1.1 *Response of coarse-grained soils*

Extensive work has been undertaken over the past 30 years to study the cyclic shear response of sands (e.g., Castro 1975, Ishihara et al. 1975, Vaid and Thomas 1995). The majority of the laboratory studies on sands have been carried out using the cyclic triaxial apparatus. However, the stress path followed in this apparatus does not simulate the cyclic rotation of principal stresses that take place during earthquake loading. The cyclic direct simple shear (DSS) test allows the variation of shear stresses in a cyclic manner in addition to the simultaneous changes in the direction of principal stresses. Therefore, the DSS test is considered more suitable for the simulation of earthquake loading conditions (Vaid and Finn 1979).

Typical results from constant volume cyclic DSS testing of loose Fraser River sand (Wijewickreme et al. 2005), conducted on specimens initially consolidated without static shear bias are shown in Fig. 3.1. The effective stress path, in Fig. 3.1(a), shows the normal effective stress reducing with each cycle of shear stress from an initial value of 100 kPa to essentially zero after 6 cycles. The shear stress versus shear strain response in Fig. 3.1(b) shows that the shear strains are very small, less than 0.1%, for the first 5 cycles, and they become very large, 10%, on the 6th cycle, when liquefaction is triggered. The strength developed after liquefaction arises from the tendency of sand to dilate and is associated with large strains and displacements. The applied stress ratio for this specimen was 0.1 and caused liquefaction in 6 cycles. The CRR is generally specified as the stress ratio to cause liquefaction in 15 cycles (deemed to be representative of a Magnitude 7.5 event), and from additional tests carried out on this material CRR was determined to be about 0.08.

The liquefaction response shown in Fig. 3.1 is typical for loose sands where the application of an additional cycle of load triggers an abrupt change in shear stiffness from stiff to soft. The soft post-liquefaction response during loading is controlled by dilation. The drop in shear stiffness upon liquefaction can be in the range of 100 to 1000 times. The shear strength or shear strength ratio available after liquefaction, called the residual shear strength (S_r) can be significant. As may be noted from Fig. 3.1, the shear strength ratio (S_r /σ'_{vo}) is at least 0.1 for loose Fraser River sand. However, experience from back-calculation of field case histories indicates that the residual strength ratio can be significantly lower than values obtained from undrained testing of uniform sand. The lower strength is likely due to upward flow of water associated with generated excess pore water pressures causing some elements to expand and lose their dilation effect and strength under field conditions, particularly those beneath layers of lower permeability.

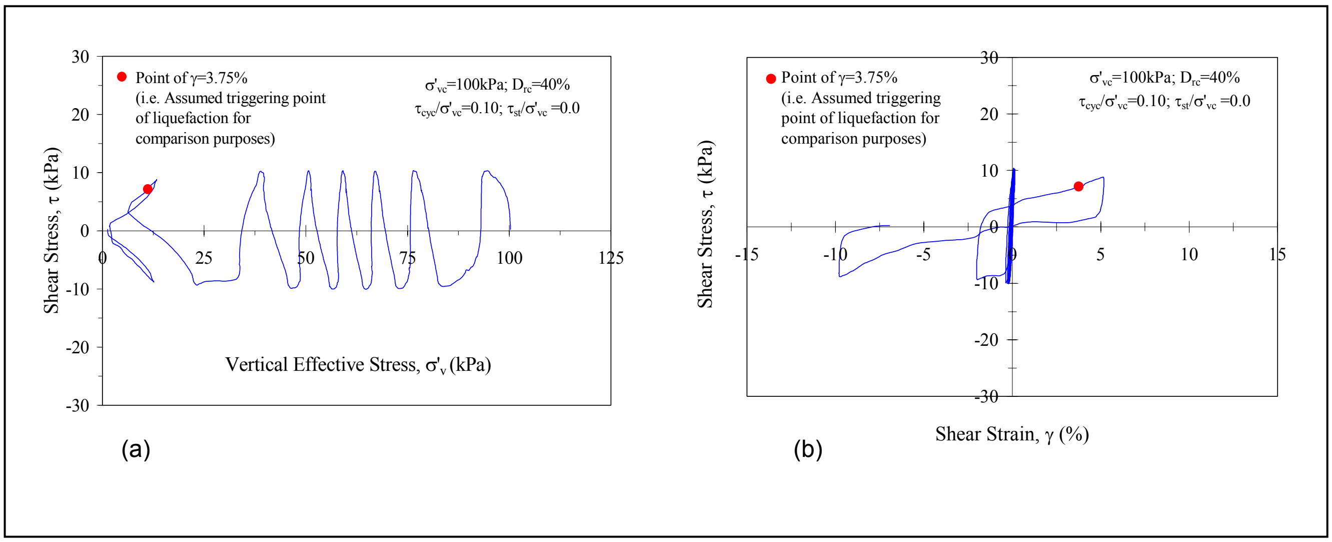


Figure 3.1 Typical response of loose Fraser River sand under cyclic DSS loading without static shear bias: (a) effective stress path; and (b) shear stress-strain response (Wijewickreme et al. 2005)

3.1.2 *Response of fine-grained soils*

Unlike for sands, the studies that have been conducted to understand the cyclic shear behavior of silt and silty sands are limited (Bray et al. 2004, Sanin and Wijewickreme 2006a and 2006b, Wijewickreme and Sanin 2007, Thevanayagam et al. 2002, Kuerbis et al. 1988).

As shown in Fig. 3.2, the response of fine-grained soils (silt and clay materials) to cyclic loading can be quite different in comparison to that for sand. This figure shows typical effective stress path and shear stress-strain response of a specimen of normally consolidated channel-fill Fraser River silt under cyclic direct simple shear (DSS) loading. The silt has an average plasticity index (PI) = 4. The effective stress path in Fig. 3.2(a) shows that the normal effective stress reduces from its initial value of 100 kPa with each increasing cycle, but not dropping below 10 kPa. After the initial few cycles, the loading (or increase in the magnitude of shear stress) part of a given cycle is associated with an increase in effective stress resulting from dilation. The shear stress-strain response [Fig. 3.2(b)] shows a gradual increase in strain with number of cycles, and there is no abrupt change in shear stiffness from stiff to soft. There is also no indication of a strength reduction below the applied cyclic stress ratio of 0.2; thus, the post-liquefaction or residual strength ratio (S_r / σ'_{vo}) is at least 0.2 for the tested silt. The stiffness reduces with each cycle; for example, after 11 cycles it is about 20 times softer than the first cycle. The results of these and many other tests (Sanin and Wijewickreme 2006a) suggest that fine-grained normally consolidated silts and clays of low plasticity can be far more resistant to liquefaction than loose sands.

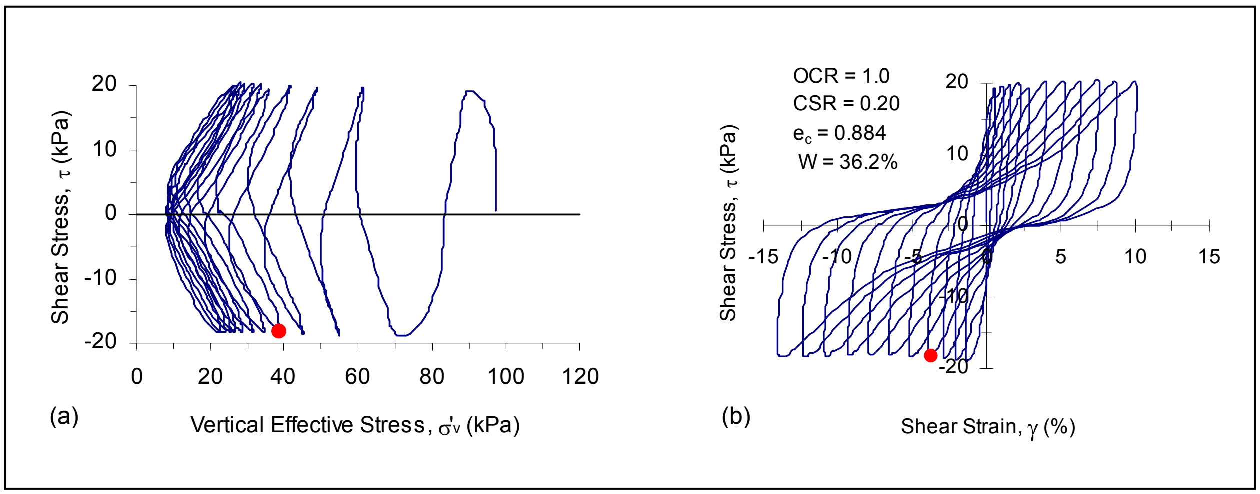


Figure 3.2 Typical response of channel-fill Fraser River silt under cyclic DSS loading without static shear bias: (a) effective stress path; and (b) shear stress-strain response (Sanin and Wijewickreme 2006a)

Sanin and Wijewickreme (2006b) also notes that the CRR vs. number of cycles to liquefaction of the tested normally consolidated Fraser River silt is not significantly sensitive to the overburden stress for stress levels in the range 85 kPa to 400 kPa. More testing would be required to extend the validity of this observation to higher stress levels.

3.1.3 *Overview Commentary - Response of Coarse and Fine-Grained Soils*

The above test results together with field experience suggest that the cyclic response of coarse-grained soils, gravels, sands and non-plastic silts should be treated differently than fine-grained plastic silts and clays. Such an approach is also in accord with common practice, in which liquefaction resistance of coarse-grained soils has been based on penetration resistance, and fine-grained soils on Atterberg limits and grain size characteristics (Youd et al. 2001). Both these approaches are indirect and are based on field experience during past earthquakes. While it might seem desirable to recover undisturbed samples and obtain a direct measure of liquefaction resistance from cyclic testing, it is very difficult and expensive to obtain undisturbed samples in coarse-grained soils. It is possible that sand elements will expand or contract during or after earthquake shaking; this suggests that the results from testing of undisturbed samples may not be representative of conditions in the field in the event of an earthquake. However, it is possible to obtain acceptable undisturbed samples from most fine-grained soil deposits (Bray et al. 2004, Sanin and Wijewickreme 2006a).

The question of the dividing line between coarse-grained and fine-grained soils from a liquefaction point of view has been discussed in some detail by Boulanger and Idriss (2004, 2006). They refer to coarse-grained soils as having “sand-like” behaviour, and fine-grained soils as having “clay-like” behaviour. They recommend that a plasticity index, PI, value of 7 be used as the demarcation between coarse and fine-grained liquefaction response.

It is also important to note that the field-measured value of Standard Penetration Test (SPT) resistance is influenced by many factors other than the soil and ground water conditions of the tested soil (e.g., energy delivered by the hammer, drill rod lengths, connections between drill rods, borehole conditions, as well as the grain size). Becker Penetration Test (BPT) is often used to determine penetration resistance in gravelly soils, and correlations between SPT and BPT have been made mainly based on data from tests conducted in sandy soils (Harder and Seed 1986, Sy and Campanella 1994). Scaled-up versions of the SPT, referred to as "Large Penetration Tests" (LPT), have also been developed to test gravelly soils. A fundamental method proposed by Daniel et al. (2003) is available to derive equivalent SPT $(N_1)_{60}$ values from LPT data. Currently, it is generally assumed that effect of grain size on the penetration resistance obtained from larger diameter penetrating tools is much less than those obtained from the smaller SPT sampler. Additional research in this field is needed to check the validity of this assumption.

While treatment of the above considerations with respect to field penetration tests are outside the scope of these guidelines, it is expected that the geotechnical engineer will carefully assess the suitability of the computed penetration resistance values prior to implementing them in the recommended approaches in this document.

Using the above background, guidelines were developed to assess the liquefaction resistance and post-cyclic response. General guidelines for the treatment of coarse-grained soils are presented in Section 3.2. Guidelines for the treatment of fine-grained soils herein have been developed combining the knowledge from the recent works by Boulanger and Idriss (2004, 2006), Bray et al. (2004), Bray and Sancio (2006), and Sanin and Wijewickreme (2006a), and are presented in Section 3.3.

3.2 Recommended guidelines for liquefaction resistance and post-cyclic response – coarse grained soils (gravels and sands)

3.2.1 *Cyclic Resistance Ratio (CRR)*

It is recommended that CRR for coarse-grained soils and low-plastic (PI < 7) fine-grained soils (if classified as per Section 3.3.1) be determined based on penetration resistance charts (see Figs. 3.3 and 3.4) in accordance with NCEER guidelines by Youd et al. (2001).

In the NCEER approach, base CRR curves are specified for a confining stress of 100 kPa (~1 ton/ft^2), a Magnitude 7.5 earthquake, and level ground conditions for use with standard penetration test (SPT) data (Fig. 3.3), cone penetration test (CPT) data (Fig. 3.4), and shear wave velocity (V_s) data (Fig. 3.5). The results of Becker penetration tests (BPT) can also be used by first converting the resistance measurements into equivalent SPT $(N_1)_{60}$ values using approaches proposed by Harder and Seed (1986) and/or Sy and Campanella (1994).

It is recommended that the chart based on shear wave velocity in Fig. 3.5 be used with caution as an indicator of liquefaction resistance, CRR, and only as a screening level tool. Shear wave velocity gives a direct measure of the elastic shear modulus of the soil at very small strain (1x 10^{-4}%). However, triggering of liquefaction occurs at much larger strains where plastic strains dominate response. Based on field measurements during past earthquakes, penetration tests which involve large strains seem to give a more reliable index of liquefaction resistance.

It is recommended that potentially liquefiable layers that show significant scatter be characterized by their 66th percentile value, i.e., 66% of the penetration test values should be larger than the selected characteristic value. This is recommended for CPT, SPT, and BPT tests because liquefaction response is dominated by the weaker zones within a soil mass.

The base CRR value obtained from Figs. 3.3 or 3.4 will be given the symbol CRR_1. The CRR for a general condition can be computed using Equation 3.1.

$$CRR = CRR_1 * K_m * K_\sigma * K_\alpha \qquad [3.1]$$

Where: K_m is a correction factor for earthquake magnitudes other than 7.5;

K_σ is a correction factor to account for effective overburden stresses greater than 100 kPa; and

K_α is a correction factor for ground slope.

The recommended K_m or magnitude scaling factor (MSF) is 1.2. This is based on the curve shown in Fig. 3.6, and an earthquake magnitude of 7.0 as recommended for the Greater Vancouver area, Section 2.2. The recommended K_σ curves depend on relative density (D_r) as well as effective overburden pressure and are shown in Fig. 3.7. From this figure:

$$K_\sigma = (\sigma'_{vo} / P_a)^{(f-1)} \qquad [3.2]$$

Where P_a is atmospheric pressure in the chosen units, f depends on relative density, D_r, and given by:

$$f = 1 - 0.005*(D_r) \text{ , for } 40\% < D_r < 80\% \qquad [3.3]$$

$D_r \leq 80\%$ can be estimated using:

$$D_r = [(N_1)_{60}/46]^{1/2} *100 \qquad [3.4]$$

NCEER does not recommend correction factors for K_α. Herein, it is recommended that K_α = 1 be used. However, the geotechnical engineer should recognize that the liquefaction resistance of loose sand with a static shear bias may be lower than without static shear bias (Harder and Boulanger 1997). On the other hand, the liquefaction resistance of dense sand with a static bias may be considerably higher than without a static bias. The appropriate value of K_α will be governed by the level of strain considered for estimating the CRR.

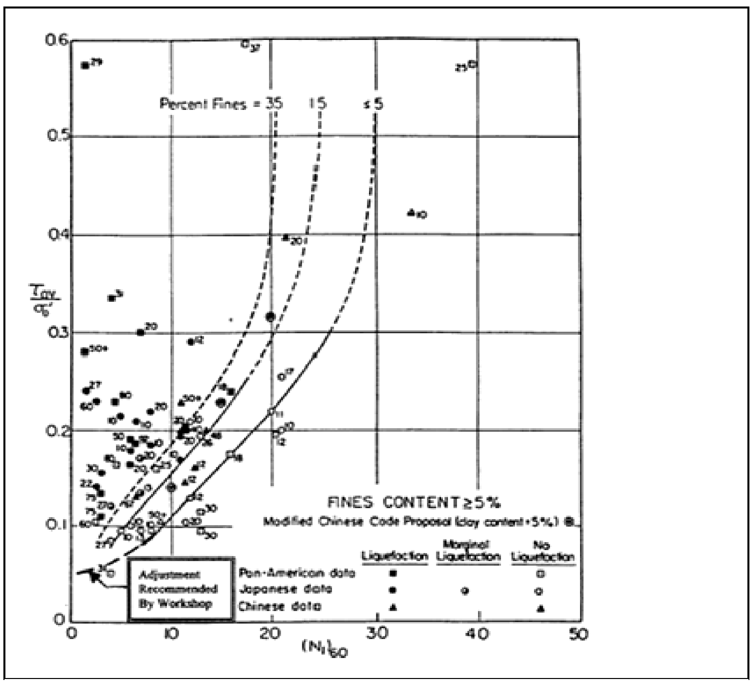


Figure 3.3 CRR_1 vs $(N_1)_{60}$ for M7.5 earthquake after Youd et al. (2001)

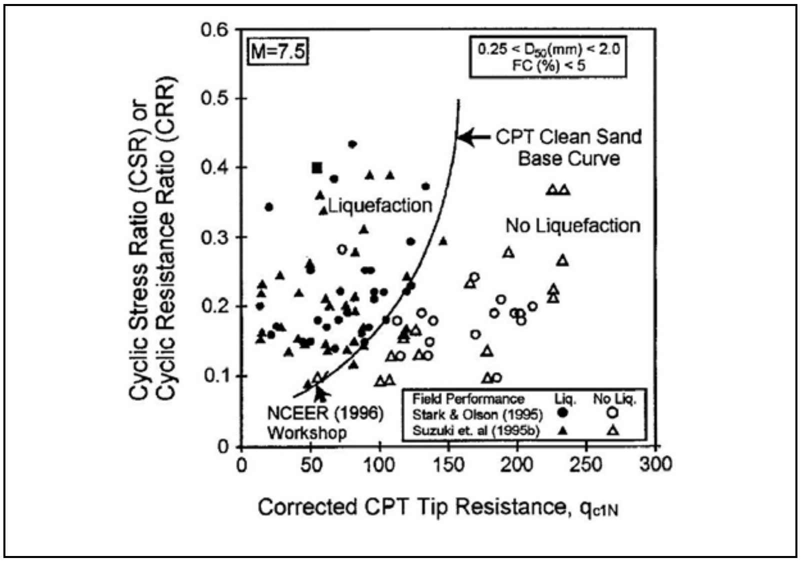


Figure 3.4 Curve recommended for calculation of CRR_1 from CPT data along with empirical liquefaction data from complied case histories [Robertson and Wride (1998) chart reproduced by Youd et al. (2001)].

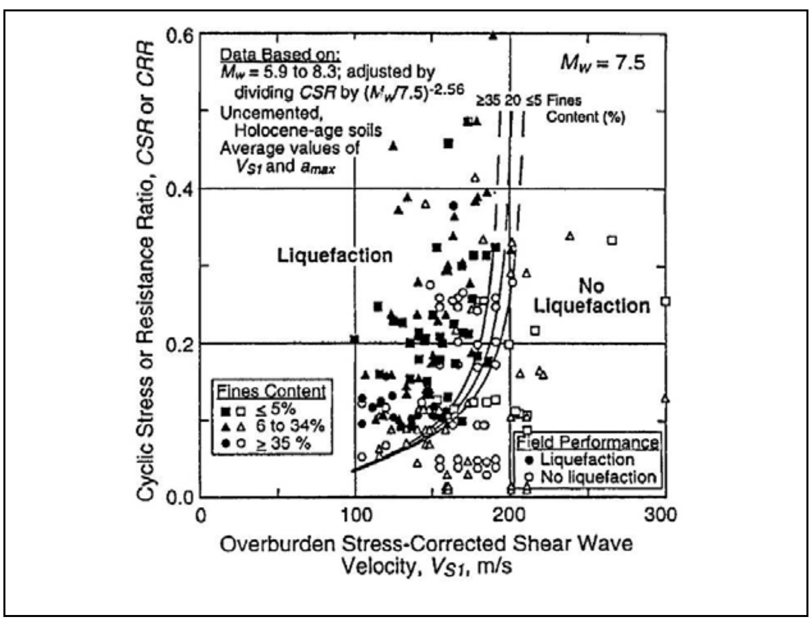


Figure 3.5 Curves for calculation of CRR_1 from shear wave velocity for clean un-cemented soils with data from compiled case histories [Andrus and Stokoe (2000) chart reproduced by Youd et al. (2001)].

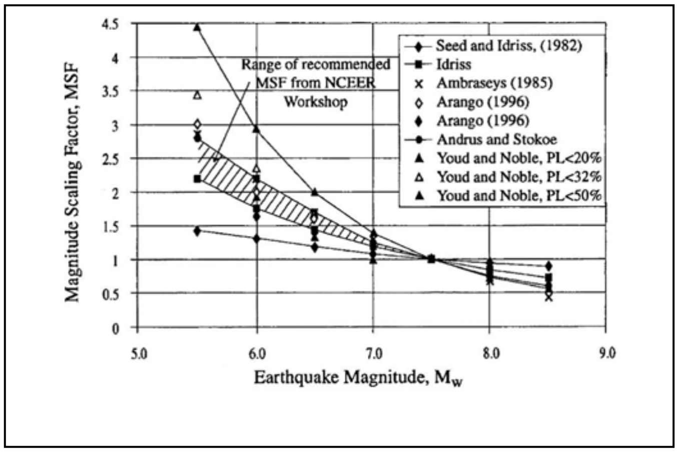


Figure 3.6 Magnitude Scaling Factors derived by various investigators [Youd and Noble (1997) chart reproduced by Youd et al. (2001)].

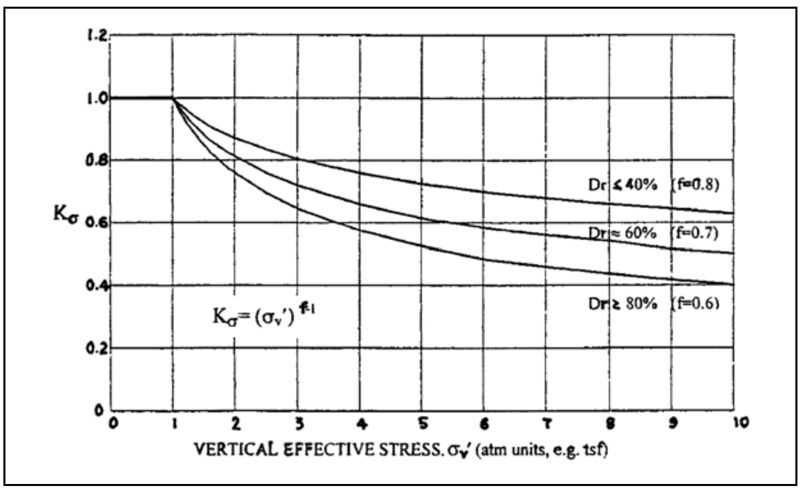


Figure 3.7 Recommended curves for estimating K_σ for Engineering practice (Youd et al. 2001)

CRR may also be attained from other more recent relationships such as those proposed by Idriss and Boulanger (2006).

3.2.2 *Residual Shear Strength*

Although residual shear strength (S_r) is not discussed in Youd et al. (2001), it is an important aspect in assessing the risk of bearing failure and/or flow slide at a given site. As mentioned in the introductory Section 3.1.1, field experience during past earthquakes indicates that residual strengths (S_r) can be much lower than values obtained from undrained tests on undisturbed samples. The reason for this may be due to upward flow of water associated with excess pore water pressure generation. If this flow is curtailed by the presence of low permeability barrier layers it can cause water to collect in zones beneath the barrier. This may cause some zones or layers to expand to a higher void ratio (void expansion), and hence a lower critical state strength. In the limiting scenario, a water film may form at the interface beneath the barrier (Naesgaard et. al. 2005, Kokusho 2003).

Based on back-analysis of field case histories, Seed and Harder (1990) have proposed upper and lower bounds on residual shear strength as shown in Fig. 3.8.

Olson and Stark (2002) present residual strength in terms of strength ratio (S_r / σ'_{vo}) based on SPT blow-counts as shown in Fig. 3.9. Their values range between about 0.05 and 0.12 for blow-counts in the range 2 to 12. Olson and Stark (2002) have also developed a relationship between residual strength ratio and CPT tip resistance as shown in Fig. 3.10.

It should be noted that the SPT blow-count from Seed and Harder (1990) is based on an equivalent clean sand value that could involve a correction as much as 5 additional blows. The Olson and Stark (2002) values have no such correction and therefore would reflect blow-counts lower than that adopted by Seed and Harder (1990) for the same case histories.

Idriss and Boulanger (2007) recognised the importance of void expansion in causing the low residual strengths obtained from back analysis of field case histories. They propose two residual strength ratio curves; a lower curve for conditions where void expansion effects could be significant, and an upper curve where expansion would be negligible as shown in Fig. 3.11 based on SPT blow-counts. The upper curve has a strength ratio corresponding to the drained strength at $(N_1)_{60}$ = 17, and the lower curve has a residual strength ratio corresponding to the drained strength at $(N_1)_{60}$ of about 30. It should be noted that there are no field data points for $(N_1)_{60}$ greater than about 15. Idriss and Boulanger recommend a clean sand N value representative of average conditions in the layer for use with their chart. Idriss and Boulanger (2007) have also developed a CPT based residual strength ratio chart as shown in Fig.3.12.

It is recommended that potentially liquefiable layers that show significant scatter be characterized by their 66th percentile value.

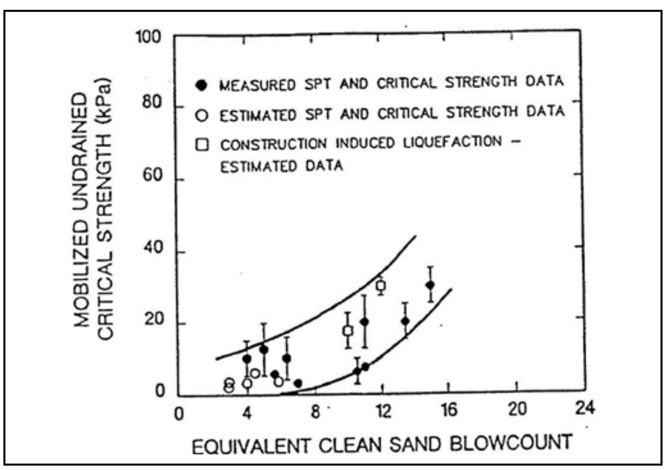


Figure 3.8. S_r and $(N_1)_{60\text{-}CS}$ relationship (Seed and Harder 1990)

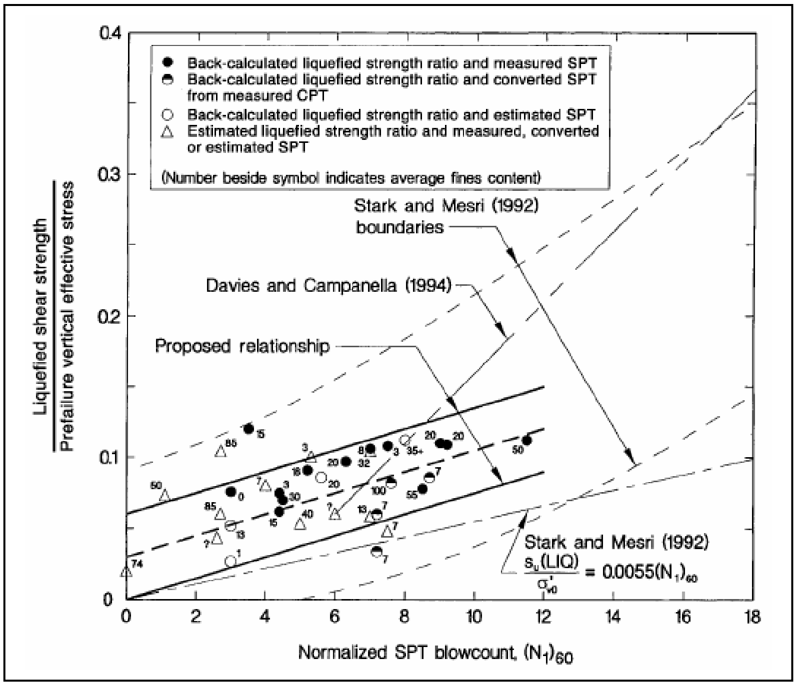


Figure 3.9 A comparison of liquefied strength ratio relationships based on normalized SPT blow count (Olson and Stark 2002)

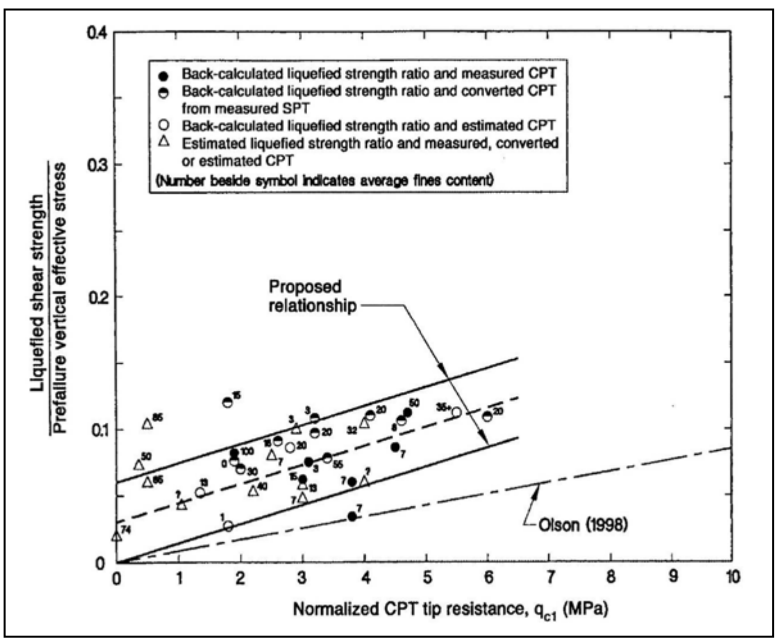


Figure 3.10 A comparison of liquefied strength ratio relationships based on normalized CPT tip resistance (Olson and Stark 2002)

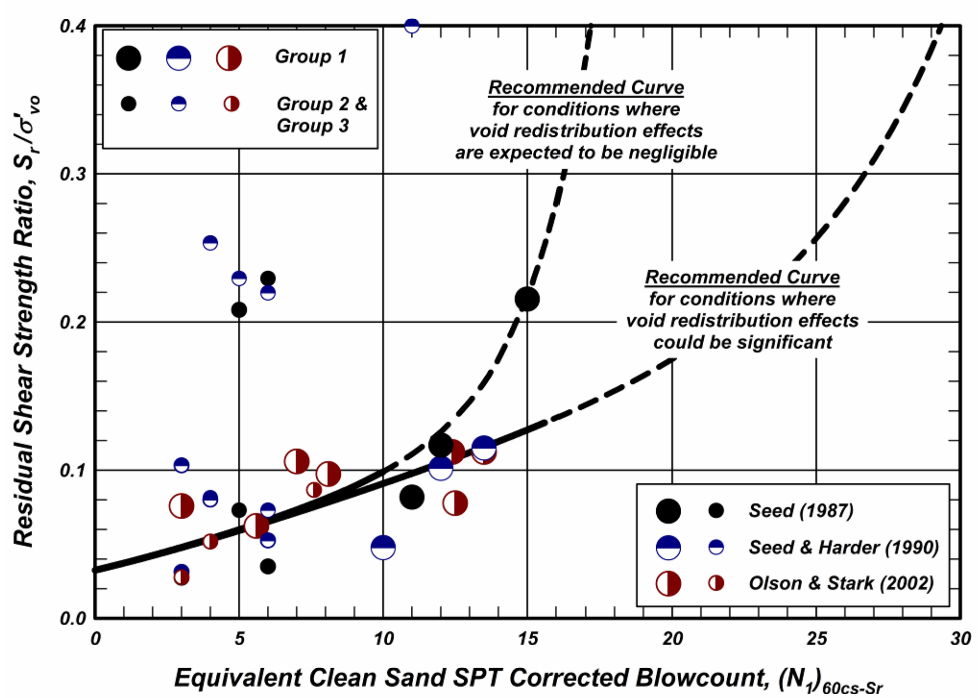


Figure 3.11 Residual strength ratio (S_r/σ'_{vo}) of liquefied soil versus equivalent-clean-sand-corrected SPT $(N_1)_{60}$, for σ'_{vo} < 400 kPa, after Idriss and Boulanger (2007).

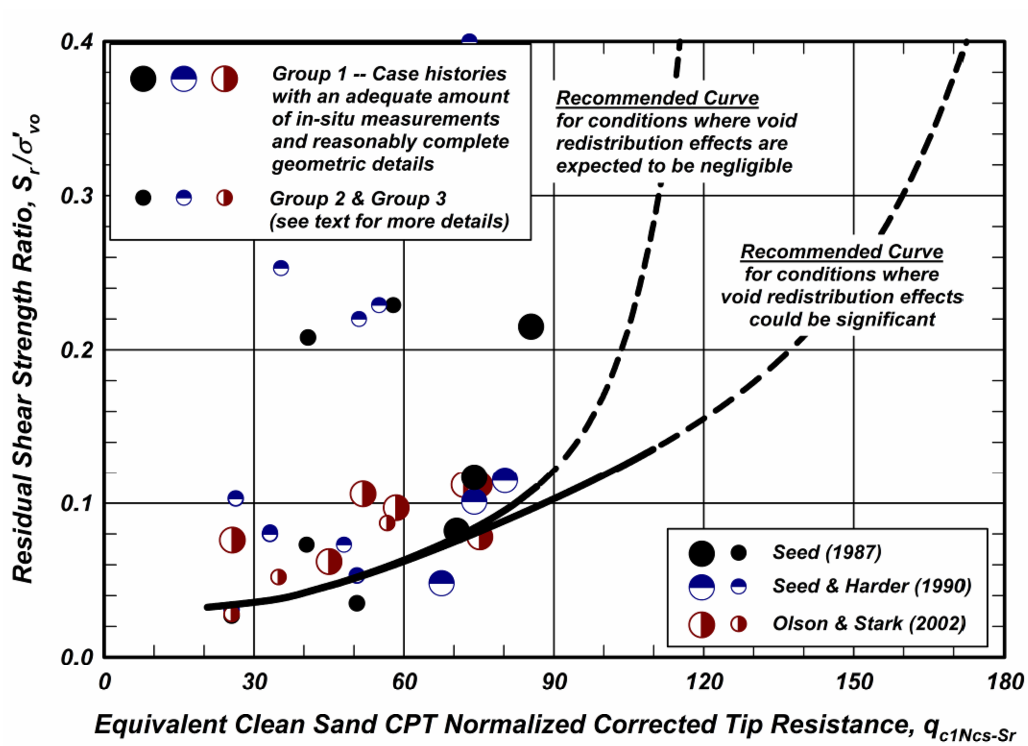


Figure 3.12 Residual strength ratio (S_r/σ'_{vo}) of liquefied soil versus equivalent-clean-sand-CPT normalized corrected resistance, after Idriss and Boulanger (2007).

3.2.2.1 Estimation of Residual strength Using SPT Blow-counts

It is recommended that for zones predicted to liquefy, the residual shear strength (S_r) be estimated from Option I or II as follows:

Option I:

(a) For normalized SPT blow-counts $(N_1)_{60}$ less than or equal to 15, use mean values or lower from Seed and Harder (1990) and/or Olson and Stark (2002) charts.
(b) For normalized SPT blow-counts $(N_1)_{60}$ greater than or equal to 30, use drained strength values.
(c) For normalized SPT blow-count values $(N_1)_{60}$ between 15 and 30, interpolate between the residual shear strength values from (a) and (b) above.

Option II:

Use Idriss and Boulanger (2007) taking into account drainage conditions and void expansion.

Where significant scatter in penetration resistance exists, the liquefiable layer should be characterized by its 66 percentile value (66% of values are greater than the selected characteristic value).

Residual strength may also be estimated from cone penetration test values by converting the cone penetration resistance to an appropriate standard penetration value and using the charts from Seed and Harder (1990).

3.2.2.2 Estimation of Residual strength Using CPT Resistance

It is recommended that for zones predicted to liquefy, the residual shear strength ratio (S_r/σ'_{vo}) be estimated Option I or II as follows:

Option I:

(a) For normalized CPT tip resistance q_{c1} less than or equal to 7.5 MPa, use mean values or lower from Olson and Stark (2002).
(b) For normalized CPT tip resistance q_{c1} greater than or equal to 15 MPa, use drained strength ratio values, (approx. 0.5).
(c) For normalized CPT tip resistance q_{c1} between 7.5 and 15 MPa, interpolate between the residual shear strength ratios from (a) and (b) above.

Option II:

Use Idriss and Boulanger (2007) taking into account drainage conditions and void expansion.

Where significant scatter in penetration resistance exists, the liquefiable layer should be characterized by its 66 percentile value (66% of values are greater than the selected characteristic value).

3.2.2.3 Estimation of Residual strength – General Comments

Residual shear strengths as estimated above can be used in numerical methods to assess post-liquefaction stability and compute lateral displacements. Although excess pore pressure (Δu) can be generated in dense sands [for example $(N_1)_{60} > 30$ or $q_{c1N} > 15$ MPa], the drained shear strength values can be used in design. This is justified since the highly dilative nature of dense sand will cause the pore water pressures to drop to their pre-earthquake values, or lower, as the material strains and gains stiffness and strength. However, if upward drainage is impeded by an overlying barrier layer, the residual strength at the interface can be considerably lower even in dense sands as observed by Kutter et al. (2004) in centrifuge tests.

Kulasingam et al. (2004) show that it is difficult to trigger a flow slide in steep slopes comprised of loose uniform sand, because even loose sand will dilate when sheared, and gain strength at least equal to its drained strength. This finding is a justification for the use of the upper residual strength curve proposed by Idriss and Boulanger (2007), when good drainage conditions are present However, if upward drainage is curtailed by low permeability layers (silt or clay), then expansion and perhaps a water film may form at the base of such layers and reduce the strength to very low values as found by Kokusho (2003) in shaking table tests, and Seid-Karbasi and Byrne (2004) and Naesgaard et al. (2005) in numerical simulations. This process is likely responsible for the low residual strengths reported by Seed and Harder (1990) and Olson and Stark (2002) from back analysis of field case histories. Idriss and Boulanger (2007) chart recognizes drainage effects on post-liquefaction shear strength. Low strengths caused by expansion are consistent with critical state concepts.

Vertical drains can be effective in enhancing upward flow of water during and following soil liquefaction and in preventing expansion and water film development leading to low residual shear strengths. The upper residual strength ratio curves proposed by Idriss and Boulanger (2007) should be used with caution and only when drainage to curtail expansion is assured.

Residual shear strengths can be used to evaluate stability and compute lateral displacements as discussed in Section 4.

3.2.3 *Post-liquefaction Settlements*

Post-liquefaction settlements occur during and after earthquake shaking. For level ground conditions the amount can be computed from the volumetric reconsolidation strains induced as the excess pore water pressures dissipate. Based on field experience during past earthquakes, the amount of volumetric strain depends on penetration resistance and the CSR applied by the design earthquake. Curves proposed by Wu (2002) are shown in Fig. 3.12 and indicate that volumetric reconsolidation strains can range between about 10% for very loose sand to 1% for very dense sands. These curves are recommended for estimating post-liquefaction settlements.

Zhang et al. (2002) have also proposed a method to estimate post-liquefaction settlements using CPT data for level ground sites. If the penetration resistance is available in terms of cone penetration testing (CPT) rather than standard penetration test (SPT) values, then the use of Zhang et al. (2002) method, or that proposed by Wu (2002) after converting CPT data to equivalent SPT values are considered appropriate. Post-liquefaction settlement may also be estimated based on Tokimatsu and Seed (1987) or Ishihara and Yoshimine (1992).

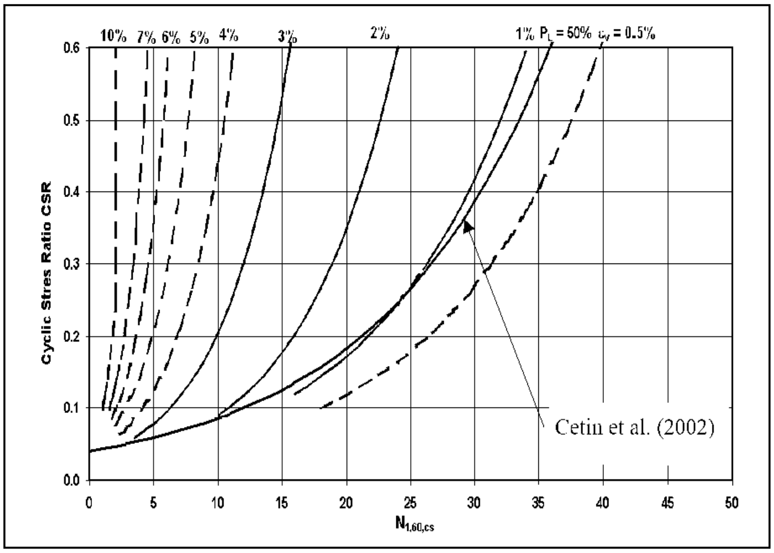


Figure 3.13 Recommended relationships for volumetric reconsolidation strains as a function of equivalent uniform cyclic stress ratio and $N_{1,60,CS}$ for M_w = 7.5 (Wu 2002)

3.3 Recommended guidelines for liquefaction resistance and post-cyclic response – fine-grained soils (silts and clays)

3.3.1 *Cyclic Resistance Ratio*

Youd et al. (2001) indicate no consensus position on the assessment of liquefaction potential of silts and clays.

It has been noted that some fine-grained soils that classify as non-liquefiable according to commonly used empirical "Chinese Criteria" (Wang 1979; Koester 1992; Finn et al. 1994) have in fact experienced liquefaction during earthquakes (Boulanger et al. 1998, Bray et al. 2004). An evaluation of data obtained from laboratory cyclic shear testing of silts also confirm the limitation of the Chinese Criteria as a tool to identify potentially liquefiable soils (Sanin and Wijewickreme 2006a; Boulanger and Idriss 2006). Based on the field performance of fine-grained soil sites in Adapazari following the 1999 Kocaeli (Turkey) earthquake, combined with data from laboratory cyclic shear testing, Bray and Sancio (2006) have proposed alternate empirical criteria to delineate liquefaction susceptibility of fine-grained soils (see Fig. 3.14). As noted in Section 3.1.3, Boulanger and Idriss (2006) have recently recommended that fine-grained soils be classified as "sand-like" (susceptible to liquefaction) if PI < 7, and "clay-like" if PI ≥ 7.

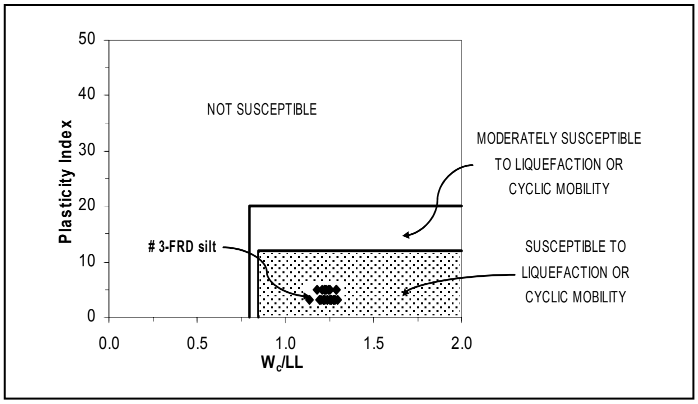


Figure 3.14 Application of the Bray et al. (2004) criteria for liquefaction assessment of the liquefaction susceptibility of Fraser River Delta silt.

Data from cyclic direct simple shear tests on undisturbed samples (Sanin and Wijewickreme 2006a) indicate that the response of low-plastic (i.e. average PI = 4%) channel-fill silt from the Fraser River delta display gradual strain development during cyclic loading. This behaviour is unlike the marked drop in shear stiffness that occurs with the onset of liquefaction that is typical in loose sands.

The current understanding of the seismic behaviour of silts is limited due to lack of laboratory and field data on the cyclic performance of natural silts. It is recommended that the following guidelines be used for the determination of liquefaction potential of silts/clays. As indicated in Section 3.1.3, the guidelines have been generated by combining the information emanating from recent work by Boulanger and Idriss (2006), Bray and Sancio (2006), and Sanin and Wijewickreme (2006a):

1) <u>PI < 7</u>: If the fine-grained material is classified under this category, its liquefaction susceptibility and CRR can be assessed using one of the following two options.

   a. Assume that the material is "sand-like", and follow the guidelines for coarse-grained soils in Section 3.2.1 to determine CRR. However, if the subject soil is similar to the low plasticity channel fill silt from the Fraser River delta [e.g., as noted by Sanin and Wijewickreme (2006a)], this approach will likely lead to a conservative assessment of its liquefaction susceptibility.

   b. Alternatively, undertake site-specific laboratory cyclic shear testing of good quality field samples [e.g., samples obtained using thin-walled tubes with sharpened (i.e., < 5°) cutting edge and no inside clearance] to determine cyclic resistance. Consideration may also be given to using available data from cyclic shear tests on undisturbed silts for the area if such use is judged appropriate after careful review (e.g., Sanin and Wijewickreme, 2006a).

2) 7 < PI < 12: If the material is classified under this category, the strain development mechanism is expected to be gradual and the material is considered less "sand-like" and less likely to liquefy. However, shear strain accumulation and settlements due to pore water pressure dissipation must be examined. Consideration may be given to using available data from cyclic shear tests on undisturbed silts for the area (e.g., Sanin and Wijewickreme, 2006a) if such use is judged appropriate after careful review.

3) PI > 12: The material is assumed to have "clay-like" behaviour where the cyclic strain development is gradual while retaining much of its original strength except may be in the case of sensitive soils (see last paragraph of Section 3.3.2 for further comments with regard to sensitive soil). The strain development mechanism is expected to be gradual, and the degradation of shear stiffness and pore pressure generation significantly lower than for the cases with PI < 12, and essentially considered non-liquefiable for design purposes.

3.3.2 *Residual Shear Strength*

It is recommended that the residual strength (S_r) for silt and clay zones be determined as per guidelines given below:

1) PI < 7: Assume that the material is "sand-like", and follow the procedures for coarse-grained soils to determine residual strength as per Section 3.2.2; or alternatively, determine S_r from site-specific laboratory post-cyclic monotonic shear testing of good quality field samples. If the subject soil is similar to the low plasticity channel fill silt of the Fraser River delta noted above, use of $S_r = 0.8\ S_u$ may be considered, where S_u = static undrained shear strength.

2) 7 < PI < 12: Use $S_r = 0.8\ S_u$;

3) PI > 12: Use $S_r = S_u$.

The above approach (Items 1, 2, and 3 above) essentially employs the understanding developed from research efforts on liquefaction susceptibility of fine-grained soils at UC Davis, UC Berkley, and University of BC. The basic premise is that the full static undrained shear strength (S_u), or most part of it, is considered available as the residual shear strength (S_r) after cyclic loading for soils having PI >7.

Caution should be exercised when applying the approaches under Item (3) of Sections 3.3.1 and 3.3.2 to sensitive and overconsolidated soils. Such soils can reach strain-softening and loss of peak strength at relatively small shear strain levels; however, the definition of transition from peak to remolded strength for strain softening soils under field conditions is difficult, and additional research is necessary to describe this behaviour. As a guide, in cases where the ratio of natural water content divided by the Liquid Limit exceeds 1.0 (or if the sensitivity of soil > 7), it is recommended that a post-peak strength value corresponding to remolded values be used, or the strength value be based on recovery and testing of undisturbed samples (i.e., less than the peak undrained strength) should be used as S_r.

3.3.3 *Post-Liquefaction Settlements*

Unlike the data base for the response of coarse-grained soils (i.e., sands), there is only very limited available information on the post-liquefaction settlements of fine-grained soils. Figure 3.15 presents the volumetric strains observed by (Sanin and Wijewickreme 2006a) for low plasticity channel-fill silt obtained from the Fraser River Delta during post-cyclic reconsolidation of laboratory DSS specimens. It may be seen that specimens that generated high excess pore water pressure ratios [r_u (= $\Delta u / \sigma'_{vo}$) ~ 100%] suffered significantly higher post-cyclic consolidation strains (2 to 4.5%). These volume changes are a reflection of significant changes in the particle fabric associated with large shear strains experienced by the specimens under relatively low effective stress conditions during previous cyclic loading. In the specimens that developed relatively small r_u (<50%), the observed post-cyclic volumetric strains were only in the order of ~0.5%.

As shown in Fig. 3.16 for channel-fill Fraser River Delta silt from Sanin and Wijewickreme (2006a), excess pore water pressure ratio (r_u) depends on the CSR and the number of load cycles. Although based on limited data, there appears to be a threshold CSR of about 0.15 below which the material will not generate large pore pressures and hence will not experience large settlements (unlikely to exceed ~0.5%).

Using these observations as a basis, it is recommended that the post-cyclic reconsolidation strains (ε_v) be estimated as given below.

1) PI < 7: If the subject soil is considered similar to the low plasticity channel-fill Fraser River Delta silt noted above, Fig. 3.16 combined with Fig. 3.15 may be used to estimate post-cyclic settlements using the following steps: (i) obtain the CSR from seismic response analysis; (ii) using this CSR value and the number of equivalent cycles for the design earthquake in Figure 3.16, by interpolation as necessary, estimate a value for the anticipated excess pore water pressure ratio (r_u); (iii) using this r_u value, estimate the potential post-cyclic volumetric strain from Figure 3.15. It is noted that for the design M7 earthquake considered herein the equivalent number of cycles is ten.

2) PI > 7: Presently, there is no data on the post-liquefaction settlements available for the fine-grained soils having moderate to high PI values. It is assumed that the post-cyclic consolidation volumetric strains for such soils are similar, or unlikely to exceed settlements observed for the fine-grained soil class having PI < 7. As such, the procedures suggested in Item (1) may be considered for the estimation of post-cyclic consolidation volumetric strains for soils having PI > 7.

Additional research in this field is needed to confirm the validity of the above approach. Depending on the importance of the project, laboratory post-cyclic consolidation testing of good quality soil samples may be considered to obtain site-specific data.

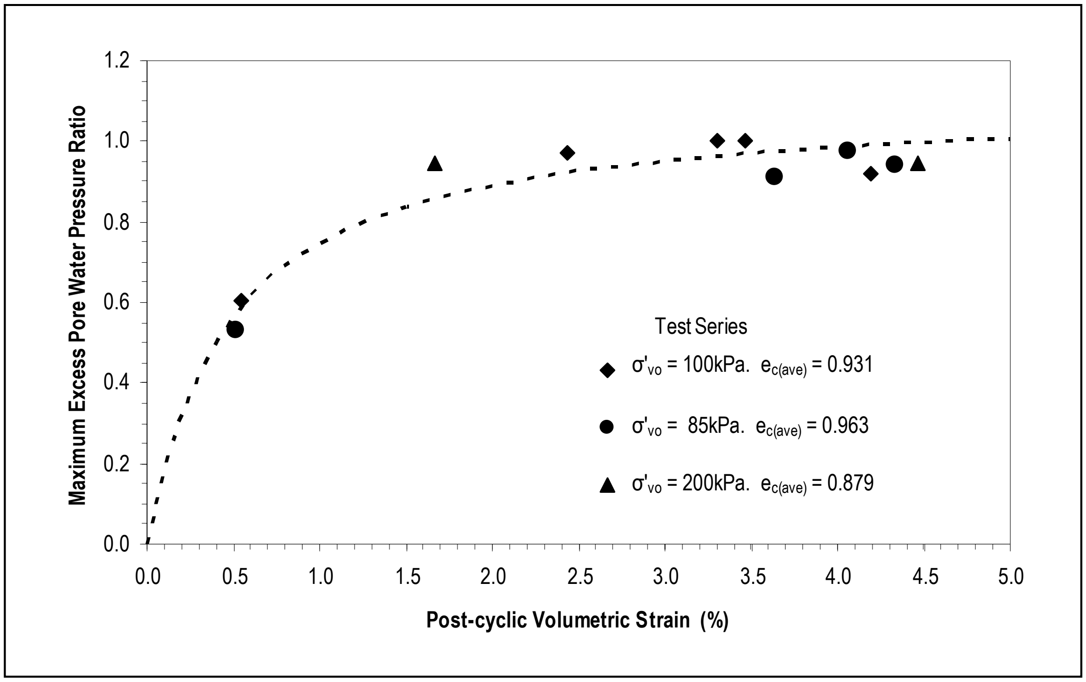


Figure 3.15 Post-cyclic consolidation volumetric strains experienced by normally consolidated channel-fill silt from Fraser River Delta (Sanin and Wijewickreme 2006a).

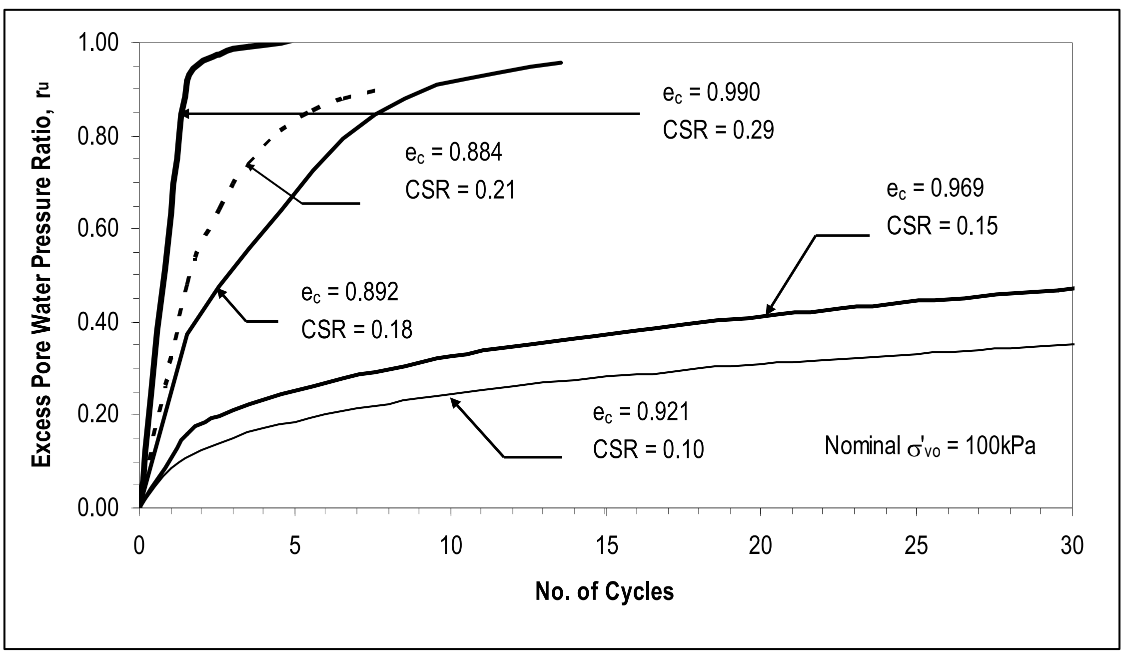


Figure 3.16 Equivalent excess pore water pressure ratio r_u [$= (\Delta u/ \sigma'_{vo})$] with number of cycles during constant volume DSS testing of normally consolidated channel-fill silt from Fraser River Delta (adapted from Sanin and Wijewickreme 2006a).

4 Liquefaction Analysis Procedures

This section discusses analysis procedures for assessing liquefaction triggering and determining induced displacements of the ground or foundations. The procedures are subdivided into categories as follows:

(a) Procedures for assessing liquefaction triggering only;
(b) Empirical correlations for estimating induced ground movements and foundation design;
(c) Numerical procedures which combine triggering assessment and assessment of the consequences.

4.1 Liquefaction Triggering

Liquefaction triggering can be assessed by comparing CSR with CRR. The factor of safety (F_t) against triggering can be computed from:

$$F_t = CRR/CSR \qquad [4.1]$$

The value of CSR can be determined using the Seed-Idriss simplified equation or by conducting a ground response analysis. Ground response analyses have traditionally been carried out using the equivalent-linear total stress analyses; however, more recent hysteretic total stress and effective stress procedures may also be used. There are other methods of liquefaction triggering assessment based on energy considerations but these are not in common usage. CRR can be assessed as described in Section 3.

4.1.1 *Seed-Idriss Simplified Equation*

For routine and non-critical projects, CSR can be assessed using Seed-Idriss Simplified Equation given below:

$$CSR = 0.65\ \sigma_{vt}/\sigma'_{vo} * a_{max} * r_d \qquad [4.2]$$

where, σ_{vt} = total vertical stress at depth of interest
σ'_{vo} = vertical effective stress
a_{max} = peak ground acceleration at ground surface
r_d = a reduction factor with depth to account for soil elasticity

In order to use the simplified equation, reasonable values for a_{max} and r_d are required. These values vary significantly (Cetin et. al. 2004) and therefore it is recommended that a site specific ground response analysis should be conducted for most projects. The simplified equation can be used for minor projects and for projects where values of a_{max} and r_d have been previously determined for similar soils profiles using a site specific ground response analysis. For minor projects, a_{max} at the ground surface can be estimated as outlined in Section 2.3. The values of r_d can be obtained from Eq. 4.3. Equation 4.3 is derived from the r_d values given by Idriss & Boulanger (2006).

$$\text{If } z \leq 4 \text{ m}, r_d = 1; \text{ If } z > 4 \text{ m}, r_d = 1 - 0.015 * (z-4) \geq 0.6 \quad [4.3]$$

where, z = depth below grade in metres.

4.1.2 *Ground Response Analysis Using Equivalent-Linear Total Stress Programs*

Liquefaction triggering is traditionally assessed by conducting an equivalent-linear-total-stress ground response analysis using the 1D program SHAKE (Schnabel 1972). Input for the ground response analysis would be the firm-ground time histories selected as recommended in Section 2.

The analyses can also be conducted in 2D using the program FLUSH (Lysmer et al. 1975) and others. The induced cyclic stress ratio CSR is given by:

$$CSR = 0.65 * (\tau_{max}/\sigma'_{vo}) \quad [4.4]$$

where, τ_{max} = Computed maximum cyclic shear stress on horizontal plane
σ'_{vo} = initial vertical effective stress

Dynamic properties of Fraser River sands and silts obtained from a limited number of cyclic shear tests were reviewed and the following recommendations are given based on these results:

SANDS: The upper-bound modulus reduction curve and the lower-bound damping curve as proposed by Seed et al (1986);

Fine-grained soils (Clayey SILT): The modulus reduction and damping curves as reported by Vucetic and Dobry (1991). The curves will dependent on plasticity index (PI) values; data available from GSC Open File 3356 (Dallimore et al. 1996) indicate the following ranges of PI for the deep marine sediments in the Fraser River delta:

Depths from 25 m to 120m, PI = 8 to 12;
Depths from 120 m to 300 m, PI = 15 to 25;
Depths from 300 m to 320 m, PI = 30 (approx).

The recommendations for Fraser delta soil deposits could be further updated as more data become available from the current research at UBC.

<u>Limitations</u>

- The procedure does not allow yielding of the soil to occur and therefore in weak soils at high levels of shaking, such as that induced by the 2% in 50 year probability of exceedance earthquake, may over-predict the response (CSR, A).
- The method will over-predict damping on small loading cycles and under-predict damping on large cycles.
- The method does not account for the softening that occurs in the soil due to pore pressure build-up and redistribution.

- The modulus reduction and damping curves for shear strains in excess of about 1% have been derived based on limited data and may not be accurate.

The limitations can be overcome by coupled effective stress analyses or, for the first two items, carrying out non-linear total stress analyses. It is suggested that this should be done in addition to the equivalent-linear analyses on critical projects.

4.1.3 *Ground Response Analysis Using Non-Linear Total-Stress Program with Hysteretic Damping*

In the equivalent linear analyses, the same soil damping is used for all cycles throughout the duration of shaking. In reality, however, small strain cycles will have significantly lower damping than high strain cycles. This shortfall can be addressed by using a constitutive model with hysteretic damping. Such models have been developed to run within FLAC (ITASCA 2005) and other programs and can be used to assess liquefaction triggering in both 1D and 2D approximations. The CSR would typically be set equal to 0.65 of the peak value and factor of safety against liquefaction would be calculated using Eq. 4.1. Other advantages of the method are that it can be readily used in 2D analyses and therefore used with sloping ground surface. Structural elements can be included and soil-structure effects modelled if desired.

2D total stress models, which track the dynamic shear stress history within each element and trigger liquefaction if a specified threshold is reached, are also available. This is discussed in section 4.3(a) below.

<u>Limitations:</u>

- Non-linear total stress programs using Masing's rule (Beresnev and Wen 1996) tend to over-predict hysteretic damping at large strains and therefore may under-predict the response.
- Does not address the stiffness or modulus changes that occur due to pore water pressure build-up and flow.
- Analysis procedures not as well established as those for the equivalent-linear (SHAKE) methods and state-of-practice is not as well established.

4.1.4 *Ground Response Analysis Using Non-Linear Effective Stress Programs*

These procedures can be used to assess both liquefaction triggering and the consequences of liquefaction and are discussed further in section 4.3(b) below.

4.2 Simplified approaches for estimating earthquake-induced ground movements and bearing capacity

Some procedures for estimating ground movements and bearing capacity for situations where liquefaction may be triggered are discussed in this section.

4.2.1 *Lateral Spreading Equation by Youd et al (2002)*

Based on field experience during past earthquakes at many sites, Youd et al. (2002) developed equations that give ground displacement as a function of simplified site configurations, soil profile properties, earthquake magnitude (M), and distance (R). For a more detailed description of the lateral spreading models and input data required, refer to Youd et al. (2002). For the Greater Vancouver area and the A2475 earthquake motion the magnitude in the Youd et al equations can be taken as 7.0 (see section 2). The distance R in can be calculated from the attenuation relationship in Youd's paper using the surface peak ground acceleration obtained from the site specific ground response analysis.

4.2.2 *Displacements by Newmark procedure*

In the Newmark method (Newmark, 1965; Franklin & Chang, 1977, Kramer, 1996)), a potential sliding block of soil is treated as a single degree of freedom rigid-plastic system and its movement under seismic loading computed from dynamics. A yield acceleration is calculated using force limit equilibrium methods. Movement only occurs when yield is reached. Total movement of a slope or other structure is calculated by integrating over the earthquake time history or using approximate equations which are generally functions of the yield acceleration, peak ground acceleration and peak ground velocity. Displacements calculated using this method are generally reasonable if the soil behaviour approximates the rigid-plastic behaviour that is assumed in the model.

Liquefied soils do not behave in a rigid-plastic manner and estimates of displacement resulting from limit equilibrium analyses using the soils residual (post-liquefaction strength (see Sections 3.2.2 & 3.3.2) will be very approximate. Items affecting the results include: the point in the time history when liquefaction is triggered, uncertainty in the magnitude of the post liquefaction residual strength, influence of the thickness of liquefied layer, yield acceleration changes with displacement and progression of liquefaction, influence of non-rigid sliding mass, and influence of ground motion incoherence over the length of the sliding mass (Martin et al., 1999). For analyses where liquefaction is a prominent feature of the failure mechanism it is recommended that the Newmark procedure be used as a screening tool only.

A free program NEWMARK can be downloaded from the following USGS site: http://earthquake.usgs.gov/resources/software/slope_perf.php This program will give displacements using both approximate equations and time history integration methods. The program also allows the yield acceleration to be varied as a function of time or displacement (ie. it allows different yield accelerations to be used before and after triggering of liquefaction).

4.2.3 *Post-Liquefaction Settlement*

It should be noted that settlements calculated from the procedures in sections 3.2.3 and 3.3.3 are those induced by consolidation of the liquefied soil only and occur independent of whether or not there is an overlying structure. Footings and other structures founded over or within liquefied soil will also deform due to shear strain within the liquefied soil resulting from stresses induced by the structure (Naesgaard et al., 1998). This shear strain typically occurs during the period of strong shaking whereas the consolidation settlements often occur following the period of strong shaking. The shear strain deformations are additional to the consolidation settlements and can be of similar or greater magnitude.

Shear strain settlements can be calculated using numerical analyses as described in section 4.3. For light structures, one to three storey buildings, founded on a non-liquefiable cohesive crust over liquefied soil, the shear strain deformations can be estimated using Fig. 4.1 from Naesgaard et al. (1998). It should be noted that the correlations in Fig. 4.1 are based on limited total-stress dynamic numerical analyses and are preliminary. For important or more heavily loaded structures, project specific numerical analyses should be carried out. In this figure, the factor of safety against post-liquefaction foundation bearing failure (Fs) is calculated by a similar procedure to that given in 4.2.4(b). The Shear strain induced settlements are in addition to the post-liquefaction consolidation settlements as described in sections 3.2.3 or 3.3.3.

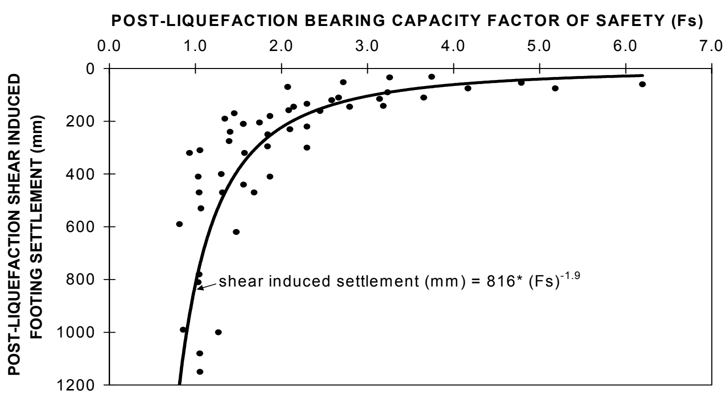


Figure 4.1 Post-liquefaction shear induced settlement vs Post-liquefaction Bearing Capacity F_s. (F_s = (footing perimeter * thickness of crust * shear strength of crust + 5.1 * residual strength of liquefied soil * area of footing) / footing load)

Differential settlements observed in case histories vary extensively. For relatively uniform soil profile across a site it is suggested that a differential settlement between structural supports of 1/2 the total settlement should be used for design (Martin et al., 1999). Alternatively the calculated differential settlement (between varying soil profiles and footing configurations) at the site plus some allowance for uncertainty can be used.

It is recommended that more detailed numerical analyses and/or ground improvement be considered for heavier structures that have foundations overlying liquefiable soil.

4.2.4 *Bearing and Punching Capacity of Spread Foundations over Liquefied Soil*

Liquefaction can result in the loss of bearing of spread foundations. This may involve either direct bearing failure of foundations founded on liquefied ground or more commonly the 'punching' of the foundation through an overlying non-liquefied crust into the weak

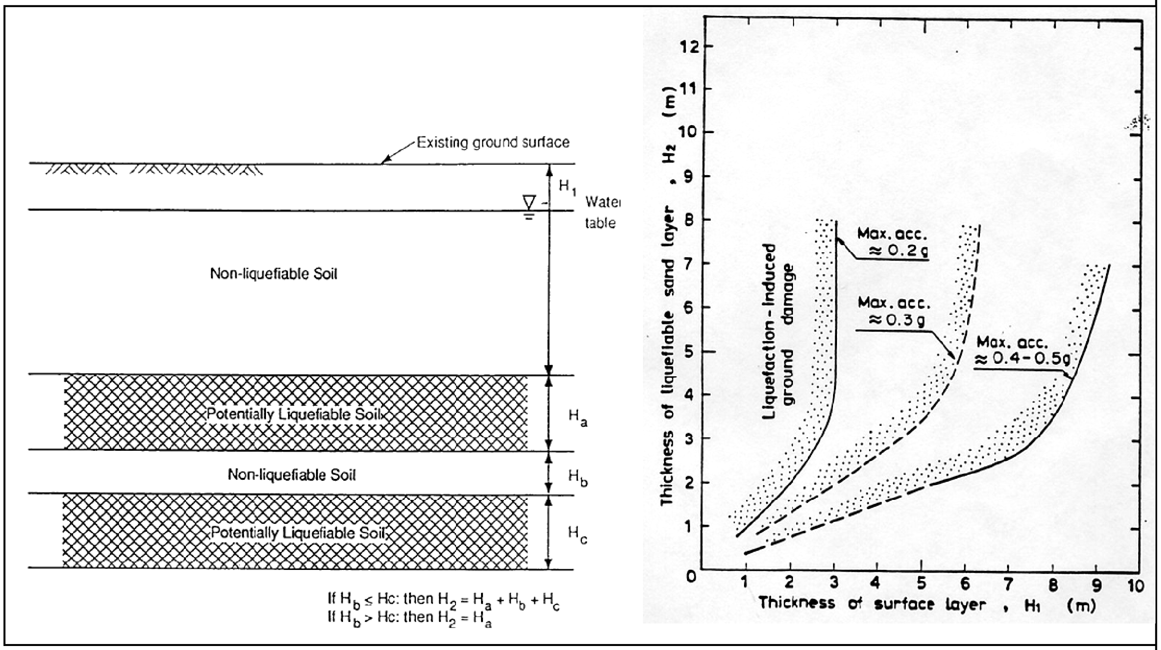


Figure 4.2 Boundary curves for site identification of liquefaction-induced damage (from Ishihara, 1985).

liquefied ground. There are no recognized analytical methods to evaluate the loss of bearing capacity at this time.

The liquefaction manifestation correlations by Ishihara (1985) can be considered for light (one and two story frame) structures. Ishihara developed charts as shown in Fig. 4.2, correlating surface manifestation of liquefaction (surface rupture and sand boils) as a function of the thickness of the liquefied layer and thickness of the overlying non-liquefied crust. If the site plots to the left of Ishihara's boundary then more detailed analyses or ground improvement should be considered. Ishihara's liquefaction manifestation chart is not applicable for buildings on sloping ground where lateral spreading may occur (Martin et al., 1999).

Other simplified procedures used by local practitioners include:

(a) Setting the foundation punching resistance such that the shear strength of the non-liquefied crust around the perimeter of the foundation is greater than the foundation load, or

(b) Setting the foundation punching resistance such that the shear strength of the non-liquefied crust around the perimeter of the foundation plus the bearing capacity of the underlying liquefied soil (using the residual strength from Section 3.2.1 & 3.3) is greater than the foundation load. In using this procedure it should be noted that: (i) the residual strength from Section 3.2.2 or 3.3.2 derived from back-analysis of slopes may not be indicative of the residual strength of liquefied soil underlying footings; and (ii) large shear strains in the order of 30 to 50% may be required for the residual strength to develop.

For important structures and structures with large loads it is recommended that numerical analyses be conducted to determine bearing capacity.

4.2.5 *Deep Foundations and Liquefaction*

Deep foundations extending through liquefiable soils will require special considerations.

- The ability of the foundation to resist lateral loads applied by the structure to the foundation may be reduced;
- The bearing (and uplift) capacity of the deep foundation may be reduced due to reduction in soil strength;
- Settlement of the liquefied soil may cause down-drag or negative friction forces on the deep foundation;
- Liquefaction-induced soil movements may cause lateral loading that will displace or fail the deep foundation.

The inertial loading from the structure and loading from kinematic differential (free-field) soil movements within the ground due to liquefaction may coincide, particularily for strong shaking where liquefaction occurs early on. This was observed in centrifuge tests and numerical analyses (Boulanger et al. 2003, 2007) and was also noted in numerical analyses for local bridge projects using 2475 year return earthquake records. Guidance on combining inertial and free-field kinematic ground displacement loading for pseudo-static design procedures can be found in Boulanger et al. (2007).

If the effects of liquefaction cannot be adequately accommodated in deep foundation design, consideration should be given to mitigation of the liquefaction using ground improvement, drainage, or containment methods (see section 6).

Procedures for analyzing pile foundations within liquefied ground include:

4.2.5.1 Limit equilibrium calculations

Deep foundation capacity estimates can be made using relatively simple force equilibrium methods. For axial compression and uplift capacity calculations, it is suggested that liquefied soil adjacent to the foundation shaft should be assumed to have zero strength. For lateral design of pile foundations the Japanese Road Association method can be used. (JRA, 1996). In this method the passive capacity of the non-liquefied crust and a pressure from the liquefied soil equal to 30% of the overburden pressure is applied as a static load on the pile. Embedment in underlying non-liquefied soils or forces from the structure is required to resist the loading.

4.2.5.2 P-Y / T-Z Analysis methods

In these methods the soil interaction with the deep foundation is made through non-linear P-Y (lateral) and T-Z (vertical) soil springs. These analyses are usually conducted using programs such as LATPILE (Byrne et al. 1984) and LPILE & GROUP (Ensoft, 2004). Free-field ground displacements due to soil liquefaction can be accommodated in some of the programs by applying the displacements to the ends of the soil springs. Discussion on the development of P-Y curves for liquefied soil can be found in (Rollins et al. 2005; and Boulanger et al. 2003, 2007).

4.2.5.3 Numerical methods

Numerical methods as described in section 4.3 can also be used to model piles within liquefied soil regime. Dynamic numerical methods may be used to determine the free-field ground displacements that can then be used in a P-Y / T-Z type analyses or dynamic analyses that include both the soil and structure may be carried out. In the later case both displacements and structural demands are determined from the single analysis.

4.3 Numerical procedures for assessment of liquefaction triggering and consequences of liquefaction

Total-stress and effective-stress numerical methods are available for assessing triggering of soil liquefaction and liquefaction consequences (displacements, stresses, etc.). Most of these methods comprise 2D dynamic analyses, however 1D and 3D methods are also available. Typically, earthquake time histories are used as input ground motions in these analyses. These procedures have the advantage that complex soil profiles and soil-structure interaction effects can be analyzed.

4.3.1 *Elastic-Plastic Total-Stress Dynamic Analyses*

In this procedure the dynamic shear stress history is tracked within each element and if a specified threshold is reached it triggers liquefaction by changing the soil element properties to post-liquefaction values. The UBCTOT model (Beaty and Byrne 1999; Beaty 2001) is run within FLAC using a modified Mohr-Coulomb constitutive model and Rayleigh damping. The liquefaction triggering threshold is set to give triggering similar to that obtained from using the Seed method (Option 1 analyses). Post-liquefaction stiffness is generally one or two orders of magnitude less than pre-liquefaction, and post-liquefaction strength is often set to the soil residual strength obtained from empirical correlations with soil density or cohesion (see Section 3.2.2 & 3.3.2).

This option gives similar initial triggering to that obtained using 1D equivalent linear (SHAKE & FLUSH). The model has two distinct differences from the equivalent linear method (i) is that once initial liquefaction has occurred base isolation effects may reduce triggering liquefaction in other zones and (ii) the model allows progressive liquefaction or stress transfer to occur (i.e. If a local zone liquefies the shear stresses that it once supported are transferred to adjacent zones which in turn may cause liquefaction to trigger in those zones). Both base isolation effects and progressive liquefaction are real effects of soil liquefaction.

The effects of soil-structure interaction and ground improvement are easily included in the analyses, however pore water flow and pore pressure re-distribution are not directly included in the analysis. They are allowed for in a very crude way by using residual strengths back-calculated from case histories. Post-liquefaction consolidation settlements would be additional to the displacements obtained using this procedure.

4.3.2 *Effective stress dynamic analysis*

There are several programs suitable for this; examples include: FLAC (with UBCSAND (Byrne et al. 1995; 2004; 2006; Puebla et al., 1997; Beaty & Byrne, 1999), TARA (Finn et al., 1986), DYNAFLOW (Prevost, 1981), and several others. Liquefaction is triggered directly by the program and corrections for magnitude, confining stress and static bias are typically not required. Often the programs are coupled and simultaneously consider the effects of pore pressure generation and re-distribution by flow. The programs typically will not give a factor of safety against liquefaction triggering. The effective-stress dynamic analyses combine both liquefaction triggering and assessment of the consequences. The effects of soil-structure interaction and ground improvement are easily included in the analyses. Pore water flow and pore pressure re-distribution, including the effects of low permeability layers or barriers and drains, can also be allowed for in the models. These models have been calibrated by back-analyzing laboratory tests, centrifuge and shaking table model tests and case histories.

There is concern that current effective stress models may not correctly model the post-liquefaction soil strength. Therefore following completion of earthquake shaking it may be prudent to check post-earthquake stability by setting the strength of zones which liquefied to their 'residual strength' (see Section 3.2 & 3.3) and then solving for static equilibrium. Elements deemed to liquefy are those having a maximum excess pore water pressure ratio $r_u \geq 0.7$.

These procedures are considered state-of-the-art/practice for estimating liquefaction induced displacements, soil-structure interaction effects and effectiveness of remedial measures.

<u>Limitations:</u>

- Pore pressure build-up and liquefaction can result in a base-isolation that will drastically decrease the response following the onset of liquefaction, especially in 1D analyses. In 2D and 3D analyses the base-isolation effect may not be as dramatic due to spatial variation of soil properties. While this effect is physically correct, small changes in soil stratigraphy, permeability and density may have a significant effect on locating where liquefaction is triggered.
- Analysis procedures are not as well established as those for the equivalent-linear (SHAKE) methods and state-of-practice is not as well established.

5 Tolerable Displacements

5.1 General Comments

In NBC 2005, the philosophy for earthquake design is to accept damage to structures. The design earthquake is considered to be a very rare event with a return period of 1 in 2,475 years (A2475). The expectation is that "typical" or "normal" structures will be near collapse, will have little lateral reserve strength left, and will have experienced lateral drifts during the earthquake of not more than 2.5%. The structures themselves will be designed either to accommodate drifts up to the 2.5% limit or they will be stiffened so as to reduce the structural drift to within limits that the structure can accommodate. Drift is defined as the difference between horizontal displacements of adjacent floors divided by the height between floors and is a measure of horizontal "distortion" of the structure

Implicit within NBC 2005 is the intent to limit damage during low to moderate level earthquake shaking. However, the code does not specifically define the return period of low to moderate shaking, the force levels, or the drift limits. One objective of the Task Force and this Guideline is to provide additional information and guidance regarding performance objectives, earthquake shaking return periods and associated limits on differential structural displacements that are implied for structures by the NBC 2005 requirements.

5.2 Performance Levels and Performance Objectives

Performance objectives and structural performance levels relate desired building functionality, earthquake return periods, and drift limits. This is described by DeVall (2003). In general, structures designed to NBC 2005 satisfy the performance levels and goals presented in documents such as the Structural Engineers of California Vision 2000 (SEAOC 1995) document and the Federal Emergency Management Agency of the United States document No. 356 (FEMA356 2000). Performance levels and objectives discussed in these documents are summarized below for information and to help in understanding the discussions and recommendations contained in this document.

5.2.1 *Structural Performance Levels*

5.2.1.1 Near Collapse - Collapse Prevention Structural Performance Level

A post-earthquake state in which the building is on the verge of partial or total collapse. Substantial damage to the structure has occurred, potentially including significant degradation in the stiffness and strength of the lateral-force-resisting system, large permanent lateral deformation of the structure, and, to a more limited extent, degradation in vertical-load-carrying capacity. However, all significant components of the gravity-load-resisting system must continue to carry their gravity load demands. Significant risk of injury due to falling hazards from structural debris may exist. The structure may not be technically practical to repair and is not safe for re-occupancy, as aftershock activity could induce collapse.

5.2.1.2 Life Safe - Life Safety Structural Performance Level

A post-earthquake state in which significant damage to the structure has occurred, but some margin against either partial or total structural collapse remains. Some structural elements and components are severely damaged, but this has not resulted in significant falling debris hazards, either within or outside the building. Injuries may occur during the earthquake; however, the overall risk of life-threatening injury as a result of structural damage is expected to be low. It should be possible to repair the structure; however, for economic reasons this may not be practical. While the damaged structure is not an imminent collapse risk, it would be prudent to implement structural repairs or install temporary bracing prior to re-occupancy.

5.2.1.3 Operational - Immediate Occupancy Structural Performance Level

A post-earthquake state in which only very limited structural damage has occurred. The basic vertical and lateral-force-resisting systems of the building retain nearly all of their pre-earthquake strength and stiffness. The risk of life-threatening injury as a result of structural damage is very low, and although some minor structural repairs may be appropriate, these would generally not be required prior to re-occupancy.

5.2.2 *Structural Performance Objectives in NBC 2005*

5.2.2.1 Basic Objective [NBC 2005 "Normal" Importance]

Minimize loss of life but accept a "Near Collapse" building performance level as defined in 5.2.1 for the A2475.

5.2.2.2 High Importance [NBC 2005 "High" Importance]

Provide an increased level of life safety and structural damage control than that provided by the Basic Objective for buildings such as schools. The building would not immediately be "operational", as defined in 5.2.1.

5.2.2.3 Essential Service Objective [NBC 2005 "Post Disaster" Importance]

Provide a building with less damage and higher probably of being at an "operational" building performance level as defined in 5.2.1 for the A2475 than a building satisfying the basic objective.

5.3 Structural Drift Limits as per NBC 2005

Structural Engineers are concerned with lateral displacement of the structure due to earthquakes, and the resulting drifts.

The NBC 2005 limits the maximum drifts for structures for the A2475 to:

- Post Disaster Buildings (Post Disaster Importance) 1.0%
- Schools and Community Centres (High Importance) 2.0%
- For all other Buildings (Normal Importance) 2.5%

Table 5.1 shows suggested guidelines between performance objectives, maximum allowed drifts, and return periods. This is a slightly modified summary taken from the VISION2000

(SEAOC 1995) and FEMA356 (2000) documents. The table shows that a building designed for the Basic Objective category, denoted in the table by B, should not have drifts exceeding 2.5% for the very rare hazard defined as the A2475 event. Such a structure should only have drifts of 1.5% for the rare (A475) hazard, and be operational for smaller hazards. The NBC 2005 drift limits are slightly different from those shown in the table, and the High Importance category is not shown but would fall between the B and E categories.

Note that the NBC 2005 drift limit of 1.0% for Post Disaster Buildings at the A2475 hazard is more restrictive compared to that recommended in Table 5.1 for the A2475 Essential Service Objective, resulting in structures that are closer to 'operational' than to 'life safe'.

Since these limits are for the A2475, they are actually more restrictive than the limits in NBC 1995 which are for the A475, reflecting the intent of NBC 2005 to reduce drifts in order to help minimize damage.

Due to the nature of earthquake forces on buildings, return periods, and structural behaviour, it can be shown that, in general, these NBC 2005 drift limits for the A2475 earthquake also satisfies the other limits for various return periods and structural performance levels as given in Table 5.1.

Table 5.1 Performance objectives, drift limits, and earthquake return periods suggested in VISION2000 (SEAOC 1995) and FEMA356 documents (slightly modified).

		FULLY OPERATIONAL	OPERATIONAL	LIFE SAFE	NEAR COLLAPSE	PROBABILITY OF EXCEEDANCE
Drift Limit		(0.2%)	(0.5%)	(1.5%)	(2.5%)	
EARTHQUAKE DESIGN LEVEL (RETURN PERIOD)	Frequent (43 years)	**B**	**U**	**U**	**U**	70% in 50 years
	Occasional (72 years)	**E**	**B**	**U**	**U**	50% in 50 years
	Rare (475 years)	**S**	**E**	**B**	**U**	10% in 50 years
	Very Rare (2475 years)		**S**	**E**	**B**	2% in 50 years

Legend:
U Unacceptable Performance for New Construction
B Basic Objective – NBC 2005 Normal Importance
E Essential Service Objective – NBC 2005 Post Disaster Importance
S Safety Critical Objective - No Proposed NBCC Category

5.4 Ground Displacements and Differential Settlements

In general, horizontal and vertical foundation displacements are ignored in building design for earthquake conditions as they are usually small and do not have much effect on the structure. However, this may not be true where buildings are founded on soils prone to liquefaction. In this case there may be significant vertical and horizontal displacements of the ground and the NBC 2005 states that these must be accommodated in the design of the foundations and the building. A detailed description of liquefaction is given in section 1. A brief summary to assist in understanding this chapter follows:

Liquefaction is the result of seismic shaking of the ground, which predominantly affects saturated loose fine-grained sands, and to a lesser extent, silts and coarse-grained soils such as gravels. Liquefaction occurs as a result of a build-up of excess pore water pressure in the soils. The shear stiffness may be temporarily reduced to almost zero, and this could result in significant loss of soil bearing capacity and subsequent failure and/or lateral and vertical displacement of some building foundations.

The increased pore water pressure eventually dissipates after the shaking stops but the soil particle matrix now has a reduced volume. The loss of volume will result in subsidence of the ground surface. Ground surface settlements due to the compaction are typically in the order of 2% to 4% (but may be much larger) of the thickness of the liquefied layer. Differential settlement can arise because of variability in the liquefied soil layer, and typically has been taken as 50% of the total settlement. This settlement will take place after the major shaking as it takes time for the excess pore pressures to dissipate. In addition, the surface soils supporting a building structure over a liquefied soil stratum may experience large horizontal displacement and spreading, particularly if there is a ground surface slope.

Additional settlement may occur under loaded footings during the earthquake caused by shearing of the soil directly under the footing (in conjunction with reduced shear stiffness). The more heavily loaded the footing the more settlement can be expected, leading to increased differential settlement.

5.5 General Discussion on Liquefaction and the Effect on the Building Structure

The following is a non-exhaustive list of items or questions that should concern the designer of structures in liquefiable areas. These are not in any order of importance but are inserted here to alert the designer to some of the issues that typically do not occur when designing structures on other types of sites. The next section will give some guidelines on how to design for some of the issues.

1) Dense or densified sites - If the site is not expected to liquefy in the A2475 year event then there are no liquefaction issues that need to be addressed.
2) If the site liquefies, then what are the expected horizontal and vertical ground displacements? Can the structure and its foundations tolerate them? Do the resulting building deformations satisfy the performance level expectations and performance objectives?

3) There is considerable uncertainty in estimates of ground displacements due to liquefaction. Methods of predicting liquefaction induced displacements are discussed in section 4. Vertical post-liquefaction consolidation settlement can be approximated from empirical correlations based on past experience. Shear induced settlement and differential vertical displacements are more difficult to estimate. Often differential settlement is taken as half the total. Horizontal displacements due to liquefaction are difficult to predict and may be estimated using empirical correlations based on field experience and/or from sophisticated analysis as described in section 4.

4) Some liquefiable soils are overlain by a non-liquefiable soil cap that can support footings vertically and provide lateral support to footings, piles, and pile caps. Assessment of such sites must account for the interaction of the soil cap with the liquefiable soils below.

5) Liquefaction can cause loss of vertical load carrying capacity. For instance, piles may lose their vertical load capacity if they are founded within the liquefiable zone. Pad footings supported on non liquefiable surface soils may "punch through" to the liquefied stratum below.

6) Horizontal and vertical ground displacements affect the building differently. Differential (not total) vertical displacements affect grade supported building structures. Uniform horizontal ground displacements will affect the piles. Differential horizontal ground displacements will try to cause the foundations to spread apart or may damage the "tie" structure and vertical load carrying system within the building structure.

7) While shear-induced ground displacements due to liquefaction take place during the shaking, volume change (consolidation) induced displacements take place largely after the strong earthquake shaking of the building stops. The lateral ground displacements, although cyclic, generally accumulate in one direction due to sloping ground or other static shear stress bias and therefore may not be as demanding on the structure compared to pure cyclic loading.

8) Lateral displacements of the building structure due to shaking tend to be somewhat proportional to earthquake intensities. Building structures are therefore expected to behave "quite a bit better" in the A475 event than in the A2475 event. It is not clear that this proportionality holds for liquefaction induced ground movements, as there tends to be a threshold at which liquefaction is triggered and the displacements may suddenly become significant. However, higher intensity shaking will result in a greater depth of liquefaction and greater settlement, and this causes larger displacements so the A2475 displacements will always be more severe than the A475 displacements.

9) Generally, if no liquefaction occurs, structures tend to return close to their initial configuration after earthquake shaking with residual displacements much less than the maximum experienced during the earthquake, unless the structure has a bias in one direction or a P-delta problem.

10) Some building sites may be designed to undergo large differential vertical displacements due to long term gravity loads. This will reduce the structures ability to accommodate further differential settlements due to liquefaction.

11) Footing rotation can affect the structure. However, typically, footings will settle vertically with little rotation, and in some cases footing rotation could alleviate forces arising from settlement.

12) NBC 2005 requires that the foundations of buildings subjected to earthquake loads be stronger than the superstructure. Where displacements due to liquefaction take place, the code states that the effects of liquefaction must be accounted for in the design. The code is mute on horizontal drift limits for piles, and whether or not piles are allowed to yield in flexure under lateral deflections due to liquefaction.

13) The design of the foundations must be such that the earthquake loads can "get out" of the building without "ploughing" through weak soil at the foundation level. This is of concern where some parts of the foundation could move relative to other foundation parts that do not move. For other types of structures where the foundation can resist relative movements, plough through the soil or sliding of the entire structure may not be of concern for collapse prevention.

14) Depending on the expected earthquake shaking, NBC 2005 requires the pile caps and footings on soil sites E and F to be tied together. This helps control the impact of horizontal ground displacements on the building, and may prevent lateral spreading from occurring within the building. Care must be taken to account for the potentially large passive soil pressures against individual footings and pile caps if lateral spreading is to be resisted.

15) For earthquake design of structures NBC 2005 uses load factors of 1.0 or less, depending upon the load. When calculating the material or element resistance capacities, NBC 2005 and the Materials Standards use:

    - Capacity design principles, where the designer locates those regions in the structure where non-linear behaviour under earthquake loading is to take place, and where the rest of the structure remains elastic.
    - The non-linear regions are specially detailed to accommodate non-linear cyclic behaviour. The resistance calculated for these regions recognizes *expected* strengths and in effect uses material resistance factors greater than 1.0 applied to the *specified* strengths. This is known as utilizing the structural over-strength in the non-linear region.
    - The remainder of the structure is designed to remain elastic and to force the inelastic behaviour into the pre-determined non-linear locations. These elastic regions require only *typical* detailing and are designed using the usual material resistance factors which are less than 1.0.

5.6 Recommended Approach to Address Some of the Issues Raised in 5.5

1) Building displacements from ground movement should be compatible with expected building performance levels at various earthquake return periods.
2) The maximum displacements due to earthquake shaking of the building tend to be non-concurrent with the maximum induced vertical ground displacements due to liquefaction since the post-liquefaction consolidation settlements will tend to occur after strong shaking.
3) Differential vertical displacements used for earthquake design should include any long term gravity load components.
4) Generally, if liquefaction does not occur structures tend to return close to their initial configuration after an earthquake and the residual displacements are often much less than the maximum lateral displacement during the earthquake, particularly for lower level earthquakes. For the purpose of assessing the combined effect of structural displacements above grade due to earthquake shaking of the building followed by vertical soil displacements due to liquefaction, it is reasonable to assume that the residual lateral displacements of the building due to earthquake shaking are about 20% of the maximum displacement permitted for the A2475.

The above displacements from the shaking "distort" the building laterally and if we assume they come back to about 20% of the allowed maximum for the A2475, it allows for more "distortion" which may come later from either the residual lateral or vertical displacements of the foundation.

Applying the above reasoning gives an allowance for the vertical displacement distortion or "vertical drift ratio" of 80% of the code "horizontal drift ratio". Vertical drift ratio is defined as the difference in vertical displacement of a column relative to its adjacent column divided by the column spacing. The effect of vertical drift, on say beam moments of a frame, is essentially the same as horizontal drift. Thus vertical drift limits are related to horizontal drift limits in this guideline.

For a "regular" or "normal" building that is allowed a 2.5% drift limit with say a 10 m bay size or 10 m span, the allowable maximum vertical differential deflection would then be 10,000 x 0.025 x 0.8 = 200 mm.

It is recognized that the differential settlement caused by the variability in the soil conditions may also depend on the horizontal spacing of the components, and it is likely to be less for small column spacings. There is very little experience with this problem, as evidenced by the rather crude assumption that differential vertical settlement is approximately half of the total settlement. It is therefore suggested that for the purposes of determining the vertical drift due to differential ground settlement, a column spacing of not less than 10 metres should be assumed. If some columns are more heavily loaded than others, this may result in additional variations in differential settlement and must be addressed by the geotechnical engineer. Where it is possible for the geotechnical consultant to provide estimates of potential vertical settlement at the column locations, then the actual column spacing should be used in the calculations.

5.6.1 *Recommendations for Ground Performance Displacement Limits*

The post earthquake ground displacement for occasional earthquakes, such as the A72 or A100, are expected to be very small. The A100 earthquake in the Vancouver region generates about 0.1g peak ground acceleration, which will generally not result in any significant liquefaction in most areas. As a consequence, no review for liquefaction effects is typically needed and the expected residual ground displacements should be negligible.

a) Vertical Displacements

<u>A2475</u> – Limit the vertical drifts to 80% of the NBC 2005 drifts as discussed in 5.6.5 above. This gives maximum vertical drifts of:

- Post Disaster Importance 1% x 0.8 = 0.8%
- High Importance 2.0% x 0.8 = 1.6%
- Normal Importance 2.5% x 0.8 = 2.0%

<u>A475</u> – Drift limits are not explicitly addressed in the NBC 2005, but it implicitly meets the limits suggested in VISION2000 (SEAOC 1995) and FEMA356 documents, as discussed in Section 5.2 and 5.3. From Table 5.1, the suggested drift limit for “normal” buildings at A475 is about 60% of the limit for A2475. Based on this, it is recommended that 60% of the full A2475 values be used for the A475, as the expected residual structural displacements at A475 should be very small. This gives limiting vertical drift values of:

- Post Disaster Importance 1% x 0.6 = 0.6%
- High Importance 2% x 0.6 = 1.2%
- Normal Importance 2.5% x 0.6 = 1.5%

These are maximum limits and it is possible that the structure may not be able to accommodate them. If the structure cannot tolerate the displacements, then the structure will require improved detailing, different kinds of foundations, or the expected ground displacements must be reduced by soil improvement. A good source of information for assessing structures for displacements of these magnitudes is FEMA356.

b) Horizontal Displacements

Horizontal displacements typically only have structural implications for piles. For rafts and adequately tied together spread footings, where the buildings tend to move as rigid bodies with the horizontal ground displacements, there are virtually no effects on the above grade structure unless the horizontal displacements lead to additional non-uniform vertical displacements. However, horizontal displacements may lead to lateral spreading (differential horizontal displacements), especially for buildings with large plan area.

- NBC 2005 and earlier versions of the code have long stated that foundations should not yield before the superstructure. However, it is mute about drift limits or yielding of piles due to lateral ground displacements due to liquefaction which takes place essentially monotonically, with the maximum usually occurring at the end of or following the earthquake. In some cases the maximum deformation can occur during the earthquake, and guidance on combining inertial and free-field kinematic ground displacement loading for pseudo-static design procedures can be found in Boulanger et al. (2003, 2007). NBC 2005 simply states that the effects of liquefaction shall be accounted for in the design. Based on this, the recommendations are:

- Allow some pile yielding under lateral deflections due to liquefaction, the degree of which depends upon the earthquake return period and ductility of the pile
- Allow larger lateral horizontal drifts for piles than is allowed for the building itself as there is no "building content" damage that needs to be considered – it is essentially only the structural performance of the pile that is of interest. Suggested maximum pile drift values are:

A475	– Normal and High Importance	– 5%
	– Post Disaster	– 1.5%
A2475	– Normal and High Importance	– Governed by ductility of the pile
	– Post Disaster	– 2.5%

But in all cases the drifts must be less than the inelastic drift capacity of the pile.

Recommendations for limits for differential and total displacements are summarized in Table 5.2. See also Figs. 5.1, 5.2, 5.3 for illustrations of some typical cases.

Table 5.2 Recommended drift limits for ground displacements

	Piles	Raft/Spread Footings
Importance	Δ_H/ℓ_p	δ_V/ℓ
A475		
Normal	5%	1.5%
High	5%	1.2%
Post Disaster (d)	1.5%	0.6%
A2475		
Normal	(e)	2.0%
High	(e)	1.6%
Post Disaster (d)	2.5% (f)	0.6%

Notes:

a. $\Delta_{H,}$ = Total horizontal displacement, see Fig 5.1

b. ℓ_p = Length of pile between points of fixity – See Fig. 5.1.

c. δ_V/ℓ = Vertical differential displacement divided by distance "ℓ" between measuring points. See Figs. 5.2 and 5.3. δ_V includes long term gravity effects.

d. Essential Post Disaster mechanical, plumbing, and electrical services may need to be able to tolerate displacements.
e. Determined by ductility and deformation capacity of the pile.
f. Pile must have sufficient ductility.

It is important to note that while the residual above grade structural displacements will typically be less than the maximum displacement during shaking, the soil displacements will be essentially monotonic and the ground displacements will tend to be at a maximum toward the end of shaking. This may not add much to the building damage but it will probably result in the building being much more distorted or out of plumb after the earthquake than if it was on firm ground. This will make repairs more difficult and costly. While it may not be a concern for the A2475 event where severe damage (near collapse) is expected, it may be an issue for the A475 event. For one or two storey structures, it may be relatively easy to "re-level" the columns and walls vertically compared to "re-plumbing" them horizontally due to lateral displacements. Taller, heavier buildings will probably be on piles or on densified sites where vertical displacements due to liquefaction will not be an issue.

c) Foundation ties and diaphragms need to be designed to resist any differential horizontal "spreading" movements in the ground, which may require design to higher force levels than the NBC 2005 minimum tie force values. For flat sites, ground spreading due to differential horizontal movement typically will not be a major concern. However, for sloping sites, sites near river banks/free-faces, or sites that have been filled above surrounding grade, spreading within the building footprint will be a concern and needs to be accounted for in the design. This can be done by:

- General ground densification, or densifying zones to retain the horizontal displacements.
- Provide raft slab/foundation systems designed such that the lateral spreading of the soil beneath the building can occur without yielding the foundations.
- Designing and detailing well defined load paths between foundations to tie them together to resist forces due to spreading such as passive pressure on footings and pile caps, and friction forces applied to the underside of slabs and footings.
- Portions of the structure located below the grade slab, such as loading docks, sumps and elevator pits, should be detailed to either resist the loads or to "break free".

d) Pile foundations should be founded below the liquefiable layer such that they can resist the dead and live load forces with very little significant vertical displacement in the pile. Pile foundations should not yield before the structure yields during the period of structural shaking of the building, and must then be able to undergo the horizontal displacement imposed by the liquefied soil without losing their capacity to resist dead and live vertical loads. The pile may yield in flexure under these displacements, but it must behave in a ductile fashion and it must not fail in shear.

5.6.2 *Soils Strength, Resistance and Load Factors*

In estimating the potential for liquefaction and resulting movements during the earthquake, such as shear-induced vertical displacement or lateral displacement, the soil resistance used should be a "cautious estimate of the characteristic strength". This is consistent with the seismic evaluation used for structures where the real strength, and not nominal or design strength, of the seismic resisting structure is used to resist the seismic forces. Thus, geotechnical strength reduction factors less than 1.0 (or inversely safety factors greater than 1.0) need not apply for the assessment of the A2475 event.

However in structures, to guard against critical or brittle failures such as shear failure or axial column failure, elements other than the ductile parts of the seismic force resisting system must be designed for the real forces from the seismic analysis, plus some live load, and the capacity must be based on the nominal strength reduced by the material strength reduction (Φ) factors, i.e. these critical elements should never reach their yield strength.

When estimating the capacity of the soil to support the gravity loads, say for the example of spread footings on a crust above the liquefied layers where failure could lead to very large displacements, the dead load plus some live load should be applied, and the soil strengths should be based on the geotechnical resistance factors (Φ factors) times the characteristic strength. This approach is consistent with the factored strength/overstrength approach used in the NBC 2005 for structures as discussed in 5.5-15).

5.6.3 *Foundation Types*

Foundation types can range from piles to rafts to spread footings. Some structural design considerations are illustrated in Figs. 5.1, 5.2, and 5.3.

5.7 Guidelines for Structural Assessment

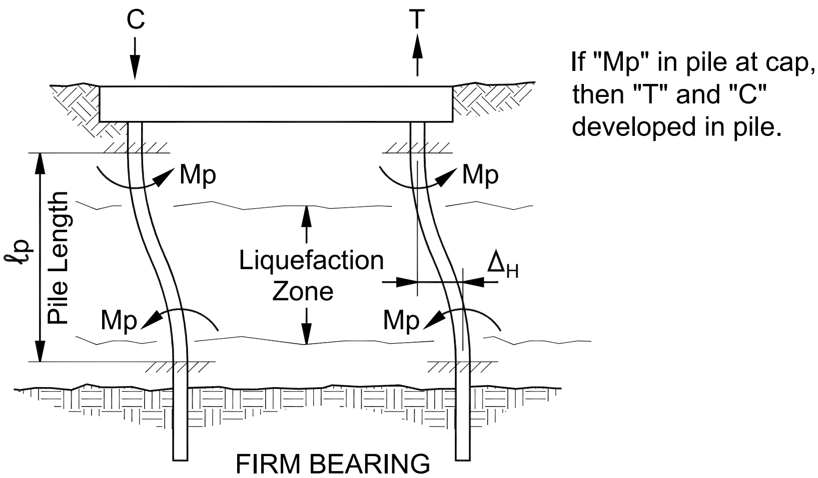
1) The maximum drift limits from ground displacement given in Table 5.2 should be met. The structure must be reviewed to determine if it can tolerate the calculated displacements.
2) The following structure types are typically more able to accommodate the drift limits given in Table 5.2 and may not need detailed checking:
    - Light wood frame
    - Long reinforced masonry walls with well-distributed bond beams and long nominally reinforced concrete walls, providing they are all well connected to the diaphragms.
3) Some of the following structure types may be able to accommodate the drift limits given in Table 5.2, but need to be checked for their deformation capacity:
    - Concrete structures of columns, beams, and slabs, and tilt up wall panels should be assessed using the CSA-A23.3 Standard or FEMA356. See Figs. 5.2 and 5.3.
    - Reinforced masonry walls without well distributed bond beams and tilt up wall panels should also be assessed, see CSA-S304.1.

- Steel moment frames and braced frames – assess using the CSA-S16 Standard.
- Steel "stick framing" – considered acceptable and being able to accommodate the displacements, providing the connections are considered ductile. Advice on this is given in the CSA S16 commentary section on conventional construction where R=1.5. The requirements are typically easy to satisfy with many properly executed standard details. Deep bolted beam connections may require short slotted holes at the top and bottom of the connection. Joist bottom chord extensions stabilizing beam column joints may be a concern and other methods, such as well designed full height stiffeners may be preferable.

4) If the structure cannot tolerate the imposed displacements then either the structure must be altered or the ground must be improved, as discussed in Sec. 6.0.

5.8 Communication between Structural and Geotechnical Engineers

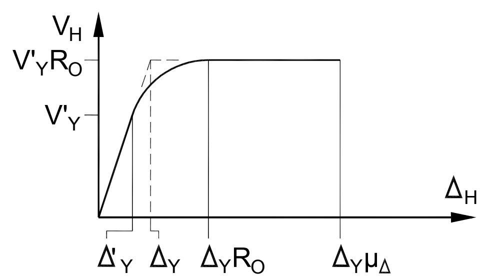
The June 1991 Task Force report on earthquake design in the Fraser Delta states quite simply that "the Structural Engineer and Geotechnical Engineer should meet to review the proposed foundation design and to discuss possible load conditions." The communication between the geotechnical engineer and the structural engineer is even more important today, and may require several iterations to resolve issues.



Ductility limits for pile assessment - A2475:

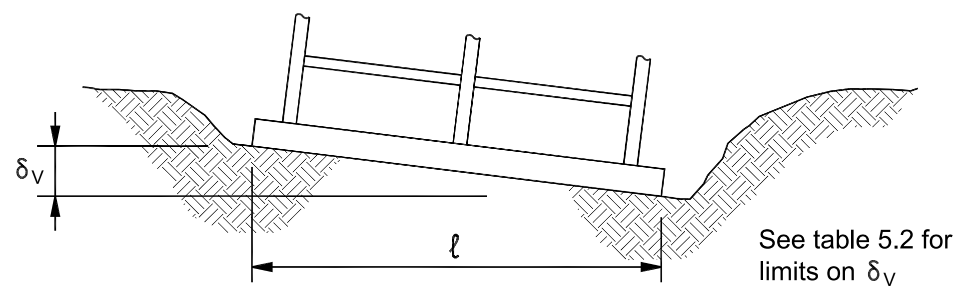
- Steel: Class 1 section: $R_O = 1.5$, $\mu_\Delta = 5$
  Class 2 section: $R_O = 1.3$, $\mu_\Delta = 1$
  Class 3 and 4 sections: must remain elastic, $s < F_Y$, $R_O = 1.0$
- Concrete: Non-prestressed: R_d and R_O as per NBC 2005, $\mu_\Delta = R_d$
  Prestressed: $R_O = 1.3$, $\mu_\Delta = 1$
- Wood: $R_d = 1.3$, $\mu_\Delta = 1$

Shear in pile(s)

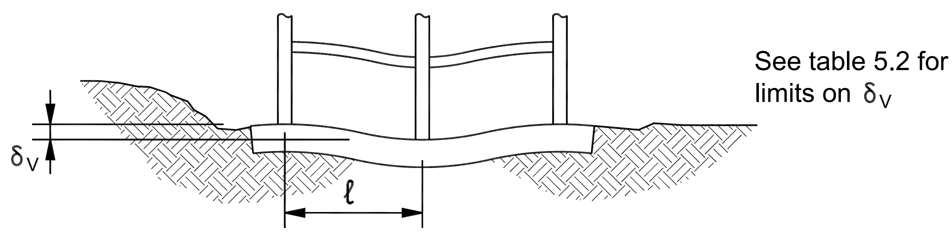


- $\Delta_H = \Delta_Y \mu_\Delta$ where Δ_H is the estimate of the horizontal displacement from liquefaction analysis. See table 5.2 for other limits on Δ_H.
- $V'y$ = Shear at factored flexural resistance.
- $V'yR_O$ = "Actual" shear at flexural resistance.
- Δ_Y = "Effective" yield displacement $R_O \Delta'_Y$, where Δ'_Y is displacement when Mp equals factored flexural resistance.

Figure 5.1 Pile Foundations - Structural Assessment



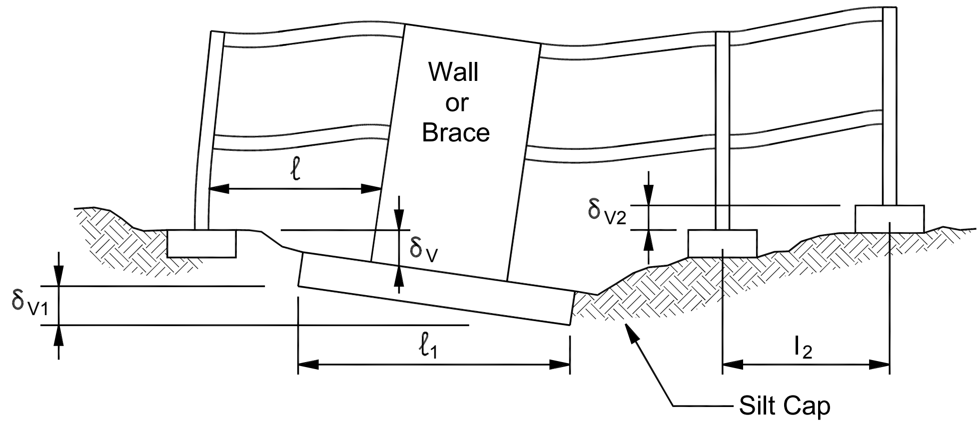
"Stiff" Raft: structure rotates as a rigid body - no "drift" in structure. Design factored resistance of raft for induced moments and shears due to lateral capacity design of structure over, or due to liquefaction displacements, whichever governs.



"Flexible" Raft: design factored resistance of raft for induced moments and shears. Check structure for induced deformations. Raft, structure and soils will interact. May require some iteration on displacement estimates.

Assess structure over using CSA A23.3-04 requirements using appropriate R_oR_d factors and, where appropriate, clause 21.12. See also FEMA 356/November 2000.

Figure 5.2 Raft Footings



- See table 5.2 for limits on δ_V.
- For cases where the footings are founded on a "silt" cap, δ_V will have components due to vertical deflection of the silt "into" the liquefied layer under the footing as well as deflection of the footing "into" the silt.
- Rotation of footings (δ_{V1} / ℓ_1 for example) can cause distortion of the structure and if it occurs at any footing, it must be accounted for. Confirm "no rotation" assumption with geotechnical engineer.
- NBC 2005 may require footings to be tied together. If differential horizontal displacements are expected, the footings must be tied and designed for this.
- Assessment for footings: Design for factored resistance to resist forces in footing.
- Assessment for structure over: Assess using CSA 23.3-04 requirements using appropriate R_dR_o values and, where appropriate, clause 21.12. See also FEMA 356 (2000).
- For braced frames use a single footing between columns under the brace.

Figure 5.3 Spread Footings

6 Ground Displacement Reduction and/or Structural Modifications

6.1 Structural Modifications

If the structure cannot accept the imposed displacements, then the following can be considered:

- Improve the structural "ductile" detailing.
- Change structural types.
- To prevent "punch through" footings founded on a "cap" over the liquefiable layer, consider increasing the footing perimeter, or adding columns to reduce the column loads.
- Place the structure on piles, providing the piles can accept the horizontal displacements.
- Place the structure on a stiff raft.

If none of the above solves the problem, then carry out soil remediation such that the ground displacements are reduced until they can be tolerated by the structure and piles.

6.2 Soil remediation

6.2.1 *General comments*

Martin et al. (1999) was used extensively in developing this section. "*Seismic Design Guidelines for Port Structures, 2001"* is also a useful reference for ground improvement.

There are many procedures for mitigating liquefaction and its consequences. Mitigation methods can be classified according to their function as listed below:

<u>Densification</u>: Densification causes the soil to go into a tighter packing and increases it's cyclic resistance. Example methods include: vibro-compaction, vibro-replacement, dynamic compaction, explosive compaction, compaction piles, gravel compaction piles, preloading, rapid impact compaction, compaction grouting, and various top-vibrated probes.

<u>Drainage</u>: Drainage dissipates excess pore water pressure thus helps maintain the effective strength of the soil. Example methods include: vertical seismic drains, wick drains, sand compaction piles and possibly stone columns in some soils.

<u>Dewatering</u>: Dewatering removes the pore water and thus volumetric compaction can occur without loss of soil strength. Dewatering is conducted by permanent pumping or drainage systems. Cut-off walls may be part of the system to minimize the volume of water pumped and effects on adjacent structures.

Mixing and solidification: These methods increase cyclic resistance by cementing the soil grains together. Examples include deep and shallow soil mixing and permeation grouting.

Reinforcement and containment: Reinforcement and containment reduces ground deformation by reinforcing or containing the liquefied soil layers with stiff inclusion or wall elements. Reinforcement and containment may also reduce the cyclic loading on the soil and thus reduce it's liquefaction susceptibility. Example methods include piles (acting as dowels), jet grout columns, slurry walls, and sheet pile cells or walls. Blocks of densified ground may also be used to contain potentially liquefiable soil.

Replacement: Liquefaction can be mitigated by removing the liquefiable soil and replacing it with non-liquefiable soil. This can be done at shallow depths using normal construction excavation, backfill and compaction equipment. Procedures such as jet grouting or vibro-replacement also cause a partial replacement to occur as part of the process.

In many mitigation designs several methods may be combined and/or one method may have more than one function. Mitigation should provide suitable protection against potential lateral spread or flow failures, bearing failure, and settlements. The choice of mitigation methods will depend on the extent of liquefaction, related consequences, and cost.

Utilities and lifeline services from outside the building structure may still be damaged even though the building is remediated against liquefaction. The use of flexible / ductile service connections at locations where differential displacements are expected (ie. between the densified and adjacent non-densified ground) will increase the robustness of the system and reduce the requirement for local repairs following the earthquake. If post-disaster performance is desired then self-contained systems such as standby generators and storage tanks should be considered. Automatic shut-off valves are also available and may reduce the hazard in some cases.

6.2.2 *Soil Improvement Methods*

Some of the potential soil improvement methods are discussed in more detail below.

6.2.2.1 Vibro-compaction / vibro-replacement

Vibrocompaction and vibro-replacement are generally performed with electric or hydraulic powered vibrators that are jetted into the ground with water or water/air mixture. In vibro-compaction the natural sand self-feeds into the void created by the vibro-flot, whereas in vibro-replacement stone is used as backfill. Top-feed and bottom-feed vibro-replacement methods are used. In the top-feed method the stone is introduced into the jetted hole from the ground surface whereas in bottom feed the stone or gravel is transmitted down the hole via a pipe and introduced at the bottom of the hole. Often bottom feed is carried out using air jetting. The latter method allows the use of finer backfill material and potentially less mixing of the native soil and stone occurs. Vibro-replacement is generally effective in soils containing less than 15 to 20% fines (Martin et al, 1999). Luehring et al., 1998 showed that non-plastic sandy silts can be densified by a combination of vibro-replacement and vertical wick drains.

The equipment should be capable of delivering sufficient centrifugal force to cause the required densification. Stone backfill materials should generally be clean and hard. Crushed stone should be used when the stone backfill is to provide reinforcement for vertical or horizontal deformations. The depth of the vibrator, stone usage, and amperage or power developed are often monitored during the work. Achievable improvement depths are typically in the 25 to 35 m range although greater depths have been achieved with special equipment. Vibro-replacement is the most widely used liquefaction countermeasure in Vancouver and elsewhere in North America (Hayden and Baez 1994). Design information and equipment characteristics can be found in many publications including Barksdale and Bachus (1983), Mitchell and Huber (1985), Dobson (1987), Baez (1995 and 1997).

6.2.2.2 Dynamic Compaction (DC)

Dynamic compaction uses impact on the ground surface to densify subsurface soils. Weights typically ranging from 10 to 30 tons are repeatedly lifted by a specially modified crane and dropped from about 15 to 40 m heights. The amount of compaction and densification depth is a function of the weight, drop height, number of drops per point and the spacing of the grid. Empirical relationships are available to design deep dynamic compaction programs (Lukas 1986). Typically treatment depths of up to 11 m are achievable in granular soils. A granular pad is often placed if surficial saturated cohesive soils are present or if the groundwater table is within 1 to 1.5 m of the surface. The major limitations of the method are limitations on the depth of compaction that can be achieved and, if used in an urban environment, vibrations, flying matter, noise, and perception of damage. For these latter reasons, work often requires 30 to 60 m or more clearance from adjacent occupied buildings or other sensitive structures.

Energy delivered to the ground, sequence spacing and timing of drops, as well as ground response in the form of crater depth and heave of the surrounding ground are important quality control parameters. The location of the water table and presence of surface "hard pans" can greatly affect the quality and outcome of the densification process. Pore water pressure within recently treated areas should be allowed to dissipate before secondary treatments are implemented.

6.2.2.3 Rapid Impact Compaction

Rapid impact compaction (RIC) is analogous to dynamic compaction in that the ground surface is impacted with a weight. The difference is that the weight and drop height are smaller and more frequent. In RIC a weight of approximately 7.5 tonne is dropped about 1.2 m to impact a 1.5 m diameter footing. A pile-driving-like hydraulic hammer is used to lift and drop the weight at 40 to 60 blows per minute. Like dynamic compaction the penetration depths are limited, however improvement depths greater than 6 m have been achieved in granular soils. The drop height, number of blows, penetration per blow, and total energy per point are monitored by a data acquisition system. Vibrations from the work will be similar to those from pile driving and must be considered if working near structures, especially if they are occupied.

6.2.2.4 Compaction Grouting

Compaction grouting involves pumping low slump, mortar-type grout under pressure to densify loose soils by compaction. Effective treatment requires that the grout push the soil aside rather than fracture or permeate it. Compaction grouting pipes are typically drilled-in or driven steel pipes of 50 mm internal diameter or greater. Pressures ranging from 700 to 2100 kPa (100 to 300 psi) are used to inject a stiff, 75 mm or less slump, cement grout. In granular soil susceptible to liquefaction refusal pressures of 2800 to 3500 kPa (400 to 500 psi) are common (Martin et al. 1999). Grout pipes are typically installed in a grid pattern of 1.5 to 3 m spacing. Often primary spacing patterns with secondary or tertiary intermediate patterns are used. Spacing and sequence of the grout points affects the quality of densification and ground movements achieved.

Grouting volumes in granular soils typically range from 3 to 12 percent of the treated soil. However, volumes up to 20 percent have been reported for extremely loose sands or silty soils. The procedure is not effective when vertical confinement is less than 2.4 to 3 m of overburden (Martin et al. 1999). Information on this technique can be found in Graf (1992), Baez and Henry (1993), Boulanger and Hayden (1995), and Warner (2004).

Slump, consistency of the mix, grout volumes, injection pressures, pore water pressures, and ground movement at the surface, next to sensitive structures, and at depth are often monitored during the work. Grout is typically injected in stages from the bottom up. At each stage a stopping criteria of grout volume, pressure, or heave is followed before proceeding with the next stage. Grout casing should be at least 50 mm internal diameter to avoid high back pressures before sufficient grout is injected. Over-injection of grout in a primary phase may result in ground heave and diminish densification effectiveness (Martin et al, 1999).

6.2.2.5 Strengthening, Solidification and Mixing methods

Strengthening, solidification and/or mixing techniques introduce grout materials into the soil voids either through permeation, mechnanical mixing, or jetting. These techniques are known as permeation grouting, soil mixing, and jet grouting.

In permeation grouting, low viscosity liquid grout is injected into the pore spaces of granular soils. Typically sodium silicate or microfine cements are used. The intent of the process is to turn the liquefiable soil into a hardened mass. It is generally more effective in soils with less than 12 to 15 percent fines (Martin et al. 1999). The method has been used on a bridge pier in Santa Cruz, California (Mitchell and Wentz, 1991), and on a tunnel in downtown San Francisco. This technique is described in detail by Baker (1982), Moseley (1993), and Warner (2004).

6.2.2.6 Jet Grouting

Jet grouting forms cylindrical or panel shapes of hardened soil-cement with strengths up to 17 MPa (2,500 psi). High 48 MPa (7,000 psi) water pressure at the nozzle is used to cut soils, mix in place cement slurry and lift spoils to the surface (Martin et al. 1999). Control of the drill rotation and pull rates allows treatment of variable soils. The procedure is described by Moseley (1993). The procedure can be used in confined spaces such as inside existing buildings. Care must be taken when using this procedure around existing structures as blockage of the return fluid during grouting can result in sustained high pressures that may cause ground heave and adversely impact existing facilities.

6.2.2.7 Soil-mixing

Soil-mixing involves mixing cementitious materials with the native soil using a hollow-stem auger and paddle arrangement. Cement, fly ash, quicklime, and other additives are used in the grout. Augers of 1 m or more in diameter can mix to depths of 30 m or more (Martin et al. 1999). The hollow stems are used as conduits to inject grout into the soil at the tip of the auger as it is advanced into the soil. Typically the liquefiable soil is contained within soil-mixed walls created by overlapping the augered columns. The procedure reduces shear strain within the soil to reduce the potential for liquefaction and confines soils that do liquefy to limit displacement. The walls also add shear strength to the overall soil-wall system. Column shear strengths of 175 kPa (25 psi) or more can be achieved, even in silty soils (Martin et al., 1999). The method has been used at Jackson Lake dam in Wyoming (Ryan and Jasperse, 1989), at Crofton paper mill on Vancouver Island (Broomhead and Jasperse, 1992) and in Japan (Schaefer, 1997).

Columns are usually tested using wet sampling, coring, CPT, pressuremeter, or seismic methods. Some variation in uniformity and strength should be expected.

6.2.2.8 Compaction Piles

The driving of piles on close centres compacts the soil by pressure and vibration. Pile shafts also have a reinforcing effect by acting as dowels between the soil layers. Timber and sand or gravel compaction piles are commonly used. If the timber piles are permanently below the water table the timber need not be treated. Piles are typically placed on 1.2 to 2 m centres and splices are sometimes used to increase depth. Sand and gravel compaction piles are often made by vibrating in a pipe with an expendable bottom plate. Upon extraction sand or gravel is introduced into the displaced void. Sand and gravel compaction piles can also be constructed using expanded base pile procedures. The spacing of sand and gravel compaction piles depends on pile diameter but is typically in the 1.2 to 2 m range. Densification performance is often tested by carrying out penetration tests between the piles.

6.2.2.9 Explosive Compaction

Explosive compaction (EC) is carried out by setting off explosive charges in the ground. The principal advantages of EC relative to other vibratory densification techniques is that it can be carried out to great depth provided the soils are largely saturated. The method requires only drilling equipment and a supply of explosives to implement. However, careful engineering design of the EC process is required to assure reasonably uniform densification (through selection of the sequencing of the blast hole patterns) and minimize offsite vibration effects (if required). The method is particularly cost effective where relatively large volumes of soil are required to be densified at depths in excess of 6 m. The density of loose deposits can typically be increased to relative densities in the range of 70 to 80% ($(N1)_{60}$ of 20 to 25 and cone penetration resistances (Q_{c1}) of 100 to 130 bars). Once an area of ground has been shot and pore pressures have largely dissipated, repeated applications ("passes") of shaking caused by controlled blast sequences causes additional settlement depending on soil density and stiffness. The degree of densification obtainable will also depend on the fines content of the sand as is the case for other methods of densification. The range of particle size for which blasting is practical is the same as for vibro-compaction.

It has been observed that where blasting is used, there is a considerable time effect on the values of penetration resistance. For the above reasons, initial evaluation of the effectiveness of the EC process is based on direct measurement of soil volume change using *insitu* settlement gauges.

6.2.2.10 Drainage Techniques

Drainage can be beneficial in both limiting the triggering of liquefaction and reducing the related deformations. Seismic drains were proposed by the late Prof. Seed as a means of mitigation in the 1970's (Seed & Booker 1977). Design procedures and the public domain program (FEQDRAIN (Pestana et al. 1997) are available. The intent of Seed's procedures was to prevent triggering of liquefaction.

Recent research has shown that pore water migration and redistribution during earthquakes is important and trapping of water under low permeability layers can lead to very weak interlayers or even a water film (Kokusho 2003; Byrne et al. 2006). It is believed that this is the reason for the low residual strengths observed in liquefaction case histories. For sites with low permeability barriers over liquefiable sand, drains can be used to mitigate this effect and reduce ground deformations and potential for flow slide failure. Drains should also be considered for reducing the pore pressure buildup within densified ground that is surrounded by liquefied soil, especially if, due to property line or access restrictions etc. building foundations are required to be close to the edge of the densified block.

The performance of the drains is dependent on the following parameters:

- soil permeability,
- drain spacing,
- vertical flow capacity of the drain,
- soil density (capacity for volumetric compression),
- filter compatibility between the drain and native soil,
- elevation of point of discharge relative to water table, and
- rate of loading provided by the earthquake (ie. if the earthquake liquefies the soil in one pulse the drains will not have time to respond and temporary liquefaction may still occur, whereas if many pulses are required to liquefy the soil then the drains may work well as the excess pore water is dissipated as it is generated).

The use of drains without soil densification may not necessarily provide adequate pore pressure relief during the period of strong earthquake shaking (unless the drains are on very close centres). However, they can have beneficial effect in preventing flow slides and reducing lateral deformations.

Drains can be constructed in several ways:

- pre-fabricated perforated pipe within a filter cloth sock - these can be installed in a drill hole or vibrated into the ground using a mandrill.
- slotted pipe with surrounding filter sand or fine gravel installed in drill holes
- traditional water wells with screen and filter
- gravel or sand compaction piles
- vibro-replacement columns constructed using filter sand and bottom-feed methods
- prefabricated wick drains

Shake table tests (Sasaki and Taniguchi, 1982) indicate that gravel drains can accelerate the dissipation of excess pore water pressures, thereby limiting the loss of shear strength and reducing the uplift pressures acting on buried structures. Following the 1993 Kushiro-Oki, Japan, earthquake, Iai et al. (1994a, 1994b) observed that quay walls having back fill treated by the gravel drain pile and sand compaction pile techniques suffered no damage, while quay walls having untreated backfill were severely damaged due to liquefaction. Seismic drains were tested at Massey Tunnel by liquefying the soil with blasting. The tests showed that both prefabricated drains with filter cloth sock and slotted pipe with surrounding filter sand drains performed well. The tests also showed that large settlements may still occur in the vicinity of the drains, and illustrated the importance of vertical flow capacity of the drain (large volumes of water have to flow over a short period of time). The tests showed that liquefaction may still be triggered in the loose sand soils located between the drains if they are subjected to high intensity short duration shocks (ie. if pore pressures build up in the soil significantly faster than it is dissipated by the drains).

An alternative drainage method is to lower the ground water level by permanent dewatering. This reduces the degree of saturation, thereby preventing the development of excess pore water pressure which would lead to liquefaction. Permanent pumping and perimeter cut-off walls may be necessary. The effects of dewatering on adjacent structures can also be a consideration. This method of mitigation is not common.

When designing drainage, consideration should be given to their effect on the local groundwater regime, especially if water-supply aquifers or pollutants are in the vicinity.

6.2.2.11 Preloading

In this procedure the soil is precompressed by placing a temporary load prior to placement of the actual foundation loads. This is usually done by placing sand fill on and slightly beyond the building footprint. The fill is left in place for a period varying from a few weeks to many months. Just prior to building construction the sand fill is removed. Recent research at Univeristy of British Columbia (Sanin and Wijewickreme 2006) has shown that the benefits (increased liquefaction resistance) of this in silty soils can be significant. In sandy soils the benefit is not as conclusive and the effects of preloading are generally not considered in liquefaction triggering and ground densification design.

6.2.3 *Lateral extent of Mitigation*

Light buildings may be able to bear on a crust over liquefied ground and treatment may not be required. Sometimes treatment is only required around portions of the edge of the site (such as at a river edge) in order to contain the soil. For heavy buildings treatment is typically required below the footprint of the building and some distance beyond. The extent of ground treatment under and adjacent to a building can be determined by analyses. Lateral spreading potential, bearing capacity and seepage conditions during and after the event should be considered (Port and Harbor Research Institute, 1997). The propagation of excess pore pressures from adjacent liquefied ground to the improved ground needs to be considered in the analyses (Iai et al. 1988, Mitchell et al. 1998). Analyses have shown that the extent of treatment beyond the edge of the building foundations can be reduced by placing seismic drains in the vicinity of the outer foundations.

Martin et al. (1999) suggest that the densified zone should extend 1/3 to 2/3 the thickness of the liquefiable layer beyond the edge of the building (for level ground conditions). Mitchell et al. 1998) suggests that (i) the densification should extend to the depth of liquefaction, (ii) that densification extend a distance beyond the building equal to the depth of densification, and (iii) that the outer portion of densification extending up from a 30 to 35 degree line from the toe of the densification should be assumed to behave as liquefied ground in design. In Vancouver densification zones for highrise buildings in the Fraser Delta (designed for the 475 year return period earthquake) typically have not extended as far beyond the building as recommended by Mitchell et al. 1998. Densification zones used to prevent lateral spreading adjacent to the Fraser River edges (designed for the 475 year return period earthquake) typically have had widths approximately equal to the thickness of the liquefiable layers.

If analyses are not carried out to confirm otherwise, it is recommended that the width of densification should (i) extend vertically the full depth of potential liquefaction, and (ii) extend laterally under the full footprint of the building and a distance equal to the thickness of the liquefiable layers (including any non-liquefiable layers between the liquefiable layers) beyond the edge of the building footprint. If lateral spreading deformations are expected in the soil around the building, then the width of densification should be sufficient that the passive capacity of the densified block can resist the forces from the surrounding moving soil mass, or the building should be designed such it can move with the soil mass without collapse.

6.2.4 *Quality Assurance*

Specialized equipment and experienced personnel are generally required for soil improvement work. The use of specialty construction companies with previous experience in similar soils and job conditions is recommended. Quality assurance requirements will vary depending on the technique being used. In general, the engineer of record or his/her representatives carries out on-site inspection and supervises the testing program. Testing is usually carried out in the middle of a grid pattern formed by the densification locations, (although this may be slightly conservative in some situations). A minimum of 48 to 72 hours after soil improvement, and sometimes much longer, should be allowed for prior to testing to permit excess pore pressures to dissipate.

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