# Evaluation of In-Situ State of Fraser River Sand

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## Abstract

Liquefaction flow slides are recurrent phenomena in the Fraser River Delta near Sand Heads. The reasons for these failures are unexplained and remained speculative. An investigation of a liquefaction flow slide that occurred in 1985 in the Fraser River near Sand Heads has been undertaken using steady state concepts. For liquefaction flow slides to occur in sand deposits, the sand must strain softening in undrained shear and the in-situ shear stresses must be greater than the available undrained shear strength. Laboratory tests have been performed on reconstituted Fraser River sand samples to establish steady state parameters. Moist tamping methods and water pluviation techniques were used for preparing soil samples in the laboratory. Shear wave velocity measurements were used to evaluate the in-situ state of the young, uncemented sand deposits. The in-situ state of the young sand deposits near Sand Heads were found to be on the loose side of steady state consistent with the field observation of instability.

Key words: In-situ state, liquefaction, shear wave velocity, steady state.

## Introduction

Instability of coastal and other underwater slopes is a natural phenomenon and its impact is felt when human development activities extend into these subaqueous environments. Safety of coastal structures such as jetties, breakwaters, lighthouses, pipelines and offshore platforms is a concern due to possible slope instability. A major source of instability in cohesionless sediments is due to liquefaction flow slides. Flow slides in loose materials can be triggered by dynamic effects, such as earthquakes, blasting and vibrations due to such activities as pile driving and by static effects such as tidal changes and other variations in water level. Terzaghi (1956) referred to the phenomenon of sudden or static liquefaction of loose sands due to minor triggering mechanisms as spontaneous liquefaction.

Castro (1969) defined liquefaction as the behavior of saturated, loose sand where increasing pore pressures due to undrained shear decrease the effective stresses resulting in a reduction in the shear resistance (strain softening) to a constant value, called steady state. The void ratio at steady state is the critical void ratio as defined by Casagrande (1936). Steady state divides two states of sand as contractant or dilatant at large strains. Robertson (1994) defined the term flow liquefaction to describe the process of strain softening in undrained shear resulting in flow studies.

Several cases of liquefaction induced flow slides in subaqueous environments have been presented by Terzaghi (1956), Morgenstern (1967), Seed (1968), Schwarz (1982), Kramer (1988), Chaney and Fang (1991), Silvis and de Groot (1995). Table 1 presents a summary of published cases of earthquake induced liquefaction flow slides in the marine environment. Table 2 summarizes published cases of spontaneous liquefaction (flow liquefaction) failures of sediments in river deltas and in coastal deposits. The statically triggered liquefaction flow slides in coastal sediments involve remarkably similar materials and similar conditions. Some slopes have very low gradients which may reflect post event gradients. Conventional slope stability analyses of the slopes as presented by Morgenstern (1967), result in high factors of safety and the slopes where considered stable. A detailed description of the submarine slides in the Netherlands was given by Silvis and de Groot (1995).

Silvis and de Groot (1995) note that three conditions must be met to trigger a flow slide in an underwater sand slope:

1. the soil must be susceptible to (flow) liquefaction (i.e. very loose),
2. the slope must be relatively steep and relatively high; and
3. an initiation mechanism must be present

Silvis and de Groot (1995) note that sand sediments are susceptible to flow liquefaction and flow slides if the sediments are very young (Holocene age) and rapidly deposited channel fills. They comment that the initiation mechanism can be very small and should always be assumed to be present. They suggest that the initiation mechanism could be caused by; "a sudden local change in water pressure due to waves from passing ships or wind waves, an increase of outflowing ground water during an extreme low tide, a quick changing soil pressure due to local shear failure or due to dredging activities, vibration caused by pile driving, and so on".

Five known major liquefaction flow slides have occurred near Sand Heads in the Fraser River Delta on the west coast of Canada between 1970-1985 (McKenna et al., 1992). This paper describes the evaluation of the in-situ state of the sand deposits near Sand Heads in the Fraser River Delta. An analysis evaluating the contributions of different environmental processes for triggering the 1985 failure has been undertaken and details are given in a companion paper (Chillarige et al. 1997).

For a flow slide to be initiated in sand deposits, one of the requirements is that the sand must be strain softening in undrained shear. Hence, an investigation of cohesionless soils for potential flow liquefaction should quantify the in-situ state of the soil deposit. In-situ void ratio and the response of the soil to various types of loading can be estimated in the laboratory using undisturbed samples. However, it is difficult and expensive to obtain undisturbed sand samples in a subaqueous environment. Hence, laboratory tests are often carried out on reconstituted samples. However, it is important that the reconstituted samples have similar density, stress levels and fabric as the soil in-situ.

Recent studies have shown that in-situ and laboratory shear wave velocity measurements together with the laboratory derived steady state characteristics of sand can be used to estimate the in-situ state of unaged and uncemented sand deposits (Robertson et al., 1995). This paper presents the results of laboratory tests on reconstituted Fraser River sand samples, the evaluation of the in-situ state of the sand deposits near Sand Heads on the Fraser River Delta using in-situ shear wave velocities and the steady state characteristics of the sand.

**Testing Program**

Field investigations were carried out in 1993 and 1994, within very recent sediments deposited at the mouth of the Fraser River near Sand Heads and are described in a companion paper (Christian et al., 1997). Seismic cone penetration tests (SCPT) yielded profiles of shear wave velocity as well as penetration resistance, sleeve friction and penetration pore pressures. Disturbed soil samples were also collected for index testing and to make reconstituted samples for shear tests. Downhole vapour sampling was also carried out to provided an estimate of the free-gas content (Christian et al., 1997).

Triaxial tests have been conducted on reconstituted Fraser River sand sampled from a borehole located near Sand Heads. The borehole was wash-drilled down to a depth of about 18 m below the seabed and sampling was carried out using a hydraulically pushed wireline Shelby tube sampler. The soil stratigraphy encountered by the boring consists of clean fine sand interbedded with organic silt. The average particle size of the sand, $D_{50}$, is 0.25 mm and the uniformity coefficient is 1.7. The average mineral composition of the sand is approximately 40% quartz, 11% feldspar, 45% unaltered (unweathered) rock fragments and 4% other minerals. Maximum and minimum void ratios in accordance with ASTM D2049 are 1.00 and 0.6, respectively. The specific gravity was determined to be 2.75.

Samples of Fraser River sand were prepared by moist tamping methods and by water pluviation techniques. The triaxial specimens were approximately 63 mm in diameter and 127 mm in height. Careful measurements of sample dimensions were performed on each sample before testing. Moist tamping yields a very loose structure for reconstituted soil samples (Ishihara, 1993). Sample uniformity was maintained in the moist tamped specimens using careful preparation techniques (Sasitharan et al., 1993). In the sample preparation by water pluviation techniques, some segregation was observed. The upward turbulence during water pluviation appears to inhibit the formation of a very loose structure by maintaining fine particles in suspension and resulting in slightly dense samples. The water pluviation technique is considered by some to simulate the deposition of sand through water as found in many natural environments and mechanically placed hydraulic fills (Kuerbis and Vaid, 1988). But, in this study, the fine particles formed thin lenses in the water pluviated sand specimens resulting in somewhat non uniform samples.

Most of the published subaqueous liquefaction flow slides occurred on the upper slopes in fjords and river deltas. An important feature of the cohesionless soils found in flow slides is a metastable very loose structure. The soil structure appears to collapse on slight provocation. When rivers deposit sediments in estuaries, coarse grained soils are deposited close to the mouth of the river and very fine grained soils are transported further offshore by the river plume. Apparent cohesion between silt and sand particles can maintain higher void ratios during deposition. Andresen and Bjerrum (1967) speculated that the presence of a small quantity of fine particles caused the uniform fine sands to settle out at higher porosities. Removal of either the finest or the coarsest constituents of the sand eliminates the capacity of the sand to form an abnormally porous aggregate (Terzaghi and Peck, 1948). Observations from the Norwegian flow slides indicate that the sands with a small amount of fines or organic matter posses the highest porosities (i.e. the loosest state). From the observed liquefaction failures due to earthquake loading in silty sands, Ishihara (1993) states that the potential for flow failure is considered to be much higher for a silty sand than for a clean sand. The liquefaction flow slides in the Fraser River Delta occurred in layered fine silty sands of very low density. A metastable loose soil structure may be a characteristic of the young sediments at the mouth of the Fraser River.

**Steady State Parameters of Fraser River Sand**

A modified Wykeham Farrance strain controlled loading machine, as described by Sasitharan (1994), was used in the testing program. Isotropically consolidated monotonic undrained and drained triaxial compression tests were performed on loose reconstituted samples. The samples were sheared at a constant strain rate of 0.15 mm/minute after consolidation to a specific confining stress.

Figure 1 shows the measured stress-strain curves for the undrained tests and their corresponding pore pressure variations in the moist tamped samples. It can be seen that the samples reached a peak deviatoric stress within 0.5% axial strain and the change in the pore pressure shows a continued increase to about 4 to 6% axial strain before reaching a constant value. The tests (UDFR3, UDFR5, UDFR10) exhibited a strain softening response to their ultimate steady states of low stress values at large strains. The stress-strain curves for the drained tests (DFR1, DFR2) and their corresponding volume changes are presented in Figure 2. When the deviatoric stress approached the ultimate steady state, the corresponding volume changes were negligible. The samples showed a decrease in volume change throughout shear.

The steady state line was established from the ultimate steady state points obtained from all the tests on the loose reconstituted samples of the Fraser River sand as shown, in Figure 3. The steady state line is bilinear in the void ratio (e) means effective stress $\left(p' = \frac{1}{3}\left(\sigma_1' + 2\sigma_3'\right)\right.$ with a break occurring at a stress level of about 800 kPa. This observation is consistent with a break in the steady state line for Erksak Sand at 1000 kPa (Been et al., 1991). The change in shearing mechanism at this stress level is likely due to some grain crushing. Beyond the break, the slope of the steady state line is much steeper. The steady state line for Fraser River sand can be approximated as a straight line in the stress range 10-800 kPa. The steady state line in the p'-q plane (where $q = \sigma_1' - \sigma_3'$)is a straight line passing through the origin. From the laboratory test results, the steady state parameters are determined as;

$\Gamma$ = 1.11 (Ordinate of the steady state line at p' = 1 kPa)

$\lambda_{ln}$ = 0.029 (Slope of the steady state line in e-ln p' plane)

M = 1.4 (Angle of shearing resistance at steady state $\Phi' = 35^\circ$)

## Shear Wave Velocity Measurements

Measurements of shear wave velocity in the laboratory have been carried out using bender elements. The equipment used for the measurement of shear wave velocity is described in detail by Robertson et al. (1995). Shear wave velocities were determined after each stage of isotropic consolidation for all the tests on the reconstituted sand samples. Table 3 summarizes test data obtained during consolidation and steady state values for all the tests.

Shear wave velocity ($V_s$) for uncemented, unaged cohesionless soils is mainly a function of both void ratio and mean normal effective consolidation stress ($p_c'$) for isotropic consolidation state. Hence, for a sand of constant void ratio, $V_s$ will increase with increasing stress. Robertson et al. (1992) suggested normalization of the shear wave velocity to the effective overburden pressure as;

[1] $$V_{s1} = V_s\left(\frac{P_a}{\sigma_v'}\right)^n$$

where:

$V_{s1}$ = normalized shear wave velocity

$V_s$ = measured shear wave velocity

$P_a$ = reference stress (typically, 100 kPa)

$\sigma_v'$ = effective overburden stress (kPa)

n = stress exponent, (usually 0.25)

The in-situ shear wave velocity is controlled by the effective stresses in both the direction of wave propagation and the direction of particle motion. The shear wave velocity, $V_s$ from the seismic CPT is controlled by the vertical and horizontal stresses. Hence, Equation (1) should include the horizontal effective stresses as well as the vertical stress. However, Robertson et al., (1992) suggested that since the horizontal stress is generally unknown, the recommended normalization includes only the effective vertical stress. The error in excluding the horizontal effective stress is generally less than 10%. However, when evaluating the in-situ state of a sand, it is important to account for $K_o$.

Robertson et al. (1995) indicated that shear wave velocity for uncemented and unaged sands can be expressed as a function of effective stress and void ratio as

[2] $$V_s = (A - Be)\left(\frac{\sigma'_a}{p_a}\right)^{na}\left(\frac{\sigma'_b}{p_a}\right)^{nb}$$

where:

$\sigma'_a$ = the effective stress in the direction of wave propagation ($\sigma_V'$)

$\sigma'_b$ = the effective stress in the direction of particle motion ($\sigma'_h = K_o\sigma'_v$)

A and B = constants for a given sand, both in m/sec

e = void ratio

For anisotropic conditions, Equation 2 can be modified as

[3] $$V_s = (A - Be)\left(\frac{\sigma'_v}{p_a}\right)^{n}(K_o)^{0.125}$$

Comparison of Equations 1 and 3 results in the relation between the normalized shear wave velocity ($V_{s1}$) and void ratio (e) as;

[4] $$V_{s1} = (A - Be)\,K_o^{\,0.125}$$

Using Equation 4 and results from the linear regression of the laboratory data ($K_O = 1$) of reconstituted samples of Fraser River sand, the shear wave velocity parameters are determined as;

$$A = 295; B = 143 \text{ and } n = 0.26$$

Using Equation 3, contours of shear wave velocity are drawn on the laboratory derived steady state line of the Fraser River sand in the e- log p' plane, as shown in Figure 4. Shear wave velocities during consolidation of the sand samples in the laboratory are also presented in Figure 4. The consolidation data for moist tamped samples fall on the loose side of the steady state line. The data for the water pluviated samples lie on the dense side. Observations from other laboratory results have also shown that the water pluviation techniques for sample preparation result in relatively dense samples (Sasitharan, 1994; Cunning, 1994, Vaid and Thomas, 1995). Using Figure 4, the in-situ void ratio relative to the steady state line can be estimated, when the field measured shear wave velocities and effective stress conditions are known.

**In-situ Tests in the Fraser River Delta**

Cone penetration tests (CPT) and the seismic cone penetrations tests (SCPT) were carried out near Sand Heads in the Fraser River Delta in water depths ranging from 5 to 10 m (Christian et al., 1997). The field work was performed as part of a wider evaluation of delta front stability by the Geological Survey of Canada. The test sites were selected to be near the crest of an active submarine channel. The nominal depth of investigation was targeted at 30 m for all sites.

Figures 5 and 6 show the soil conditions from two CPT profiles (CPT1 and CPT2), in terms of CPT penetration resistance ($Q_t$), sleeve friction ($F_s$), friction ratio ($R_f$, ratio of the sleeve friction to the cone penetration resistance) expressed as percentage, and penetration pore pressures (u). These two sites are separated by a distance of about 300 m with CPT2 located about 200 m behind the headscarp of the submarine canyon. The profiles consist of clean sands interbedded with approximately normally consolidated non-plastic silts. These formations are channel fill deposits and are consistent with the geological observations. The cone penetration resistance profile of CPT2 shows that the deposits are loose sands with frequent silt layers. It can also be observed from both CPT profiles that loose deposits of sands with silt seams are underlain by more dense deposits below a depth of about 20 m of depth. The two profiles show that the deposits of sands with silt seams in the upper 20 m depths appear to be loose. These deposits are very recently deposited and are therefore, expected to be uncemented and unaged.

## Estimation of In-situ State

From Equation 4 and the location of the steady state line, Robertson et al., (1995) derived an expression to estimate the in-situ state parameter, $\psi$, from in-situ shear wave velocity measurements, as follows;

[5] $$\psi = \left(\frac{A}{B} - \Gamma\right) - \left(\frac{V_{s1}}{B\left(K_o\right)^{0.125}} - \lambda_{\ln}\left[\frac{\sigma_v'}{3}\left(1 + K_o\right)\right]\right)$$

Hence, in-situ state parameter can be estimated from in-situ shear wave velocity measurements provided the location of the steady state line is known (i.e. $\Gamma$, $\lambda_{\ln}$). The boundary between a loose and dense sand at large strains is defined by a state parameter $\Psi = 0$ (Been & Jefferies, 1985).

Seismic cone penetration tests (SCPT) were carried out in an area close to the crest of the foreslope near Sand Heads at the mouth of the Main Channel of the Fraser River. A typical in-situ shear wave velocity profile obtained from the seismic CPT is shown in Figure 7. Robertson et al. (1992) suggested a generalized profile to estimate $\psi = 0$ based on a normalized shear wave velocity ($V_{s1}$) between 140 m/sec and 160 m/sec. Fear and Robertson (1994) suggested that $V_{s1}$ = 160 m/sec can be used as the approximate dividing line for contractant and dilatant behavior at large strains for most quartz sands.

Based on the laboratory studies described in this study, the boundary for $\psi = 0$ for Fraser River sand is also shown in Figure 7. This boundary is based on an estimation of $K_o$ = 0.43, that is, ($K_o = 1\text{-}\sin \phi'$). This boundary indicates that much of the sand above a depth of 10 m has a $\psi > 0$, that is, contractant loose state. The sand below has a state close to $\psi = 0$. The generalized criteria suggested by Robertson et al. (1992) provides a slightly more conservative estimate of in-situ state, as shown on Figure 7.

Based on the in-situ measured shear wave velocity profile, the region of possible sand state is estimated and is also presented in Figure 4. The in-situ state is clearly above the steady state line and hence, indicates strain softening response in undrained shear at large strains. The in-situ state based on the measured in-situ shear wave velocity values appear to have a state looser than the states attainable using laboratory water pluviation techniques. The agreement between the field observations of flow slides and the predicted strain softening states of the sands support the suggested interpretation based on shear wave velocity.

Figure 4 presents the evaluation of the state of sediments in the upper depths (< 20 m) at the crest of the slopes at the mouth of the Main Channel of the Fraser River, near Sand Heads. The mean confining stress for the deposits is estimated as $\sigma_1'(1+2K_o)/3$, where $K_o$ is 0.43. The state appears to follow a steep line indicating the influence of $K_o$ consolidation. Also shown in Figure 4 is the isotropic consolidation states from laboratory testing on very loose samples. During isotropic consolidation, the samples have moderate consolidation compared to the in-situ states. Along the $K_o$ consolidation line, the shear stresses induce additional consolidation of the soil. In sandy slopes, rotation of principal stresses results in further densification. Hence, the state of the recently deposited sands at the mouth of the Fraser River appear to follow the steep line in Figure 4.

The void ratios of the shallow sand deposit can be compared to the ASTM derived $e_{max}$ and $e_{min}$, as shown in Figure 4. The in-situ state appears to have some higher void ratios than $e_{max}$ indicating a relative density of the sand at the mouth of the Fraser River less than 0% at shallow depths, and about 5% at a depth of about 36 m. This infers that some soils in-situ can exist at a relative density of less than 0%. ASTM maximum void ratio values should therefore, be used as a guide rather than taken to be absolute limits.

Figure 4 demonstrates that flow liquefaction could occur in very loose, freshly deposited and uncemented sands of the Fraser River. Laboratory prepared samples are very young samples. The sand deposits at the delta front of the Fraser River are also relatively young. Hence, the framework described appears to be effective in establishing the in-situ state of the young deposits of the Fraser River. Corrections may be necessary when using Figure 4 for the evaluation of in-situ state of aged sands.

## Conclusions

Laboratory triaxial compression tests and in-situ tests were carried out to characterize young, uncemeneted sand at the mouth of the Main Channel of the Fraser River. Moist tamping techniques for sample preparation resulted in very loose sand samples, whereas water pluviation techniques resulted in denser samples. The steady state characteristics of the sand combined with in-situ shear wave velocity measurements were used to estimate the in-situ state of the sand. The estimation of in-situ state of the sand deposits shows that the deposits appear to be loose of steady state and hence, potentially strain softening in undrained shear at large strains and have void ratios comparable to those obtained from moist tamping techniques. The relative density of the freshly deposited a sand at the mouth of the Fraser River appears to be less than 10% and in places less than 0%. The interpretation of a very loose state is supported by the field observations of flow slides and by the other site investigation data (Christian et al., 1997). A companion paper (Chillarige et al., 1997) describes the possible trigger mechanisms that may cause the observed subaqueous flow slides.

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| Slides | Magnitude | Deposits | Nature of soils | Slope angles | Reference |
|---|---|---|---|---|---|
| New Madrid earthquake, 1811 (Mississippi River Banks) | | River Bank | Sands | | Seed (1968) |
| Alaska (Yakutat), 1899 | | Submarine deposit | Deltaic marine sediments (silty sand and gravel) | | Seed (1968) |
| Alaska ( Valdez), 1908 | | Submarine deposit | Deltaic marine sediments (silty sand and gravel) | | Seed (1968) |
| Messina cone, 1908 | 7.5 | Submarine deposit | Sand/silt | 4 degrees | Ryan and Heezen (1965) |
| Alaska (Valdez), 1911 | 6.9 | Submarine deposit | Deltaic marine sediments (silty sand and gravel) | | Seed (1968) |
| Alaska (Valdez), 1912 | 7.25 | Submarine deposit | Deltaic marine sediments (silty sand and gravel) | | Seed (1968) |
| Chile, 1922 | 8.3 | Submarine deposit | Sand with silt | 6 degrees | Morgenstern, (1967) |
| Sagami Wan, 1923 | | Submarine deposit | Sand | | Menard(1964) Morgenstern (1967); |
| Kwanto (Tokyo), 1923 | 8.2 | Coastal hill sides | | | Seed (1968) |
| Grand Banks, 1929 | 7.2 | Submarine deposit | Fine sand and silt | 3.5 degrees | Heezen and Ewing (1952) |
| Suva, Fizi, 1953 | 6.75 | Submarine deposit | Sand | 3 degrees | Houtz and Wellman (1962) |
| Orleansville, 1954 | 6.7 | Submarine deposit | Sand | 4-20 degrees | Heezen and Ewing (1965) |
| Sanfransico, 1957 | 5.3 | Lake banks | Aeolin beach sands | | Seed (1968) |
| Chile (Puertto Montt), 1960 | 8.4 | Coastal deposits | Loose sands and silts | | Seed (1968) |

Table 1. Earthquake Induced Flow Slides in Submarine Deposits

| Slides | Magnitude | Deposits | Nature of soils | Slope angles | Reference |
|---|---|---|---|---|---|
| Alaska (Valdez), 1964 | 8 | Submarine deposit | Silty sand and gravel | 15-20 degrees | Holish and Hendron (1975) |
| Alaska (Seward), 1964 | 8.3 | Submarine deposit | Loose to medium sand, gravel | 15 -20 degrees | Holish and Hendron (1975)<br>Seed (1968) |
| Alaska (Valdez), 1964 | 8.3 | Submarine deposit | silty sands and gravel | 4-10 degrees | Morgenstern (1967)<br>Seed (1968) |
| Seattle,1965 | 6.7 | Coastal bluff | | | Seed (1968) |
| Klamath River Delta, 1980 (California) | 6.5 | Submarine deposit | Fine sand | 0.25 degrees | Field et al. 1982 |

Table 1. Earthquake Induced Flow Slides in Submarine Deposits

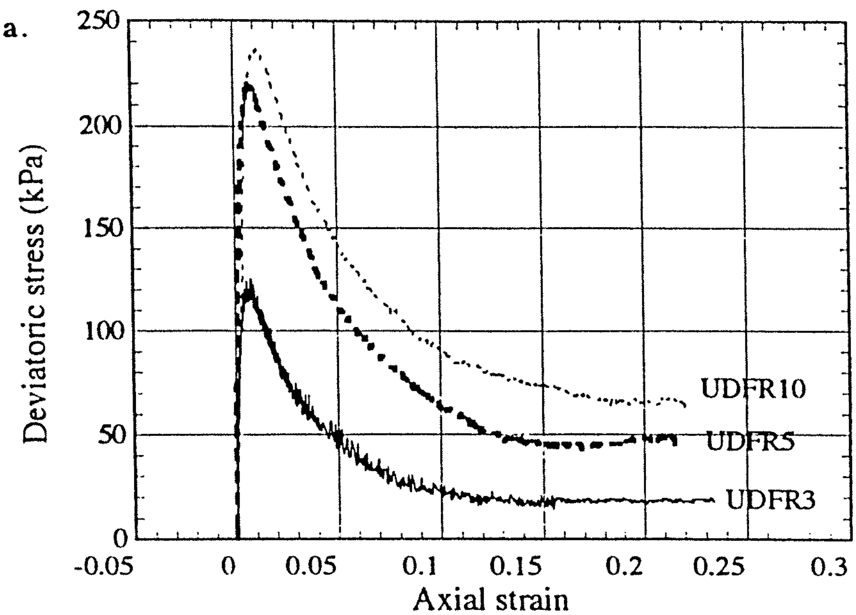
| Flow slide | Nature of soil | Failure conditions | Slope angles | Reference |
|---|---|---|---|---|
| The Netherlands | Loose fine sand | Low tides | 15 degrees | Koppejan et al., (1948) |
| Magdalena River delta,1935 | Sand and silts | Rapid sedimentation | 2 to 3 degrees | Menard(1964); Morgenstern (1967) |
| Helsinki Harbour, 1935 | Sand and silts | Rapid filling | 4-5 degrees | Andresen and Bjerrum (1967) |
| Follafjord slides, 1952 | Loose fine sand, silt | Dumping of dredged soils | 5 to 7 degrees | Terzaghi (1956); Bjerrum, (1971) |
| Orkdalsfjord, 1930 | Loose fine sand, silt | Low tides | 5-10 degrees | Terzaghi (1956)<br>Andresen and Bjerrum (1967) |
| Finnivaka Slide, 1940 | Loose fine sand, silt | Low tides | | Bjerrum (1971) |
| Hommelvika, 1942 | Loose fine sand | Low tides | | Bjerrum (1971) |
| Trondheim 1888 | Loose Fine sands, silt | Low tides | 8-15 degrees | Terzaghi(1956); Bjerrum (1971) |
| Western Norwegian fjords | Fine sand and silt | River sedimentation | 1.5-3 degrees | Aarseth,I et al. (1989) |
| Scripps Cayon, 1959/1960 | Sand | Free gas and storm waves | 25-35 degrees | Dill (1964); Morgenstern (1967) |
| Puget Sound, 1985 | Loose sands | Low tides | 16 degrees | Kraft, L.M. et al.(1985) |
| Skagway, Alaska, 1994 | Loose silty sands | Low tides of 4m | 35 degrees | Morgenstern, (1995, pers. comm) |
| Howe Sound, 1955 | Fine sands and gravel | Low tides | 27-28 degrees | Terzaghi(1956) |
| Kitimat fjord, 1975 | Loose silty sands | Low tides of 6m | 19 degrees | Morrison, K.J. (1984) |
| Nerlerk Sand berms,1983 | Loose sands | Fill placement | 10-12 degrees | Sladen et al. (1985,a) |
| Fraser River delta,1985 | Loose fine sands, silt | Low tides of 5m | 23 degrees | McKenna and Luternauer(1987) |

Table 2. Statically Induced Liquefaction Flow Slides in Submarine Deposits

| Test # | Mean pressure p' (kPa) | e during consolidation | Vs (m/sec) | p' at steady state (kPa) | q at steady state (kPa) |
|---|---|---|---|---|---|
| | 148.43 | 1.06 | 150 | | |
| UDFR3 | 164.63 | 1.047 | 151 | | |
| | 214.42 | 1.033 | 165 | | |
| | 265.4 | 1.03 | 179 | | |
| | | | | 13.24 | 17.73 |
| | 126.63 | 1.07 | 142 | | |
| | 152.73 | 1.06 | 158 | | |
| UDFr5 | 203.12 | 1.05 | 173 | | |
| | 303.29 | 1.03 | 203 | | |
| | 427.46 | 1 | 226 | | |
| | | | | 34.53 | 46.72 |
| | 194.3 | 0.8 | | | |
| UDFR7 | 244.1 | 0.79 | | | |
| | 293.87 | 0.79 | | | |
| | | | | 1024 | 1530 |
| | 202.47 | 1.012 | 183 | | |
| | 224.95 | 1.009 | 189 | | |
| UDFr10 | 273.68 | 1 | 200 | | |
| | 325.12 | 0.99 | 213 | | |
| | | | | 49.65 | 64.47 |
| | 162.2 | 1.1 | 156 | | |
| | 186.79 | 1 | 165 | | |
| DFr1 | 236.57 | 1.09 | 180 | | |
| | 289.96 | 1.08 | 196 | | |
| | | | | 535.61 | 743.2 |
| | 204.49 | 1.11 | 164 | | |
| | 230.13 | 1.11 | 180 | | |
| DFR3 | 279.47 | 1.1 | 192 | | |
| | 328.2 | 1.09 | 205 | | |
| | 427.77 | 1.07 | 228 | | |
| | | | | 806.62 | 1138.94 |
| | 307.83 | 0.89 | 201 | | |
| | 357.63 | 0.882 | 213 | | |
| DFr5 | 408.03 | 0.879 | 227 | | |
| | 457.83 | 0.875 | 232 | | |

Table 1. Shear wave velocity measurements during consolidation and stresses at steady state

a.




b.

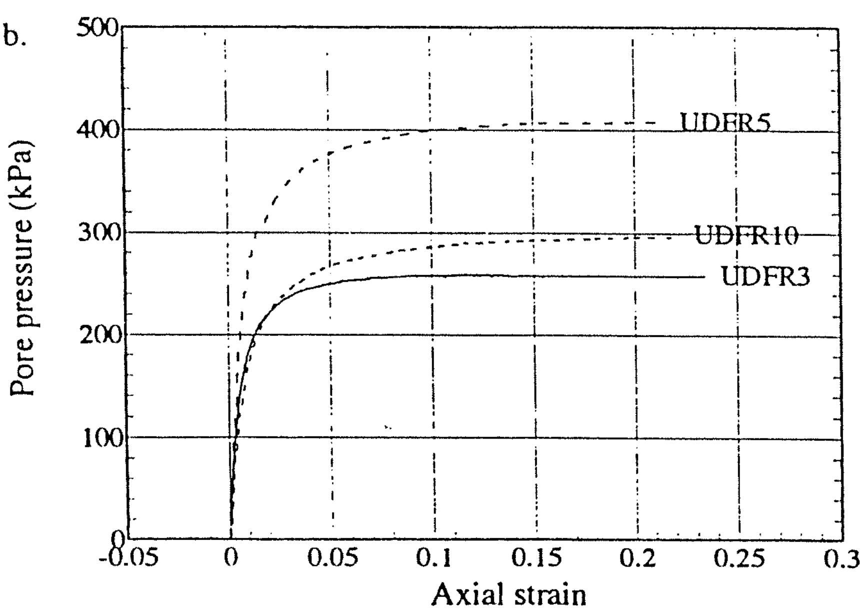
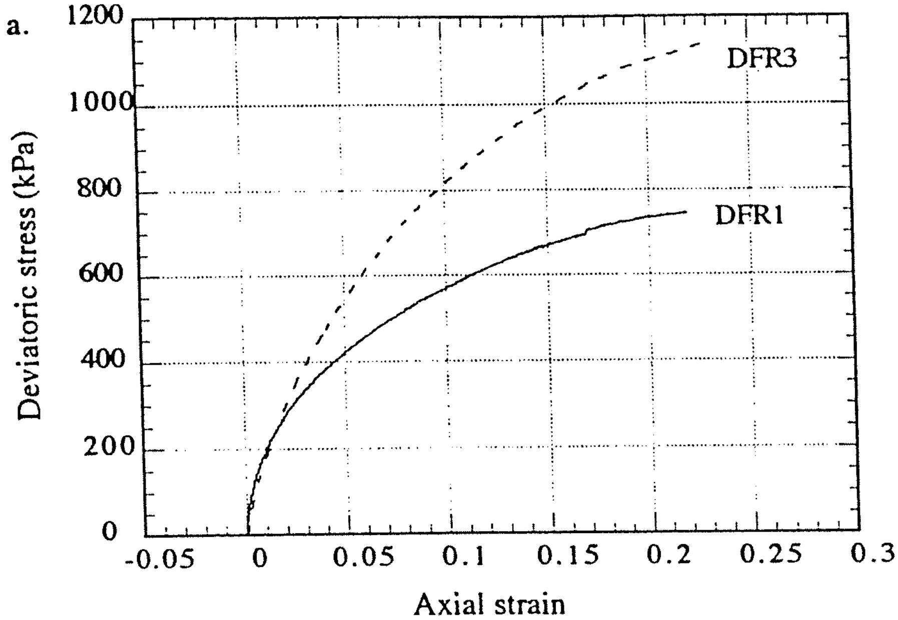



Figure - Results of consolidated undrained triaxial tests
a). Stress -strain curves b). Pore pressure variation



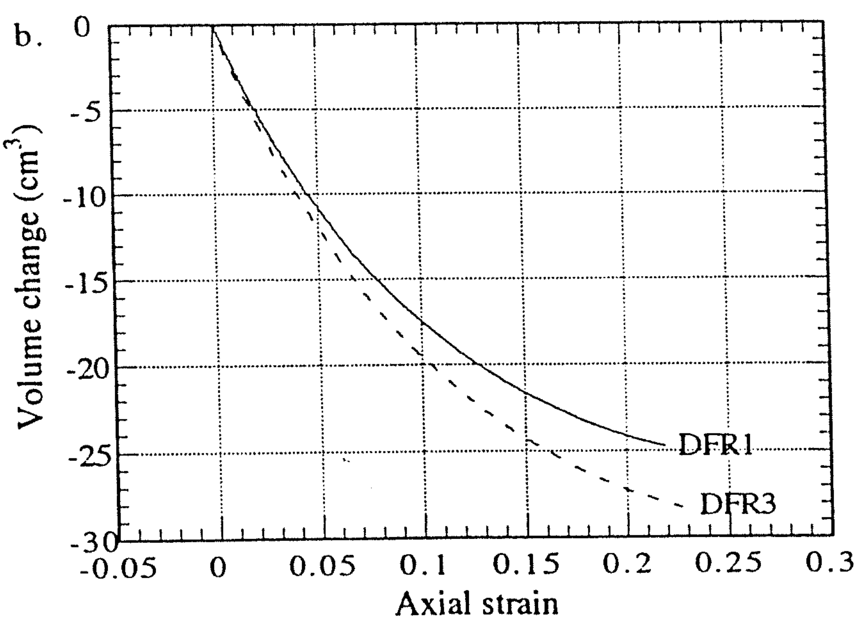
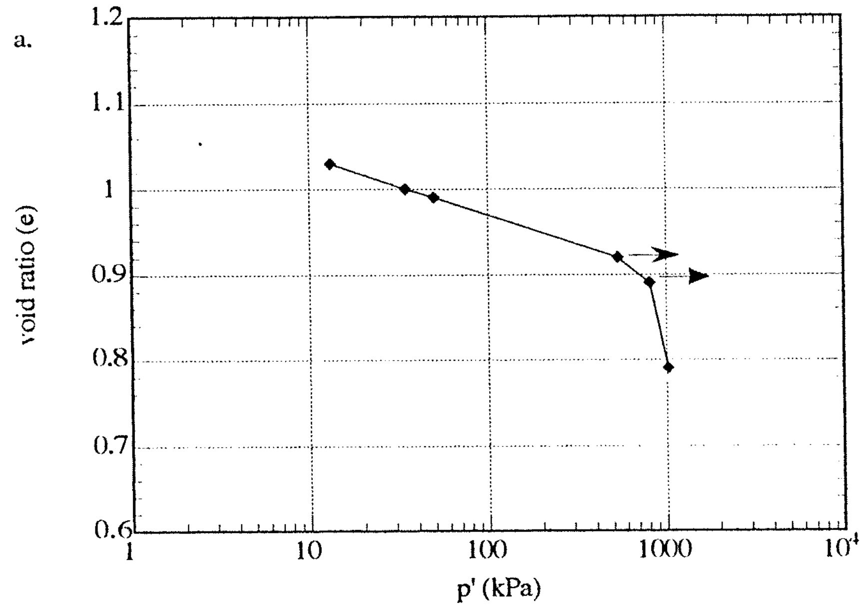




Figure . Results of consolidated drained triaxial tests
a). Stress - strain curves b). Volume change

a.




b.

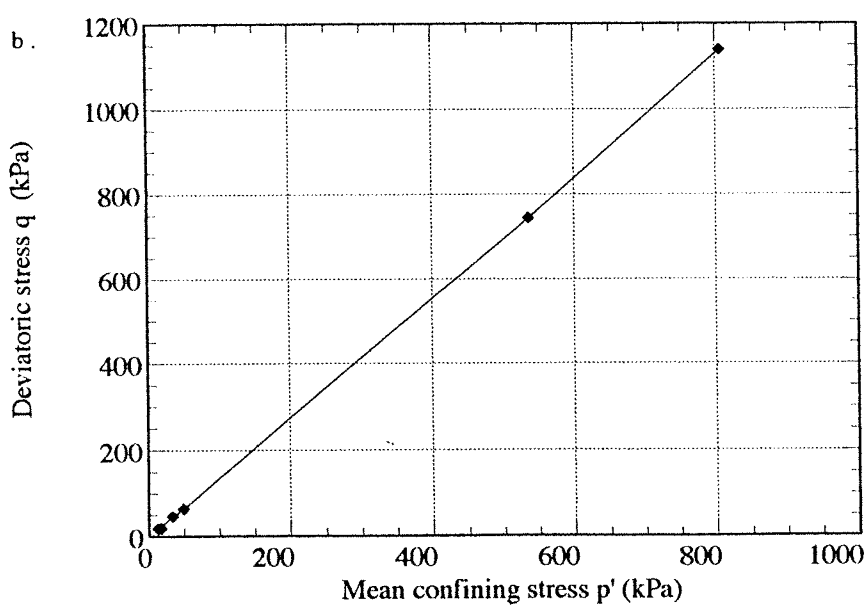



Figure Steady state line - Fraser River Delta sand

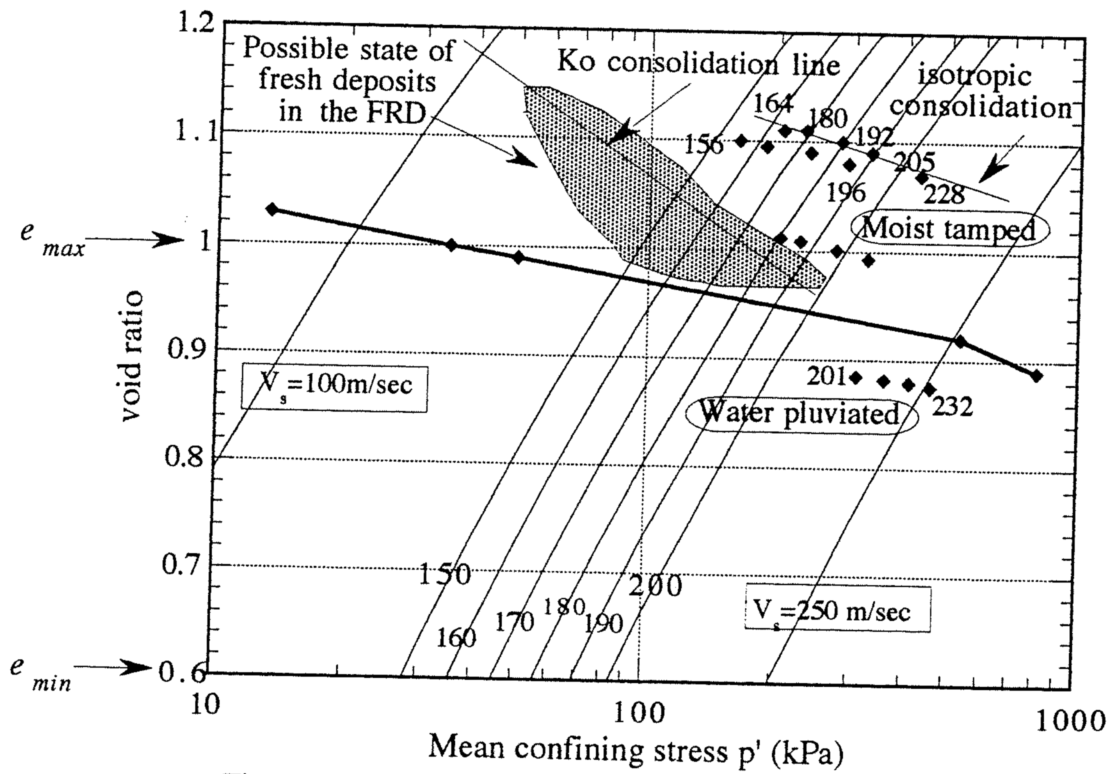
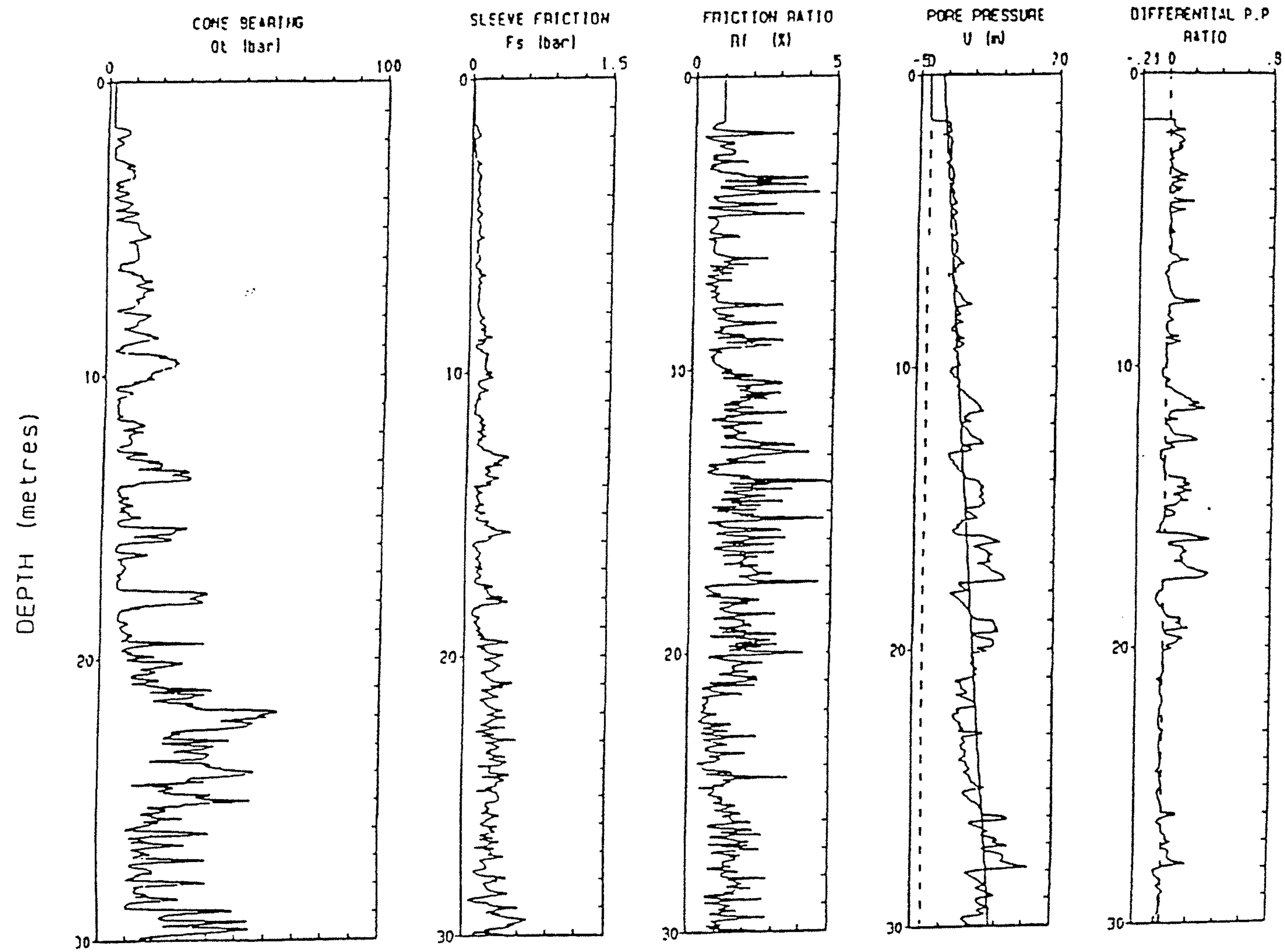



Figure Evaluation of the state of fresh deposits in the FRD

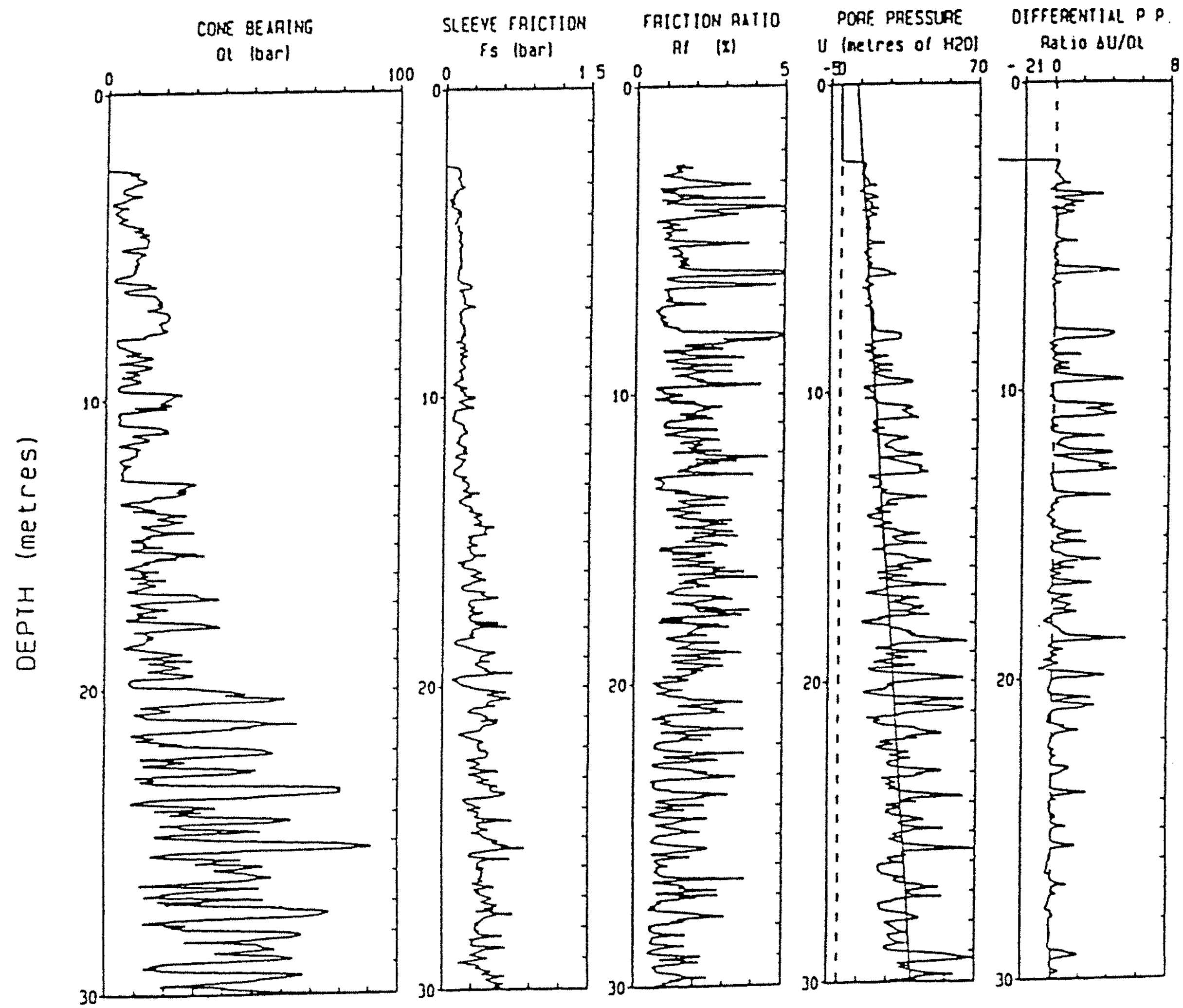
(4)




Depth Increment .05 m Max. Depth 30.95 m

Figure CPT2 profile at Site B (Figure 2) in the Fraser River Delta

5




Depth Increment .05 m Max. Depth 30.00 m

Figure CPT1 profile at Site D (Figure 2) in the Fraser River Delta

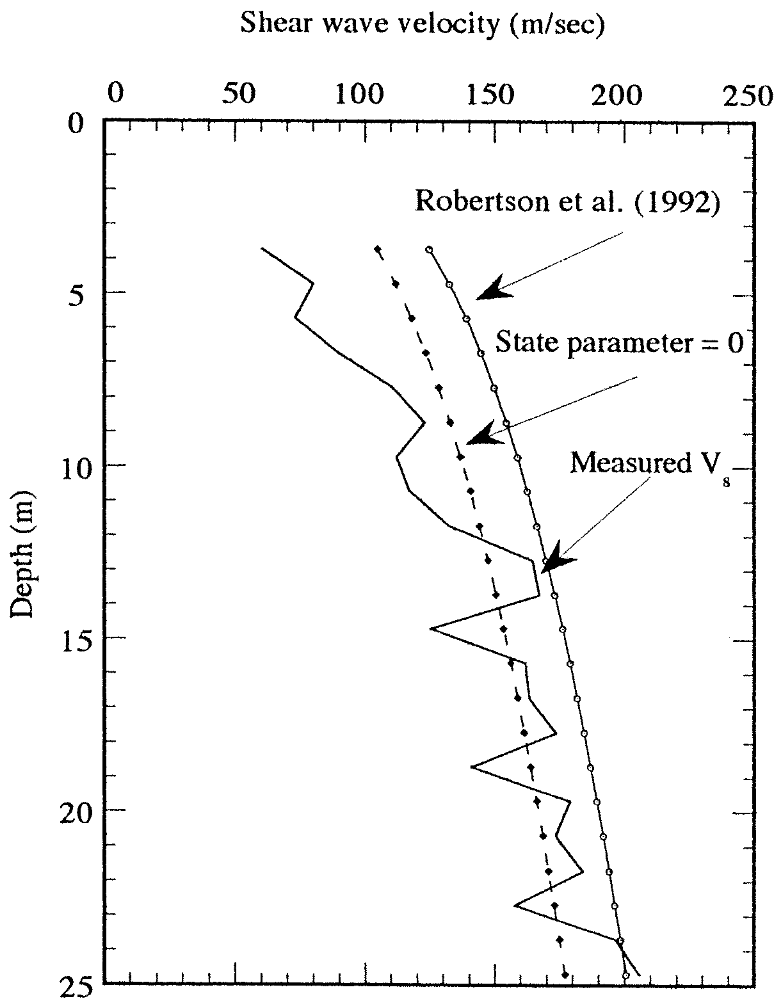



Figure  Assessment of liquefaction potential from shear wave velocity measurements