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Strain Level and Uncertainty of Liquefaction Related Index Tests

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Abstract

From observations of the performance of natural deposits and man made earth structures under static and earthquake undrained loading conditions, liquefaction resistance has been correlated to large strain measurements, e.g., corrected Standard Penetration Test blow count, $(N_1)_{60}$, and cone tip resistance, q_T. The shear wave velocity can be correlated to $(N_1)_{60}$ and q_T in some soils via weak statistical relationships. Therefore, shear wave velocity has also been proposed as an index of liquefaction resistance. Examination of data from three sites, however, shows that statistically tenable correlations do not exist between q_T and small strain shear modulus, G_{max}. Consequently, shear wave velocity, a very low strain measurement closely related to G_{max}, may not be an appropriate index of liquefaction resistance, which is a large strain phenomenon. In contrast, the statistical behavior of the pressure expansion curves measured in a self-boring pressuremeter test is quite similar to that of cone tip resistance at these sites. Thus, the self-boring pressuremeter data may be a viable alternative for assessment of liquefaction potential. An analytical procedure to derive the properties related to the undrained loading response of cohesionless soils from a self boring pressuremeter test has been proposed. The procedure is illustrated with an example.

Introduction

Soil resistance to liquefaction can be directly estimated from laboratory testing of undisturbed sand samples obtained via ground freezing. However,

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ground freezing is expensive to carry out and the sampling procedures may cause disturbance if executed without extreme care. As a result, indirect methods based on index tests are often employed for evaluation of liquefaction potential instead of laboratory tests. For instance, SPT (Standard Penetration Test) and CPTU (Piezocone Penetration Test) are used routinely as index tests for static and cyclic liquefaction resistance. The shear wave velocity (V_S) has also been suggested as an index of soil liquefaction resistance in static (Fear and Robertson, 1995) and cyclic (Robertson et al., 1991; Bierschwale and Stokoe, 1984) loading. An alternative approach has been proposed by Byrne et al. (1995[b]) which derives the volumetric behavior of the soil from inverse modeling of in-situ cavity expansion test. From the volumetric behavior, static liquefaction potential can be assessed for the undrained state. This paper examines the feasibility of the procedures based on shear wave velocity and in-situ cavity expansion for estimating liquefaction resistance.

Indirect Methods of Assessment of Liquefaction Potential

Examination of several case histories shows that the SPT blow count duly corrected for overburden pressure and driving energy, $(N_1)_{60}$, can be related to the liquefaction resistance of soils (Seed, 1979). This approach has been accepted as a standard procedure for evaluation of liquefaction potential. Using an empirical relationship between q_T and SPT blow count, Robertson et al. (1983) extended the procedure based on SPT for evaluation of liquefaction potential to include cone penetration data. Both $(N_1)_{60}$ and q_T weakly correlate to the shear wave velocity, V_S, through site and soil specific relationships. Thus it has been argued (Bierschwale and Stokoe, 1984; Robertson et al., 1992) that the shear wave velocity can also be used as an index of liquefaction resistance.

Both CPTU and SPT induce very large shear strains in the soil around the probe (van den Berg, 1994). In comparison, the shear strain in a downhole seismic test for measuring the shear wave velocity is much smaller: usually less than 10^{-4} %. The liquefaction phenomenon, which involves generation of excess pore water pressure of a magnitude comparable to the confining pressure, occurs at shear strains of about 0.01 to 1 %. Since soil behavior depends on the strain level to a great extent, an ideal index test of liquefaction resistance should impart shear strains of similar magnitude in the medium. However, field performance of the large strain measurements (e.g., $(N_1)_{60}$ and q_T) as indicators of liquefaction resistance has been quite satisfactory. Therefore, to establish the validity of a small strain property as an index of liquefaction resistance, one needs to examine whether the statistical behavior of the small and large strain measurements are similar in a natural geologic environment. Such an exercise is undertaken in this paper.

Data over a wide strain range can be obtained from a self-boring pressuremeter test (SBPMT). Therefore, it appears logical to use the data from this test to evaluate liquefaction potential. To examine whether this approach is tenable, the statistical behavior of the self boring pressuremeter data was compared with that of CPTU.

Description of Sites

The data used in this study come from tests carried out at three sites in the site characterization activity of the Canadian Liquefaction Experiment (CANLEX) Project. The first site near Fort McMurray, Alberta is in a tailings dam operated by Syncrude Canada Limited. The tailings are deposited here inside a compacted perimeter dyke by hydraulic means. The coarse tailings are the first to settle out near the point of discharge forming a beach above water. This mode of deposition leads to the formation of layers of medium to dense sand ($D_{50} \approx 0.15$ mm). The target zone at this location was between 28 and 40 m depth and comprises primarily of beach deposits. The water table was at a depth of 21 m. For more details about the site refer to Sobkowicz and Handford (1990). Near the tailings dam, at a location called J-Pit, an earth embankment was built on a very loose foundation to conduct a field test on static liquefaction in Phase III of CANLEX Project. The foundation was carefully constructed by under water deposition of Syncrude Sand in the loosest possible state. Data from this site will be used in an illustrative example later in this paper.

The other two sites are situated in the Fraser River Delta near Vancouver, BC. The general stratigraphy at Massey Tunnel comprises of layers of clean loose sand between 6 m and 15 m depth. The depth of water table is approximately 2.5 m. The zone of interest at this location extends from a depth of 7 m to 15 m. At KIDD # 2, below a 1.5 m thick desiccated silt layer near surface, lies a unit of silty fine sand which grades to sand at a depth of about 6 to 8 m. Below the silty sand clean sand is found. The relative density of the clean sand layer ranges between 30 and 90 %. The data used in this study from this site pertains to the layers between 7 and 20 m depth. A more detailed description of the near surface geology of Massey Tunnel and KIDD # 2 has been given by Monahan et al. (1995). In the illustrative example of this paper, a cavity expansion test performed at Massey Tunnel will be analyzed.

The In-Situ Testing Program

A total of four CPTUs with seismic measurements were carried out at Syncrude tailings dam in the detailed site characterization activity in Phase I of the CANLEX Project. In the seismic measurements, a hammer and shear beam at surface was used as the seismic source. In two CPTUs an accelerometer was used as the receiver. In the other tests, a seismometer was used. In the SBPMT a probe with a central jetting system was used. In-situ cavity expansion tests were performed at three depths in this sounding. For the inflation of the probe in the layer of interest, bottled nitrogen at high pressure was used. The results from CPTU sounding indicates a uniform free draining cohesionless material in the target zone. These data show a fairly constant value of cone bearing varying between 12 and 16 MPa, and a fairly constant friction ratio of about 0.8 %. The shear wave velocities range between 212 and 298 m/s. Campanella et al. (1995) give a more detailed description of this testing program.

At Massey Tunnel, seven seismic piezocone penetration tests (SCPTU) and two SBPMTs were performed. At the KIDD # 2 site, eight SCPTUs and two SBPMTs were carried out. In the seismic measurements at Massey Tunnel and KIDD # 2, beam and hammer was used as the source of shear wave and an accelerometer was used as the receiver. A number of cavity expansion tests were performed at various depths in each of the four self-boring pressuremeter tests at these two sites. The self-boring pressuremeter used at Massey Tunnel employed a central jetting system while a shower head system was used at KIDD # 2. For inflation of the probe, compressed air was used. The piezocone penetration tests indicate a free draining cohesionless soil in the target zone at both the sites. At Massey Tunnel the cone tip resistance was between 4.5 and 9.0 MPa and a fairly constant friction ratio of 0.4 % was observed. The shear wave velocities varied between 150 and 220 m/s. The cone tip resistance was between 2 and 20 MPa and a uniform friction ratio of 0.25 % was measured at KIDD # 2. The shear wave velocities ranged between 150 and 230 m/s. More details regarding this testing program can be found in Reports on Activities 3A and 3B of the CANLEX Project (In-Situ Testing Group, UBC, 1995[a], 1995[b]).

Relationship Between G_{max} and q_T

Fig. 1 presents a scatter plot between the cone tip resistance and the small strain shear modulus at the sites described above. The values of G_{max} presented in the figure were estimated from shear wave velocities from downhole SCPTU measurements using the relationship $G_{max} = \rho V_S^2$ and an appropriate value of the total mass density, ρ, of the medium. To minimize the effects of density on the G_{max}-q_T relationship, data from layers at similar densities were only used. The frequency distributions of the relative densities estimated from SCPTU data following Robertson and Campanella (1986) are also shown in Fig. 1. The statistics of linear regression between log q_T and log G_{max} indicate that the correlation is poor for the all the sites (Table 1).

Significant scatter has also been reported by other researchers who examined similar data (e.g., Rix and Stokoe, 1991; Lee, 1992). It is therefore apparent that small strain properties do not correlate to large strain measurements

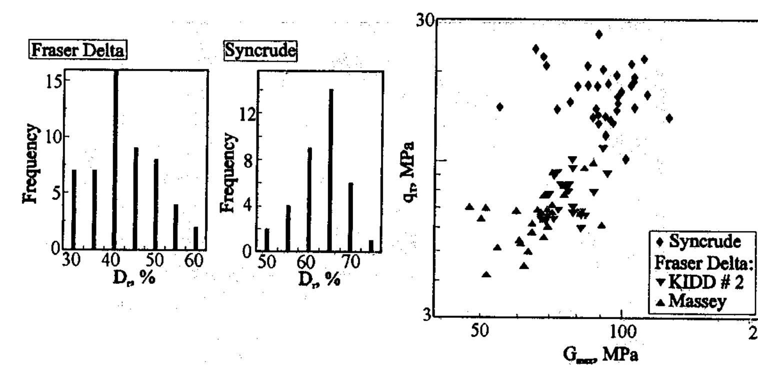


Fig. 1. Relationship of Cone Tip Resistance and Small Strain Shear Moduli

precisely. Since the large strain measurements - q_T or $(N_1)_{60}$ - correlate with the resistance of soils to liquefaction, a small strain property (V_S or G_{max}) may not be an appropriate indicator of liquefaction resistance.

Variability in Self-boring Pressuremeter Test Data

The data used in this exercise are plotted in Fig. 2. The effective cavity pressure in an SBPMT is calculated by subtracting the ambient pore water pressure from the gas pressure inside the cavity. The cavity strain, ϵ_θ, is equal to the radial deformation divided by the original radius of the cavity. The cone

Table 1. Statistics of the Correlation Between G_{max} (MPa) and q_T (MPa).

Site	log q_T =	# of Obs	r^2	|t|	tabled $t_{0.05}$
Massey & KIDD # 2	0.832 log G_{max} - 0.697	53	0.337	5.09	
Massey	0.780 log G_{max} - 0.608	32	0.301	3.60	2.042
KIDD # 2	0.620 log G_{max} - 0.295	21	0.096	1.40	2.093
Syncrude	0.227 log G_{max} + 0.765	35	0.035	1.09	2.042

Note: If |t| exceeds the tabulated values of two-tailed t distribution, the null hypothesis can be rejected, i.e., the coefficients of log G_{max} are significant to confidence levels better than 5 %.

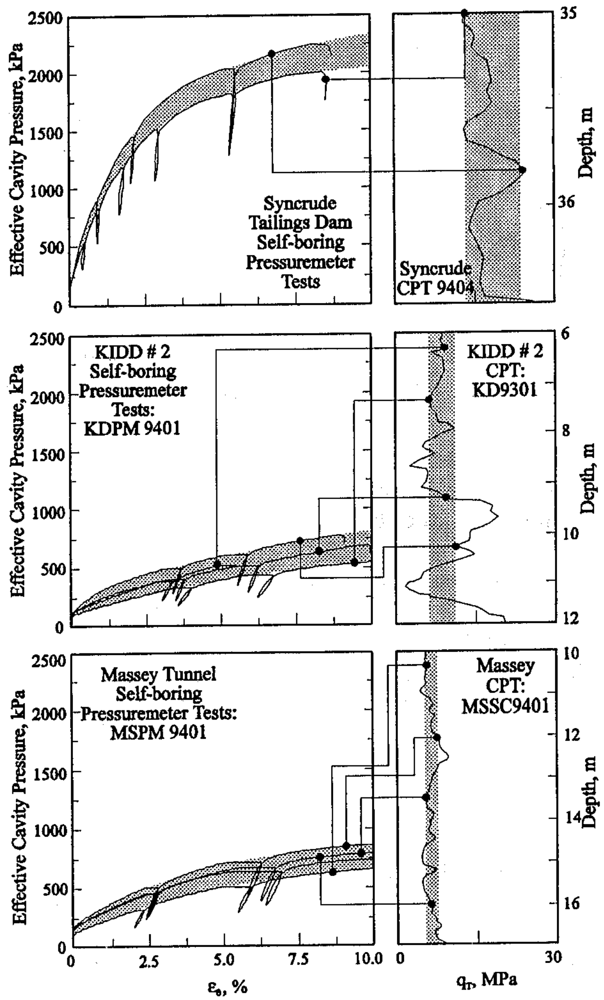


Fig. 2. Variability in the In-Situ Cavity Expansion Data

tip resistance from CPTUs carried out adjacent to the locations of the SBPMTs are also shown in the figure.

For simplicity, we assume that the spread, i.e., the difference between the maximum and the minimum observed value, in a measured quantity is a good statistical representation of its variability. To establish the similarity (or contrast) in the statistical behavior of the self-boring pressuremeter data and the cone tip resistance, we proceed as follows.

- Existence of a qualitative relationship between the pressure-expansion response in an SBPMT and cone tip resistance can be inferred from the data because the soil layers with larger cone tip resistance also show stiffer pressure-expansion response at relatively large values of ϵ_θ.
- We know in addition that the large strain measurement, q_T, exhibit a finite variability. Thus, for the pressure expansion curves to have a similar statistical characteristic, the spread in the latter must also be finite at very large strains. To check for this, we plot the observed spread in pressure expansion curves versus ϵ_θ (Fig. 3). This plot clearly shows that the spread in the pressure expansion response increases quickly to a finite value at $\epsilon_\theta \approx 2.5$ % and remains largely unchanged thereafter.

Therefore it can be concluded that the statistical behavior of the pressure expansion response in an SBPMT and the measured cone tip resistance is quite similar for a cohesionless geologic medium. Thus, like CPTU, the self-boring pressuremeter test may provide useful information in liquefaction studies. A simple procedure for estimation of liquefaction related soil properties from the pressure expansion data is outlined in the following section.

Interpretation of In-Situ Cavity Expansion Test

The interpretation basically involves derivation of the element behavior from the cavity expansion data (Fig. 4). First, the drained response of a cylindrical cavity under increasing internal pressure is computed numerically using an appropriate constitutive model and reasonable estimates of model parameters. The response of the numerical model is compared with the measured response

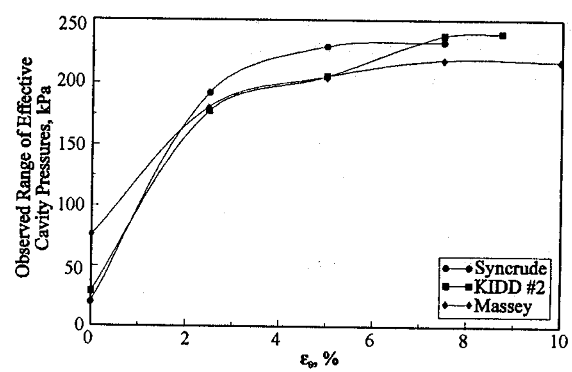


Fig. 3. Variability in the Observed Effective Cavity Pressure Over a Wide Strain Range

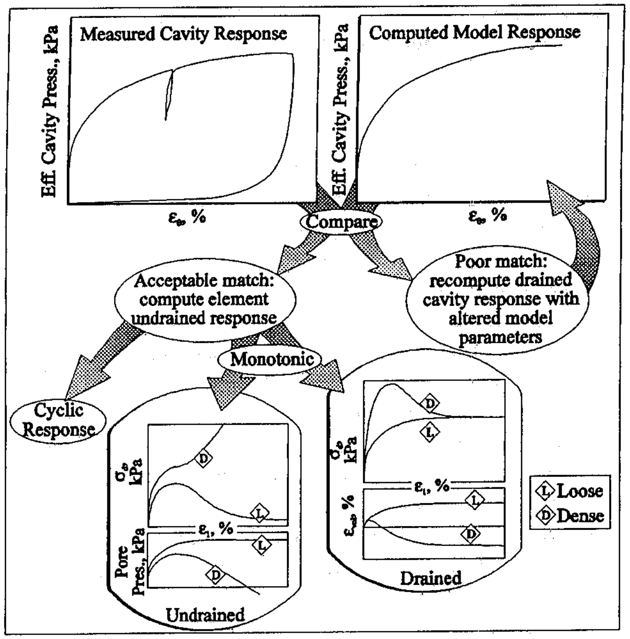


Fig. 4. Interpretation of In-Situ Cavity Expansion Test: a Flow Chart

in a cavity expansion test. The model parameters are altered until a reasonable match between the model and the measured response is achieved. Once a reasonable match is achieved, the model parameters are used to compute the undrained cyclic or monotonic response of a soil element. Although computation of cyclic element response provides useful quantitative information such as number of cycles to liquefaction, in this paper we will only consider the characteristics of monotonic undrained behavior for simplicity. The undrained response is computed from the drained parameters by considering the volumetric constraint of pore water.

Typically, the undrained monotonic response of a soil element exhibits one of the following characteristics.

- For very loose sands the pore water pressure increases to a value nearly equal to the in-situ confining stress and does not decrease upon further straining. The deviator stress reaches a peak value following which it decreases continuously until it reaches a small residual value at large strains. A sample showing this type of response is susceptible to flow failure or static liquefaction.

- Medium dense sands also generate a significant pore water pressure in undrained loading. Consequently, the deviator stress decreases to a residual value following a peak. However, upon further straining the material tends to dilate. Consequently, the pore water pressure starts to decrease and the shear strength increases. A deposit showing this type of behavior may develop significant strain in an undrained loading but a catastrophic flow failure may be precluded.
- Dense sands do not exhibit any strain softening under undrained loading. In this case, the pore water pressure starts to decrease after reaching a relatively small positive value as the skeleton tends to dilate.

As has been mentioned earlier, the estimated model parameters can be used for computing a cyclic response. From the computed cyclic response, design parameters such as number of cycles to liquefaction can be found. The derived model parameters can also be used for computing the cyclic response of an earth structure under an arbitrary loading.

The Constitutive Model

The interpretation procedure essentially involves inverse modeling of observed response of a cylindrical cavity under increasing internal pressure in a continuum to derive the model parameters. The non-linear inverse problem is solved by manual iteration. The solution process is greatly facilitated when a constitutive model with the following desirable characteristics is adopted:

- capability to capture soil behavior reasonably using a few parameters,

and

- availability of reliable a-priori estimates of the magnitudes of some model parameters and a reasonable knowledge about the bounds of the values of majority of the remaining parameters.

A constitutive model fulfilling these requirements has been developed by Byrne et al. (1995[a]) and was used in analyses presented later. The constitutive model is described very briefly as follows. For more details refer to Byrne et al., (1995[a], 1995[b]). The model defines stress ratio, η, as the ratio of the maximum shear stress, t, to the mean normal stress, s. In a two dimensional loading (e.g., plane strain), $t = (\sigma_1'-\sigma_3')/2$ and $s = (\sigma_1'+\sigma_3')/2$, where σ_1' and σ_3' are the major and the minor principal effective stresses, respectively. For determining the magnitude of the plastic strain increment vector on the plane of maximum shear, the following relationship between the stress ratio increment, $d\eta$, and the plastic shear strain increment, $d\gamma^p$, is assumed:

$$\frac{d\eta}{d\gamma^p} = K_g^{\,p}\left(\frac{s}{P_a}\right)^{n_p-1}\left(1 - R_f\frac{\eta}{\eta_f}\right)^2 \qquad (1)$$

where K_g^p, n_p and R_f are model parameters, P_a is the atmospheric pressure, and $\eta_f = \sin\phi'$. The peak effective stress friction angle is given by:

$$\phi' = \phi_1' - \Delta\phi \times \log[s/P_a] \tag{2}$$

where ϕ_1' is the peak friction angle at s = 1 atmosphere and $\Delta\phi$ is the decrease in the friction angle for a 10 fold increase in s. The peak effective stress friction angle in plane strain loading is larger than the corresponding triaxial values by 4° (for loose sands) to 9° (for dense sands), Lee, 1970. Following Taylor (1948), the direction of the plastic strain increment is determined from:

$$d\epsilon_v^{\,p}/d\gamma^p = \sin\phi_{cv} - \eta \tag{3}$$

where $d\epsilon_v^p$ is the incremental plastic volumetric strain and ϕ_{cv} is the constant volume friction angle. Upon determination of the plastic strain increment on the plane of maximum shear from Eqs. (1) and (3), the plastic strain increments in the coordinate directions are computed. For calculating the elastic strain increment for a given stress increment, two material constants are needed: the bulk modulus, B_e, and the shear modulus, G_e. The stress level dependency of these elastic moduli is modeled by the following expressions:

$$\begin{aligned} B_e &= K_b^{\,e} P_a (s/P_a)^{m_e} \\ K_e &= K_g^{\,e} P_a (s/P_a)^{n_e} \end{aligned} \tag{4}$$

where K_b^e, m_e, K_g^e and n_e are model parameters. The elastic and the plastic strain increments are added to compute the total strain increment.

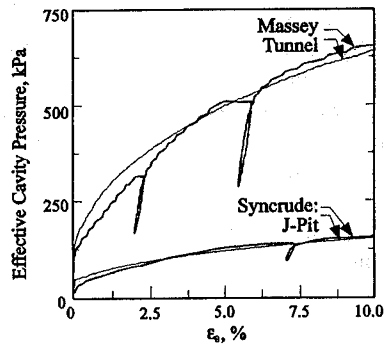
A-Priori Information on Model Parameters

Installation related disturbance near the borehole wall in a pressuremeter test affects the reliability of the data at low strains. This renders the estimation of the elastic parameter, K_g^e, from the virgin pressure expansion curve of an SBPMT difficult. The problem can be avoided by using supplementary seismic information from an adjacent downhole SCPTU which gives a more precise estimate of K_g^e. Alternatively, the unload reload data from loops performed during an SBPMT can be used. In the analyses presented in the following section, we use downhole seismic measurements to calculate K_g^e. In the absence of material specific information, for loose medium and dense granular materials K_g^e may be assumed to be approximately 300, 600 and 1200, respectively. The corresponding bulk modulus can be estimated assuming a value of Poisson's ratio between 0.11 and 0.23 (Hardin, 1978). The exponents, n_e and m_e, are equal to about 0.5 for a wide range of sands (Duncan et al., 1980). The constant volume friction angle, ϕ_{cv}, for many types of sand have been reported in the literature (Sasitharan et al., 1994) and is known to be independent of history of loading and sample fabric. A value of $0.5\times(\phi_1'-\phi_{cv})$ can be assumed for $\Delta\phi$ in the absence of material specific information. The parameter R_f varies from a value of 1.0 for very loose deposits to about 0.75 for dense sands. The remaining model parameters, K_g^p, n_p and ϕ_1' are estimated from fitting the model to test data over a wide strain range, e.g., data from an SBPMT or a laboratory element test on an undisturbed sample. Although both K_g^e and K_g^p increase with density, it follows from the earlier discussion on comparison of small and large strain material properties that these parameters may not correlate with each other well. Actually, K_g^e and K_g^p are affected to a different degree by prestraining, density, ageing and fabric.

Analysis of Self-boring Pressuremeter Test

Two cavity expansion tests performed at Massey Tunnel and J-Pit were analyzed as plane strain problems to illustrate the principles of the proposed procedure. A finite difference computer code (FLAC version 3.2; Cundall, 1993) was used in the computation. Large strains were accommodated in the simulation by updating the nodal coordinates during loading. The appropriate value of K_g^e was calculated from the down-hole shear wave velocities from an adjacent SCPTU. The predicted and observed cavity response from two tests are shown in Fig. 5. The model parameters for the simulated response are also listed in the figure.

The derived model parameters were used to compute the expected element behavior in an anisotropically consolidated undrained triaxial extension test (Fig. 6). Plotted in this figure is the deviator stress, σ_d, and $\Delta u/\sigma'_{hc}$ against the axial strain, ϵ_{axial}. Δu is the pore water pressure and σ'_{hc} is the horizontal stress at consolidation. Undisturbed samples were obtained from these sites for laboratory testing. The laboratory triaxial extension test data for undisturbed samples M94 F4 C4-2 (Massey Tunnel) and FS26 C3-1 (J-Pit) are also shown in Fig. 6. In M94 F4 C4-2 and FS26 C3-1, the samples were consolidated to a horizontal effective stress of 62 and 56 kPa, respectively. The corresponding vertical stresses were twice those values. The observed behavior compares reasonably with the element response predicted from the self-boring pressuremeter tests. It appears, therefore, that in a more elaborate analysis of an earth structure comprising of similar material, the estimates of the material parameters can be used to compute the undrained response under an arbitrary loading.



Parameters

Massey Tunnel: MSPM9401, 10.30m depth, σ_h'= 71 kPa	Syncrude: J-Pit CANT4, 4.8 m depth, σ_h'= 30 kPa
$K_b^e = 770$	$K_b^e = 470$
$K_s^e = 615$	$K_s^e = 360$
$K_s^p = 1845$	$K_s^p = 290$
$n_e = m_e =$	$n_e = m_e =$
$n_p = 0.5$	$n_p = 0.5$
$R_f = 0.90$	$R_f = 0.95$
$\phi_{cv} = 32°$	$\phi_{cv} = 31°$
$\phi_i' = 40°$	$\phi_i' = 36°$
$\Delta\phi = 3°$	$\Delta\phi = 2.5°$

Fig. 5. Modeling of Self-boring Pressuremeter Tests

Summary and Conclusions

The performance of large strain tests such as SPT and CPTU in assessment of liquefaction potential is understood to be quite satisfactory. In this paper we examined whether G_{max} (or V_S) and self-boring pressuremeter tests have relevance in liquefaction studies. To check for their feasibility, we compared the statistical behavior of G_{max} (or V_S) and the pressure expansion curves in an SBPMT with that of cone tip resistance. Since the pressure expansion response in a self-boring pressuremeter test and the cone tip resistance exhibit similar statistical qualities, the pressuremeter test appears to have some relevance in liquefaction related studies. However, the correlation between the cone tip resistance and G_{max} (or V_S) was found to be statistically untenable. Hence, the relevance of the small strain shear modulus or shear wave velocity in liquefaction assessment becomes questionable. A procedure for the interpretation of self-boring pressuremeter tests to derive liquefaction related material properties was proposed. The technique was illustrated and evaluated against laboratory experiments. The results indicate that the model parameters back figured from pressuremeter data can predict the undrained monotonic response of cohesionless soils.

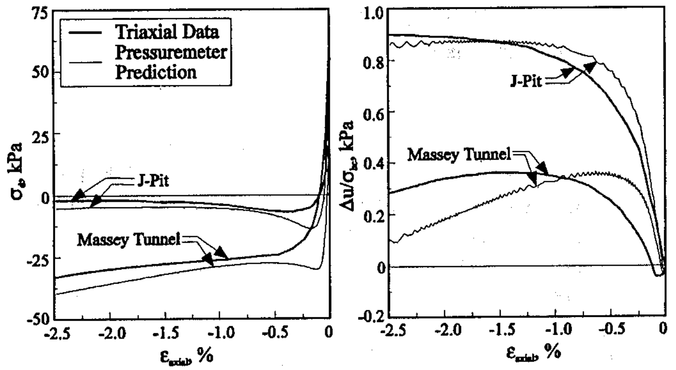


Fig. 6. Derived and Observed Element Behavior

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