# USE OF RECENT CONE PENETRATION TEST TECHNOLOGY IN EVALUATING GEOTECHNICAL PROPERTIES OF MINE WASTE

By

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## SYNOPSIS

The in situ determination of the geotechnical properties, groundwater conditions and the evaluation of liquefaction potential of mine tailings using recent cone penetrometer technology is described. Methods for evaluating soil behaviour type and the following soil parameters are presented: relative density, $D_r$; friction angle, $\emptyset$; soil moduli, and pore water pressure, u. The evaluation of liquefaction potential using the cone bearing resistance, $Q_c$, and the shear wave velocity, $V_s$, is described. Also, techniques for evaluating fines content, groundwater gradients and groundwater contamination are discussed.

## INTRODUCTION

This paper discusses recent cone penetration test technology, in particular the seismic cone penetration test (SCPT) as a technique for evaluating the geotechnical properties, including the susceptibility to liquefaction, of mine tailings. The SCPT is becoming increasingly popular as an in situ test for detailed and economic site investigation and geotechnical design. The SCPT is particularly appropriate for use on mine tailings backfill and tailings dams constructed by hydraulic filling techniques, where the tailings are comprised of materials which typically range from clay to sand size. In these types of materials, the SCPT is perhaps the best technique for determining stratigraphic sequencing and geotechnical properties.

From the recorded independent values of tip resistance, $Q_c$; sleeve friction, $F_s$; pore pressure, $U_t$ and shear wave velocity, $V_s$, certain critical geotechnical parameters can be determined and subsequently used in both static and dynamic analyses relating to problems involving fine mine waste.

In recent years the SCPT has been used successfully to accurately and economically evaluate the geotechnical characteristics of many different types of tailings including those derived from coal, copper, gold, gypsum, iron molybdenum, platinum and uranium mining operations.

This paper briefly discusses the seismic cone penetration test. Recommended ways in which to determine the following soil parameters are presented: relative density, $D_r$; friction angle, $\phi$; soil moduli, and pore water pressure, u. The evaluation of liquefaction potential using the cone bearing resistance, $Q_c$ and the shear wave velocity, $V_s$ is described. Also, techniques for determining fines content, groundwater gradients and groundwater contamination are discussed.

## THE SEISMIC CONE PENETRATION TEST (SCPT)

### The Seismic Cone

A typical 10 ton subtraction type seismic cone is illustrated in Figure 1. The cone has a tip end area of 10 $cm^2$ and friction sleeve area of 150 $cm^2$. The cone is designed with an equal end area friction sleeve and a tip end area ratio of 0.85. The cone is pushed into the mine waste hydraulically using a drill rig with a down pressure capacity in the order of 10 tons. A pore pressure filter is located either directly behind the cone tip or on the face of the cone tip. The filter is made of porous plastic and is 5.0 mm thick. Typically the pore pressure elements are saturated in glycerin under vacuum pressure prior to the field program. Pore water pressure dissipations can be recorded at regular intervals during pauses in the penetration.

The cone is capable of recording the following independent parameters at 5 cm. depth intervals: tip resistance, ($Q_c$); sleeve friction, ($F_s$); dynamic penetration pore pressure, ($U_t$); temperature, (T); and cone inclination, (i). These parameters are printed simultaneously on a printer during penetration and stored on a bubble cassette cartridge for future reference. The data stored on the bubble cassette is easily transferred to a personal computer for cataloging and analyses.

The cone is also instrumented to record both shear and compressional wave arrivals. The original equipment and procedures for conducting the seismic wave velocity measurements are as developed by Rice (1984), Laing (1985) and Robertson et al. (1986). Seismic wave velocity measurements are typically conducted at 1 m intervals during pauses in penetration for the addition of push rods. Before taking the wave velocity measurements the cone rods should be decoupled from the drill rig to avoid the transmission of vibrational energy down the rods.

Shear waves are generated by horizontally striking a steel beam with a trigger activating 7.5 kg. hammer. The beam is typically held down by the rear stabilizers of the drill rig (Figure 2). A reaction weight of about 10 tons provides excellent coupling of the beam to the ground surface. The beam is offset from the cone and this distance should be accounted for in all velocity calculations. Compressional waves are generated by vertically striking a solid stem auger with the hammer. The auger is screwed vertically into the mine waste to a depth of about 1.5 metres. The auger is offset from the cone and again, this distance should be accounted for in all calculations. The shear and compressional waves are detected by one or more horizontally active geophones or accelerometers located in the cone. The geophone signal is typically amplified prior to processing in a storage oscilloscope. The sampling frequency during testing is in the order of 50 kHz.

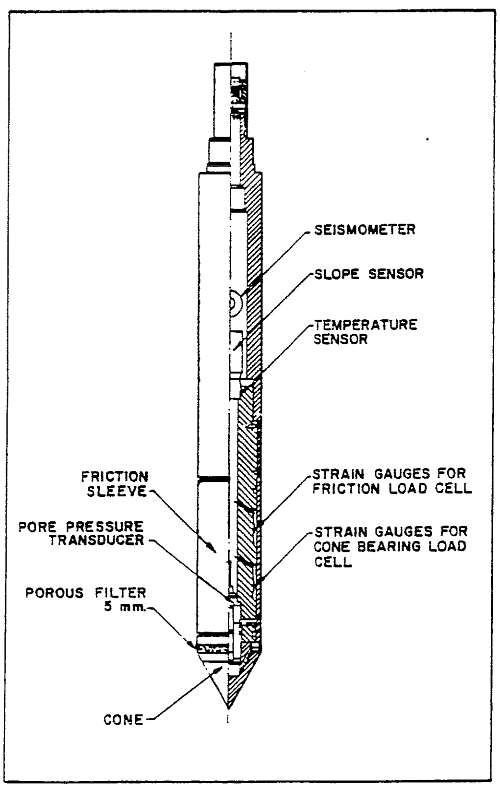



Fig. 1. SEISMIC PIEZOCONE

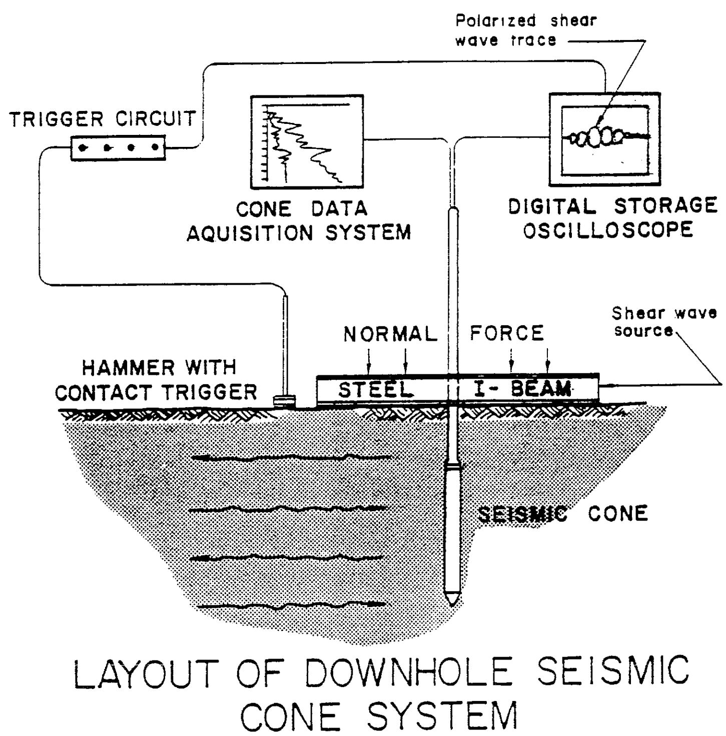



Fig. 2. DOWNHOLE SEISMIC CONE SYSTEM

Polarized wave traces are obtained for shear wave velocity determination by striking each end of the beam. These polarized wave traces are overlayed and the arrival and crossover markers (Fig. 3a) determined. The shear waves generated by this type of source are vertically propagating waves with a horizontal particle motion. They have a small shear strain amplitude (<0.001%) and are thus representative of the small strain behaviour of the soil skeleton. A single wave trace is obtained for compressional wave velocity determination by striking the auger once. The wave trace is inspected and the p-wave arrival (Fig. 3b) is determined. The compressional waves generated by this type of source are vertically propagating with a corresponding vertical particle motion.

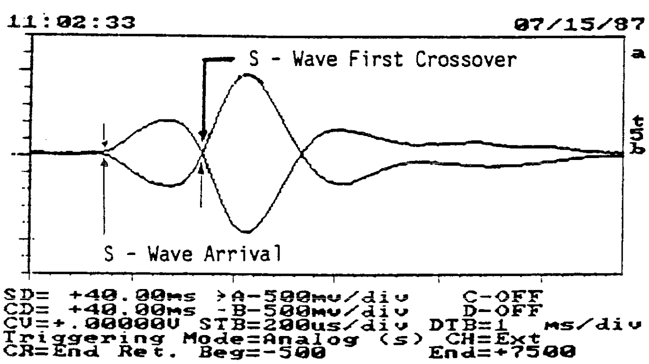



Fig. 3a SHEAR WAVE ARRIVAL

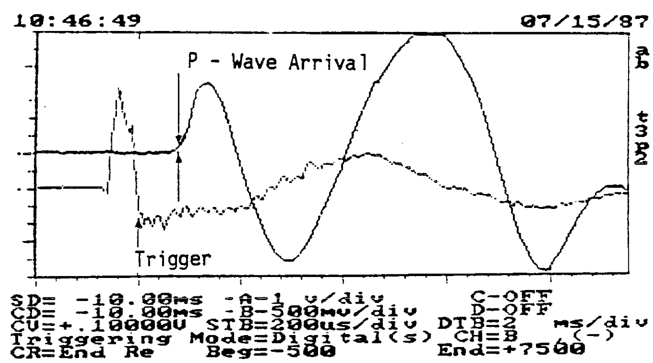



Fig. 3b COMPRESSIONAL WAVE ARRIVAL

The shear and compressional wave arrivals are converted to interval velocity measurements using the arrival times determined for each 1 metre testing interval. Each arrival is corrected for the horizontal offset of the source by assuming a straight line travel path. This correction is only significant in the upper several metres. From the corrected times and change in depth, the interval velocities are calculated. The interval velocity is representative of the material between the depths where the measurements are taken.

## Typical Seismic Cone Data

Figures 4a, 4b and 4c illustrate the typical field data collected during a seismic cone penetration test and some of the parameters determined from the field data.

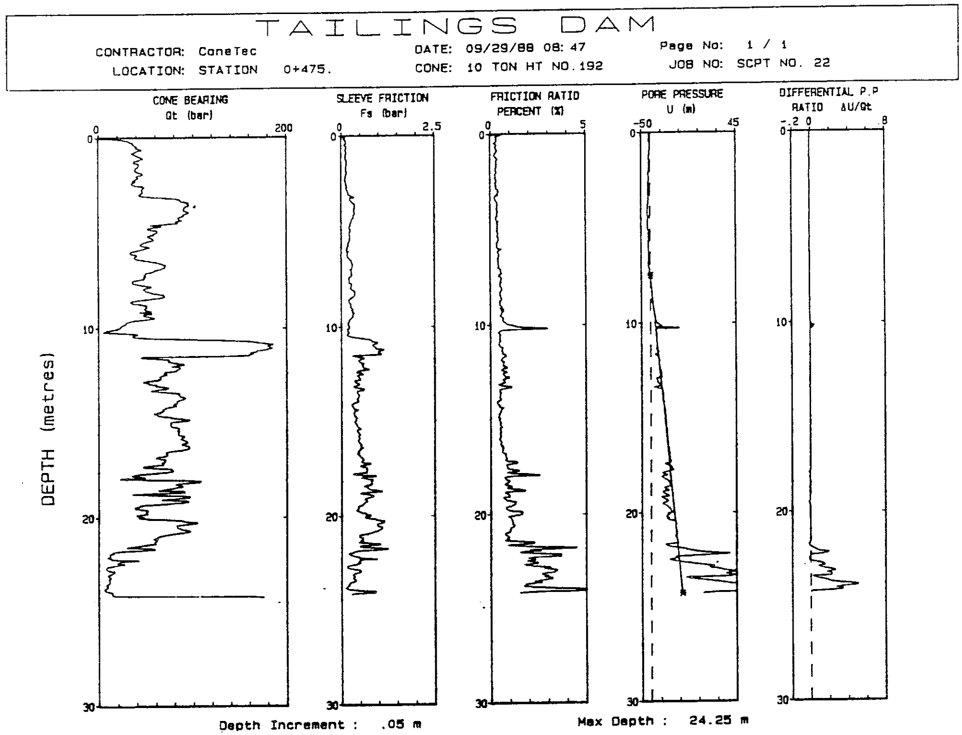



Fig. 4a CONE PENETRATION TEST DATA

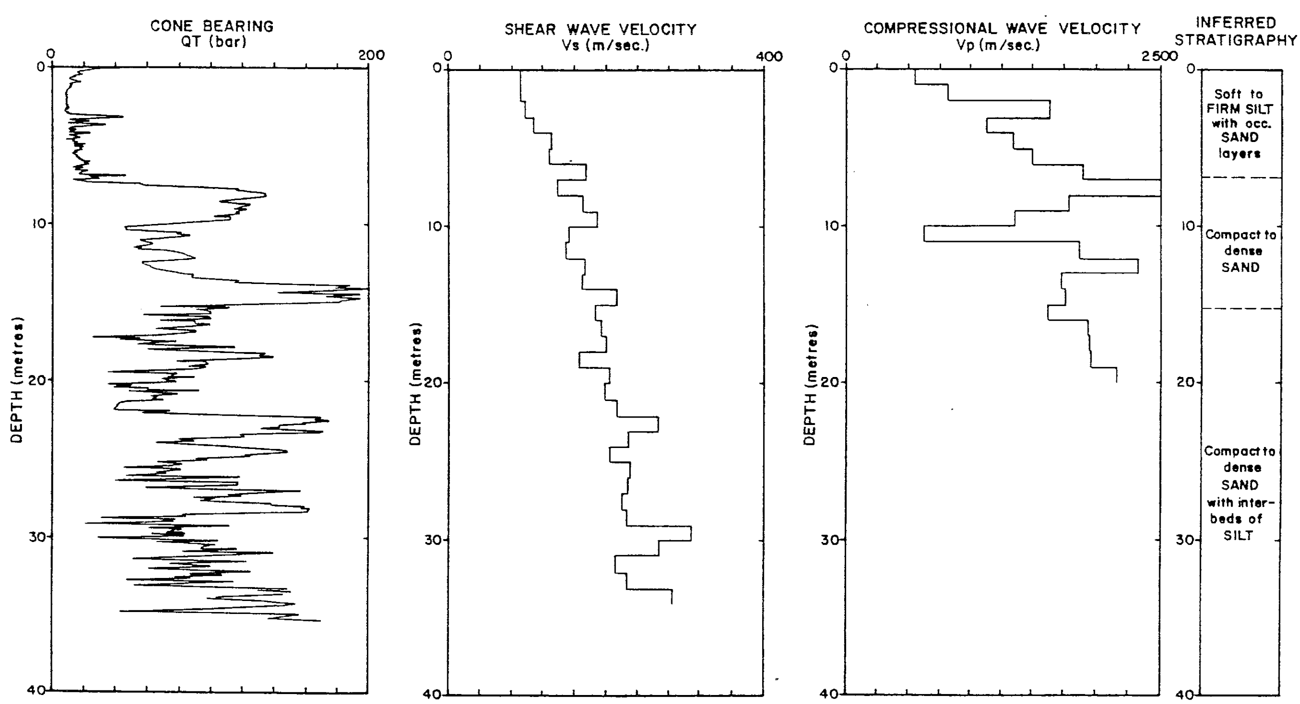



Fig. 4b SEISMIC CONE DATA

TAILINGS DAM

Contractor: ConeTec
Comments :STATION 0+475.
On Site Loc:SCPT NO. 22
Tot. Unit Wt. (avg) : 19 kN/m^3

CPT Date :09/29/88 08:47
Cone Used :10 TON HT NO. [illegible]
Water table (meters) : 7.5

| DEPTH (meters) | (feet) | Qc (avg) (bar) | Fs (avg) (bar) | Rf (avg) (%) | SIGV' (kPa) | SOIL BEHAVIOUR TYPE | Eq - Dr (%) | PHI deg. | SPT N | Su kPa |
|---|---|---|---|---|---|---|---|---|---|---|
| 0.25 | 0.82 | 30.42 | 0.11 | 0.37 | 2.37 | silty sand to sandy silt | 60-90 | >48 | 10 | UNDEFINED |
| 0.50 | 1.64 | 39.40 | 0.12 | 0.31 | 7.12 | sand to silty sand | 70-80 | 46-48 | 10 | UNDEFINED |
| 0.75 | 2.46 | 42.22 | 0.14 | 0.34 | 11.87 | sand to silty sand | 70-80 | 44-46 | 11 | UNDEFINED |
| 1.00 | 3.28 | 40.80 | 0.14 | 0.35 | 16.62 | sand to silty sand | 60-70 | 42-44 | 10 | UNDEFINED |
| 1.25 | 4.10 | 42.86 | 0.13 | 0.30 | 21.37 | sand to silty sand | 60-70 | 42-44 | 11 | UNDEFINED |
| 1.50 | 4.92 | 42.30 | 0.16 | 0.37 | 26.11 | sand to silty sand | 60-70 | 40-42 | 11 | UNDEFINED |
| 1.75 | 5.74 | 39.98 | 0.15 | 0.37 | 30.86 | sand to silty sand | 50-60 | 38-40 | 10 | UNDEFINED |
| 2.00 | 6.56 | 44.84 | 0.15 | 0.34 | 35.61 | sand to silty sand | 50-60 | 38-40 | 11 | UNDEFINED |
| 2.25 | 7.38 | 40.40 | 0.13 | 0.32 | 40.36 | sand to silty sand | 50-60 | 38-40 | 10 | UNDEFINED |
| 2.50 | 8.20 | 37.58 | 0.13 | 0.34 | 45.11 | sand to silty sand | 50-60 | 36-38 | 9 | UNDEFINED |
| 2.75 | 9.02 | 47.72 | 0.16 | 0.33 | 49.86 | sand to silty sand | 50-60 | 36-38 | 12 | UNDEFINED |
| 3.00 | 9.84 | 46.08 | 0.19 | 0.40 | 54.60 | sand to silty sand | 50-60 | 36-38 | 12 | UNDEFINED |
| 3.25 | 10.66 | 68.68 | 0.27 | 0.39 | 59.35 | sand to silty sand | 60-70 | 38-40 | 17 | UNDEFINED |
| 3.50 | 11.48 | 92.66 | 0.28 | 0.30 | 64.10 | sand | 70-80 | 40-42 | 19 | UNDEFINED |
| 3.75 | 12.30 | 93.18 | 0.38 | 0.40 | 68.85 | sand | 70-80 | 38-40 | 19 | UNDEFINED |
| 4.00 | 13.12 | 93.64 | 0.36 | 0.39 | 73.60 | sand | 60-70 | 38-40 | 19 | UNDEFINED |
| 4.25 | 13.94 | 97.70 | 0.36 | 0.41 | 78.34 | sand | 60-70 | 38-40 | 18 | UNDEFINED |
| 4.50 | 14.76 | 85.36 | 0.30 | 0.35 | 83.09 | sand | 60-70 | 38-40 | 17 | UNDEFINED |
| 4.75 | 15.58 | 61.22 | 0.25 | 0.41 | 87.84 | sand to silty sand | 50-60 | 34-36 | 15 | UNDEFINED |
| 5.00 | 16.40 | 59.42 | 0.24 | 0.41 | 92.59 | sand to silty sand | 50-60 | 34-36 | 15 | UNDEFINED |
| 5.25 | 17.22 | 55.84 | 0.23 | 0.41 | 97.34 | sand to silty sand | 50-60 | 34-36 | 14 | UNDEFINED |
| 5.50 | 18.04 | 54.96 | 0.24 | 0.43 | 102.08 | sand to silty sand | 40-50 | 34-36 | 14 | UNDEFINED |
| 5.75 | 18.86 | 48.18 | 0.20 | 0.42 | 106.83 | sand to silty sand | 40-50 | 32-34 | 12 | UNDEFINED |
| 6.00 | 19.69 | 42.64 | 0.16 | 0.38 | 111.58 | sand to silty sand | 40-50 | 30-32 | 11 | UNDEFINED |
| 6.25 | 20.51 | 38.04 | 0.18 | 0.47 | 116.33 | silty sand to sandy silt | <40 | 30-32 | 13 | UNDEFINED |
| 6.50 | 21.33 | 46.56 | 0.22 | 0.46 | 121.08 | sand to silty sand | 40-50 | 30-32 | 12 | UNDEFINED |
| 6.75 | 22.15 | 68.68 | 0.30 | 0.44 | 125.83 | sand to silty sand | 50-60 | 34-36 | 17 | UNDEFINED |
| 7.00 | 22.97 | 64.88 | 0.29 | 0.45 | 130.57 | sand to silty sand | 50-60 | 32-34 | 16 | UNDEFINED |
| 7.25 | 23.79 | 51.28 | 0.29 | 0.56 | 135.32 | sand to silty sand | 40-50 | 30-32 | 13 | UNDEFINED |
| 7.50 | 24.61 | 45.58 | 0.24 | 0.54 | 140.07 | sand to silty sand | <40 | 30-32 | 11 | UNDEFINED |
| 7.75 | 25.43 | 40.44 | 0.24 | 0.59 | 143.59 | silty sand to sandy silt | <40 | <30 | 13 | UNDEFINED |
| 8.00 | 26.25 | 51.32 | 0.27 | 0.53 | 145.89 | sand to silty sand | 40-50 | 30-32 | 13 | UNDEFINED |
| 8.25 | 27.07 | 59.88 | 0.36 | 0.60 | 148.19 | sand to silty sand | 40-50 | 32-34 | 15 | UNDEFINED |
| 8.50 | 27.89 | 60.92 | 0.33 | 0.54 | 150.48 | sand to silty sand | 40-50 | 32-34 | 15 | UNDEFINED |
| 8.75 | 28.71 | 38.20 | 0.22 | 0.58 | 152.78 | silty sand to sandy silt | <40 | <30 | 13 | UNDEFINED |
| 9.00 | 29.53 | 50.28 | 0.27 | 0.53 | 155.08 | sand to silty sand | 40-50 | 30-32 | 13 | UNDEFINED |
| 9.25 | 30.35 | 51.72 | 0.33 | 0.63 | 157.37 | sand to silty sand | 40-50 | 30-32 | 13 | UNDEFINED |
| 9.50 | 31.17 | 53.82 | 0.32 | 0.59 | 159.67 | sand to silty sand | 40-50 | 30-32 | 13 | UNDEFINED |

Dr - All sands (Jamiolkowski et al. 1985) PHI - Durgunoglu and Mitchell 1975 Su: Nk= 12.5

**** Note: For interpretation purposes the PLOTTED CPT PROFILE should be used with the TABULATED OUTPUT from CPTINTR1 (v 3.02) ****

Fig. 4c INTERPRETATION OF CPT DATA

## DETERMINING GEOTECHNICAL PROPERTIES OF MINE WASTE

Based on cone penetration test field data collected over the years, soil classification charts have been developed and correlations for the estimation of soil parameters compiled. During the last ten years the accuracy of these correlations has been significantly improved based on large numbers of cone penetration tests carried out both in the field and in large calibration chambers. From the cone tip resistance, ($Q_c$); sleeve friction, ($F_s$); and dynamic ($U_t$) and equilibrium ($U_o$) pore water pressures, and by utilizing the correlations presented in the following charts (Figures 5, 6, and 7), material behaviour type, relative density, $D_r$; and friction angle, $\emptyset$; can be estimated.

It is important to note that the interpretation of cone penetration test data and the derivation of soil parameters are dependant on such characteristics as in situ stress state, stress and strain history, cementation, sensitivity, aging, anisotropy and fabric or structure. The effects of these characteristics on the interpretation of in situ tests have been discussed in detail by several researchers including: Jamiolkowski et.al, 1988, Robertson, 1989, and Robertson and Campanella, 1983.

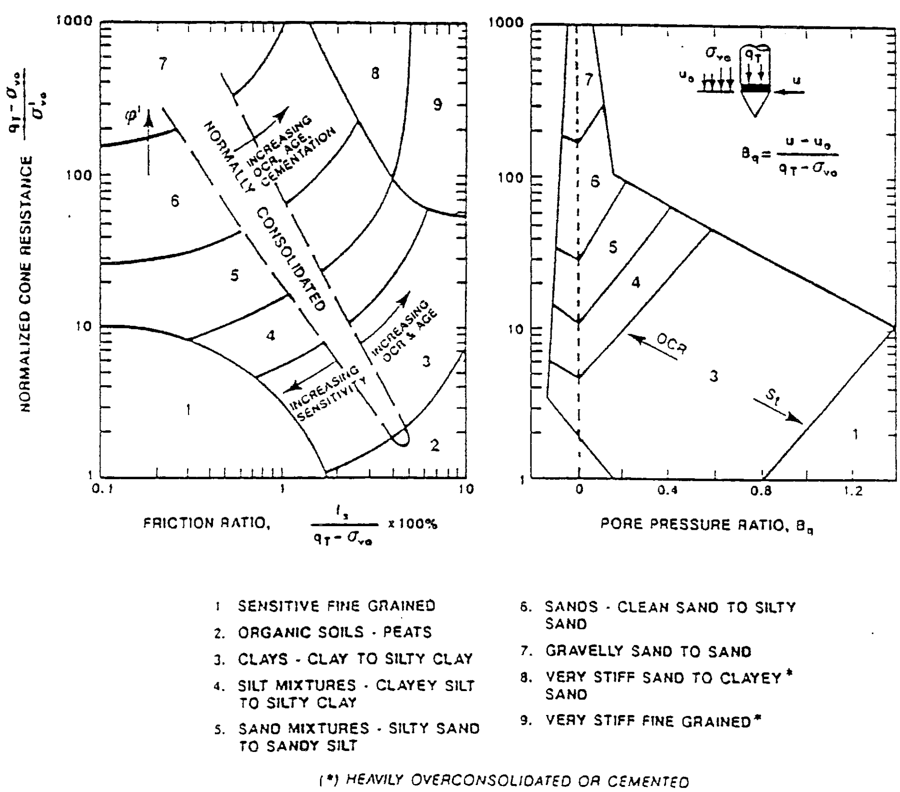



Fig. 5 SOIL BEHAVIOUR CLASSIFICATION CHART
BASED ON NORMALIZED CPT AND CPTU DATA
(Robertson, 1989)

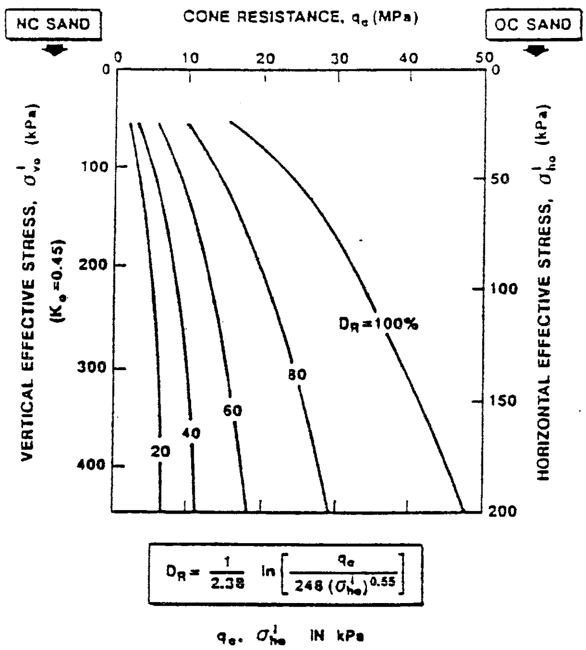



Fig. 6 RELATIVE DENSITY CORRELATION FOR UNCEMENTED, UNAGED SILICA SAND (Baldi et al, 1986)

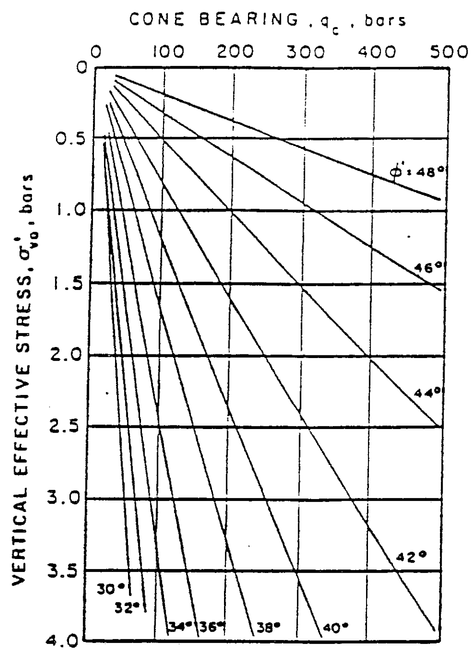



Fig. 7 PEAK FRICTION ANGLE CORRELATION FOR UNAGED UNCEMENTED SILICA SAND (Robertson et al, 1983)

Using the seismic wave velocities ($V_s$ and $V_p$) the small-strain moduli can be calculated directly. When carrying out deformation studies, soil modulus is the primary parameter. Soil modulus is primarily a function of effective stress level, induced strain level, and the imparted stress and strain history. For most well designed structures and foundations the average induced strain is typically less than about 0.1% (Jardine et.al., 1986, Jamiolkowski and Robertson, 1988). The determination of soil modulus in the laboratory at small strains is extremely difficult due to such factors as sample disturbance, the measurement of strain at the sample boundary and the simulation of in situ stresses. The measurement of the soil modulus from the in situ measurement of seismic wave velocities using the seismic cone is done at a known stress (in situ) and strain ($\gamma <$ 0.001%) greatly improving the accuracy of the measured modulus. Soil modulus is required for both simplified linear elastic analyses and more complicated non-linear elastic and elasto-plastic finite element analyses.

The small strain moduli and some additional elastic parameters can be calculated according to the following expressions:

- Shear Modulus : $G_{max} = \rho \, V_s^2$
- Constrained Modulus : $M_{max} = \rho V_p^2$
- Poisson's Ratio : $\mu = \dfrac{1\text{-}2\ (Vs/Vp)2}{2\text{-}2\ (V_s/V_p)^2}$
- Young's Modulus : $E = 2\rho V_s^2\ (1 + \mu)$
- Bulk Modulus : $B = \rho\ (V_p^2 - 4/3\ V_s^2)$

Strain amplitude for body waves can be calculated from the following equations:

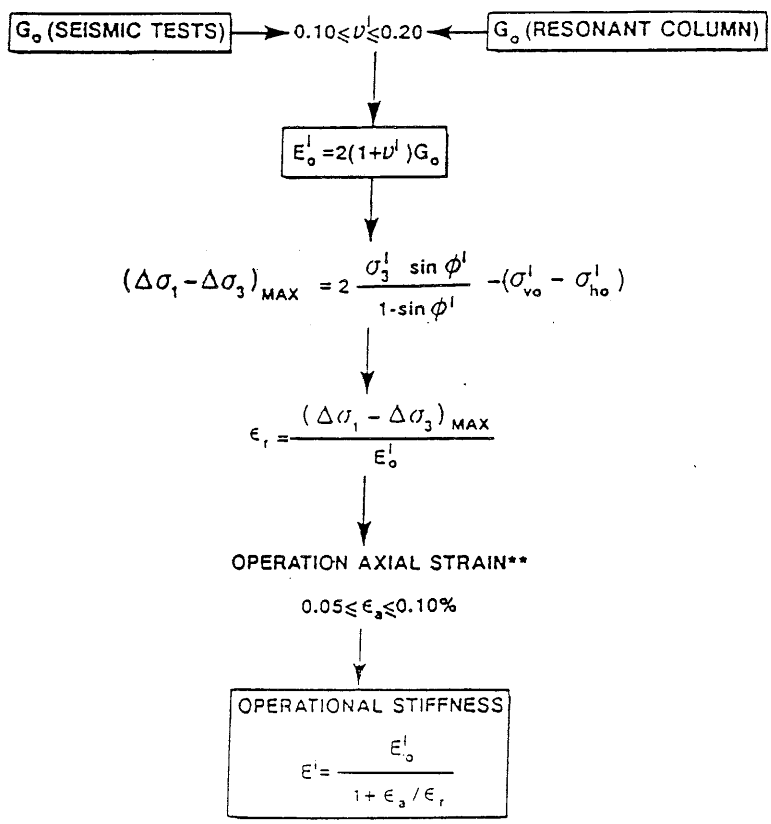
- Shear Strain : $\gamma = u_s/V_s$
- Normal Strain : $\xi = u_p/V_p$

For the above expressions $V_s$ is the shear wave velocity, $V_p$ is the compression wave velocity and $\rho$ is the mass density of the material ($\rho = \gamma/g$ where $\gamma$ is the total unit weight and g is the acceleration due to gravity). $u_s$ is the shear wave particle velocity and $u_p$ is the compression wave particle velocity.

In saturated soils the compression wave velocity ($V_p$) travels through the water phase with a velocity of about 1600 m/s. Therefore, the above expressions for $M_{max}$, $\mu$ and B are mainly applicable for unsaturated soil deposits. However, the upper layers of many tailings dams are often unsaturated and hence these expressions can be valuable.

The range of strain rates encountered in this type of a downhole seismic test is about 0.0001 to 0.01 in./in./sec (Stokoe et al., 1987). This range of strain rates is similar to rates experienced in static testing and in motions recorded during light to moderate earthquake shaking. Therefore the above approach for calculating parameters is applicable to many common geotechnical problems.

Figure 8 illustrates an approach that can be used to determine the drained Young's modulus (E′) for any strain level from the low strain shear modulus ($G_{max}$ or $G_0$) for granular soils. The Poisson's ratio to be used for determining E′ in drained soils typically ranges between about 0.1 and 0.2.




*(*) PROCEDURE APPLICABLE ONLY TO OC DEPOSITS*

*(**) BURLAND (1988); BATTAGLIO & JAMIOLKOWSKI (1987)*

Fig. 8 DETERMINATION OF E′ FOR GRANULAR SOILS FROM Gmax = Go (after Jamiolkowski et al, 1987)

DETERMINING MINE WASTE LIQUEFACTION SUSCEPTIBILITY

The empirical evaluation of liquefaction susceptibility of mine waste can be carried out effectively from the SCPT using the recorded values of $Q_c$ and $V_s$ (Finn et.al., 1989). The SCPT provides sufficient data to allow for liquefaction assessment using three widely used evaluation techniques. These are: (1) the cone penetration test (CPT) approach (Seed and deAlba, 1986); (2) the threshold shear wave velocity/shear strain approach (Dobry et al., 1981) and (3) the shear wave velocity approach (Stokoe et al., 1984). These various techniques provide ways in which to make a deterministic assessment of liquefaction potential for a specified design earthquake based on field performance during past earthquakes.

The critical cone penetration resistance ($Q_{c,crit}$) required to prevent liquefaction during earthquakes of any specified magnitude can be determined using the empirical liquefaction assessment chart developed by Seed and deAlba (1986) for non to low plasticity soils shown in Figure 9. A comparison between the normalized measured cone penetration resistance, $Q_{cn1}$, and the critical cone penetration resistance to resist liquefaction, ($Q_{c,crit}$) gives a clear indication of the potential for liquefaction. The critical values of cone penetration resistance depend on the magnitude, or more specifically the induced ground accelerations, of the given earthquake. Figure 10 illustrates the critical $Q_c$ as a function of depth for two levels of anticipated ground acceleration corresponding to two example design earthquake magnitudes compared to the measured $Q_{cn1}$ for the soil at a mine tailings site. The measured penetration resistances have been corrected for fines content and plotted as an equivalent $Q_c$ value with less than 5% fines to correspond to the data base compiled by Seed and his colleques. Figure 10 suggests that liquefaction is possible where the measured $Q_c$ is less than the critical $Q_c$ for a given earthquake magnitude.

The threshold strain/velocity approach evaluates the propensity for sandy and silty soils to develop excess pore water pressure during ground shaking. Dobry et.al. (1981) propose that if saturated cohesionless sands and silts have shear wave velocities less than the threshold shear wave velocity for a given seismic event, then during shaking there will be sufficient energy to compact the material thereby generating some excess pore water pressure. In order for liquefaction to occur, there must be the generation of excess pore water pressure. Although the threshold strain/velocity approach readily identifies soils which will develop pore water pressures during seismic excitation, this approach does not indicate whether the magnitude of the pore pressures generated will be large enough to cause liquefaction. Figure 11 compares the measured shear wave velocities at a mine site with the threshold velocity profiles corresponding to a threshold strain of 0.01% for peak accelerations as low as 0.10g. The data presented in this figure suggests that even at relatively low accelerations excess pore water pressures will be developed in the mine tailings. However, whether the magnitude of these pore pressures will be sufficiently large to cause liquefaction will depend on the magnitude and duration of shaking.

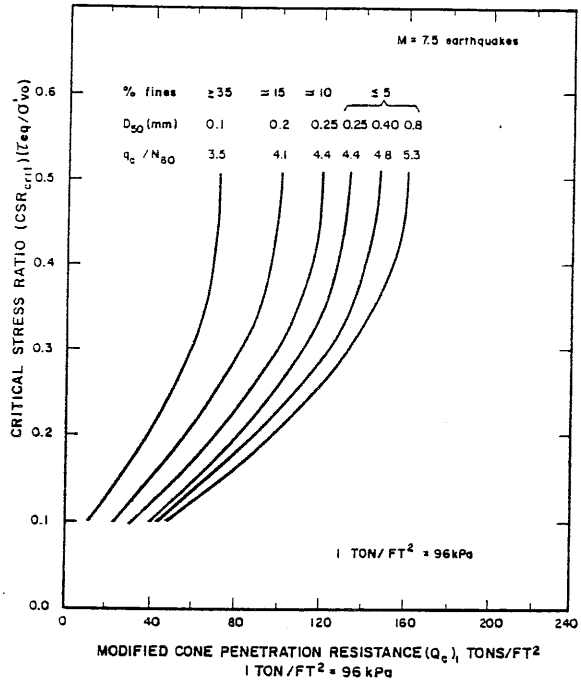



Fig.9 RELATIONSHIP BETWEEN CYCLIC RATIO CAUSING LIQUEFACTION AND Qcl FOR SANDS AND SILTY SANDS (after Seed and deAlba, 1986)

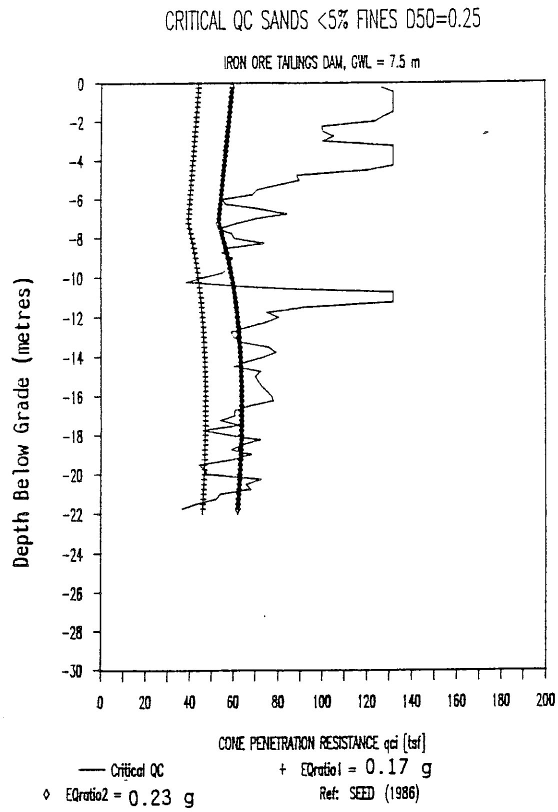



Fig.10 DEPENDENCE OF Qcl ON EARTHQUAKE MAGNITUDE AND PEAK GROUND ACC. (adapted from Seed, 1986)

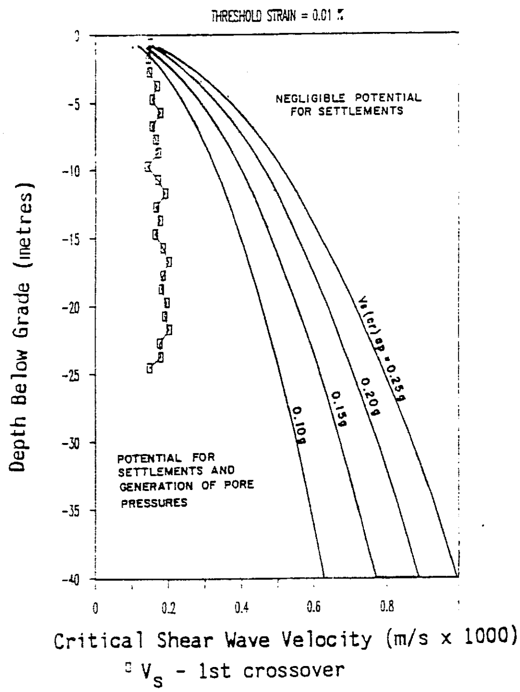



Fig. 11 THRESHOLD SHEAR WAVE VELOCITIES COMPARED WITH IN SITU SHEAR WAVE VELOCITIES

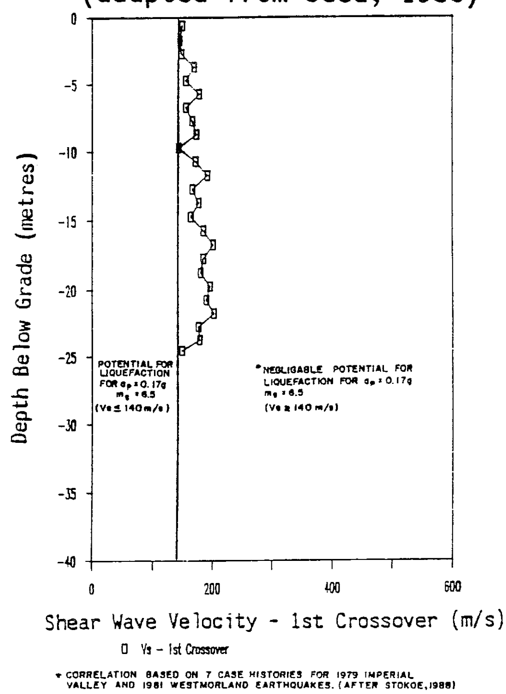



Fig. 12 SHEAR WAVE VELOCITY CRITERION COMPARED WITH IN SITU VELOCITIES

The shear wave velocity criterion developed by Stokoe et.al. (1984) is based upon an empirical assessment of what the minimum, or critical, skeletal shear wave velocity a soil must possess to preclude full liquefaction. The data base upon which this direct relationship between shear wave velocity and liquefaction potential is based is somewhat limited for large magnitude earthquakes. For smaller earthquakes with accelerations of up to 0.17 g and magnitudes to 6.5 the relationship is becoming reasonably well defined (Stokoe and Nazarian, 1985; Holzer et.al., 1988). Based on liquefaction events with peak surface accelerations of 0.17 g and magnitudes of about 6.5 which occurred during the last 10 years in areas such as the Imperial Valley of California, the limiting criterion for liquefaction is $V_s$ = 140 m/s. An example of the shear wave velocity criterion is shown in Figure 12 for shear wave velocities measured in a tailings dam. If the measured shear wave velocities are less than 140 m/s liquefaction is possible for peak surface accelerations of 0.17 g or greater.

## GROUNDWATER - FLOW AND CONTAMINATION

An important criteria in any stability or seepage analyses of mine waste is the groundwater flow regime. The addition of pore pressure measurements during the SCPT enables the flow regime to be determined in an extremely cost effective manner. During pauses in the cone penetration the equilibrium pore pressure and permeability parameter estimates can be determined. Fortunately most fine mine wastes consist of silty and sandy soils, hence the time required to reach equilibrium pore pressure is quite short, typically less than 30 mins. After a series of SCPT's a complete picture of equilibrium pore pressures can be compiled to determine the overall flow regime (Fig. 14). In finer grained tailings, the dissipation records can be used to predict both permeability and compressibility parameters for seepage and deformation analyses.

A critical criteria for liquefaction to occur is that the mine waste must be saturated. The measurements of dynamic and dissipated pore pressure combined with the compressional wave velocity provide a positive measure of degree of saturation.

A recent development in cone penetration test technology has been the addition of the ability to make chemical measurements on the groundwater. Water quality can now be determined using new cone penetration test equipment. A major area of concern for many mine waste deposits is the nature of groundwater seeping out from or beneath the tailings impoundment. New chemi-cones (Fig. 15) have the ability to determine the conductivity, pH, salinity and temperature of the groundwater. Future developments will include the ability to determine ion specific measurements. The ability to determine groundwater chemistry and soil parameters in one operation on a near continuous basis allows for the accurate profiling of contaminated groundwater plumes as well as some estimate of the rate and direction of flow through the tailings and underlying soils. Identification of the lateral and vertical extent of contaminants enables the engineer to rapidly implement a remedial works or recovery program thereby mitigating the potential damage caused by contaminated groundwater seepage.

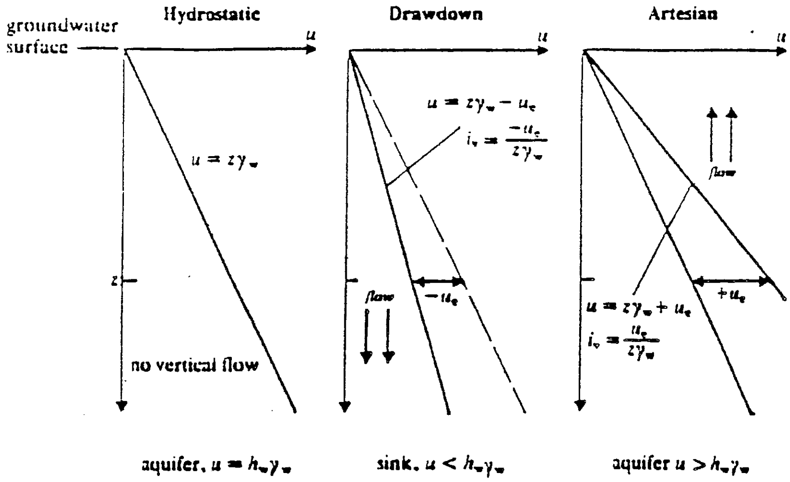



Fig. 14 IDEALIZED VERTICAL FLOW GRADIENTS (R.J. Mitchell, 1983)

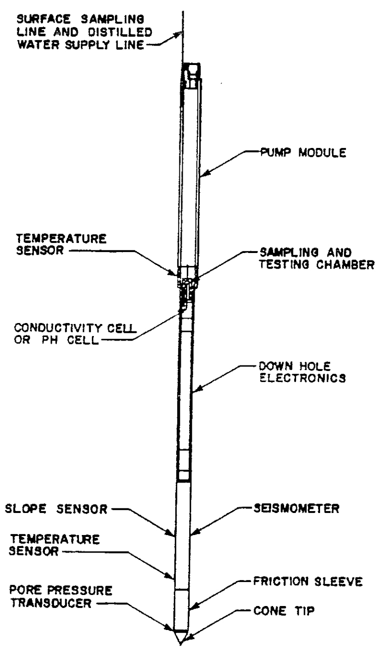



Fig. 15 CHEMI-CONE PENETROMETER

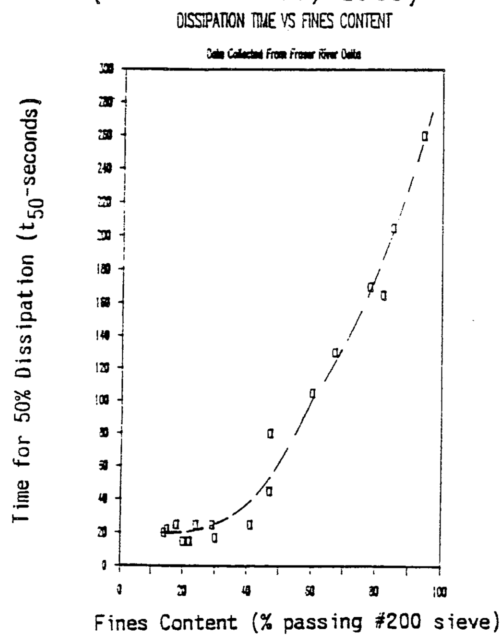



Fig.16 PROPOSED CORRELATION FOR ESTIMATION OF FINES CONTENT BASED ON TIME FOR 50% DISSIPATION (Woeller, 1988)

For PI $\leq 11$ and piezometer element located 5 mm behind the shoulder of the cone tip.

## ESTIMATION OF FINES CONTENT FROM CPTU DATA

The fines content of sandy soils is extremely important when evaluating liquefaction potential (Seed et.al, 1986). For cohesionless soils (PI< 11), it has been recognized that the cone penetration resistance, $Q_c$; decreases with increasing fines content (Fig. 9). Therefore, when $Q_c$ is used for the assessment of liquefaction potential it is essential to adjust for fines content in order to obtain the correct result. Figure 14 shows a proposed correlation for the estimation of fines content based on the time for 50% dissipation of excess pore pressure recorded using the cone pore pressure transducer. More information is required to further develop this proposed correlation. It would seem reasonable that a better developed correlation would include a set of curves with each curve corresponding to a specific range of $D_{50}$. An estimate of fines content is also useful as an additional tool for estimating soil permeability.

## CONCLUSIONS

Recent advances in cone penetration test (CPT) technology have significantly improved our ability as geotechnical engineers to determine the composition, strength and stiffness of most mine tailings. The addition of seismic wave velocity measurements to the CPT has added a new dimension to the application and interpretation of geotechnical properties of mine waste.

The combination of the seismic downhole method and the CPT provides an extremely rapid, reliable, and economic means of determining stratigraphy, strength and modulus information in one sounding.

The measurement of penetration and equilibrium pore pressures during the CPT has also improved the interpretation, especially for liquefaction analyses, and has enabled a rapid evaluation of steady state seepage conditions within tailings impoundments.

Water quality can now be measured using new CPT equipment. Hence, groundwater and soil parameters can be determined in one reliable and comparably economical operation on a near continuous basis.

The use of these recent developments in CPT technology have been briefly described in this paper. Continued advances in situ testing techniques can be expected and, when combined with these recent developments in the CPT, should even further improve our ability to accurately determine key strength, groundwater and geochemical design parameters of mine waste.

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