Cone penetration tests at active earthquake sites

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ABSTRACT: In-situ test results from a geothermally / seismic active area were evaluated to identify the applicability of empirical correlations available to estimate shear wave velocities and cyclic resistance ratios. The site investigation program comprised piezocone penetration test (CPTU), seismic cone penetration test (SCPT), temperature cone penetration test (TCPT) and soil borings. The temperature at the seabed is around 40 degrees Celsius with a temperature gradient of 5 degrees Celsius per meter depth. This paper presents CPTU data, measured and computed shear wave velocities, cyclic stress ratios and soil temperature measurements.

1 INTRODUCTION

Soil investigations were performed at an offshore geothermally / seismic active area. The surrounding area of this site comprises Miocene-Pleistocene volcanic units. The soil investigation program for the specific site comprised piezocone (CPTU), seismic cone (SCPT) and temperature cone penetrations tests (TCPT) and soil sampling borings. The in-situ tests and soil borings were terminated when weathered rock materials were encountered. Routine laboratory tests were performed offshore on the recovered soil samples comprising moisture content and unit weight. The majority of the soil comprises sandy silt/clayey silt. Tests performed at onshore laboratories include classification tests, cyclic triaxial tests, resonant column tests and direct shear tests. The CPTU data were evaluated to derive various soil parameters.

2 SOIL INVESTIGATION PROCEDURES

The soil investigation was carried out from the purpose built Fugro geotechnical vessel *Mariner*. Wireline Push (WIP) Samplers were used to obtain undisturbed samples from the soil borings. The cones were hydraulically pushed into the soil with a maximum 3m stroke at a constant rate of 20mm/s from each test depth. At the end of each 3m stroke or where premature refusal was encountered, the borehole was drilled and advanced to the next test depth. A piezocone with 60^o cone and 10 cm^2 base area with u_2 type pore pressure element was used. For seismic measurements, geophones are incorporated into the cone to detect the shear waves that are generated at the seabed with a hammer blow on a block. After performing each seismic test, the piezocone was pushed to the next depth that is generally 1.0m deeper. The temperature cone comprises a temperature measurement device excluding the pore pressure elements. The water depth at investigation site varied between 13.3m and 65.9m.

3 PIEZOCONE PENETROMETER TESTS

The basic measurements cone resistance (q_c), sleeve friction (f_s) and porewater pressure (u) were recorded from the CPTU. Figure 1 presents family plots of corrected cone resistance, friction ratio (Fr) and pore pressure ratio (Bq) values. Figure 2 presents a typical CPTU results profile.

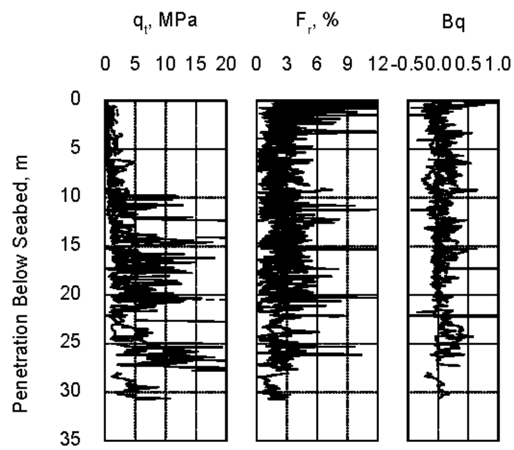

Figure 1 Family Plots of CPTU Test Results

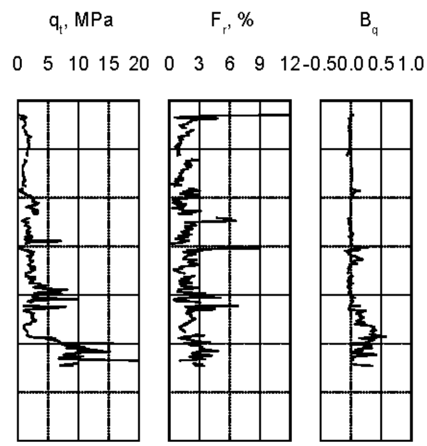

Figure 2 CPTU Test Results of Location 1

4 SEISMIC CONE PENETROMETER TESTS

The SCPT were performed to measure the shear wave velocities (V_s) that are required to compute the shear modulus (G). A shear wave is generated by means of a hammer blow and simultaneously the seismograph is triggered and subsequently the seismic wave arrivals at the geophone array are recorded. Figure 3 presents a measured shear wave velocity profile from SCPTresults.

The shear wave velocity profile and the cone resistance profile have some similarities. Therefore they were plotted against each other to evaluate the relationship. This plot is presented in Figure 4.

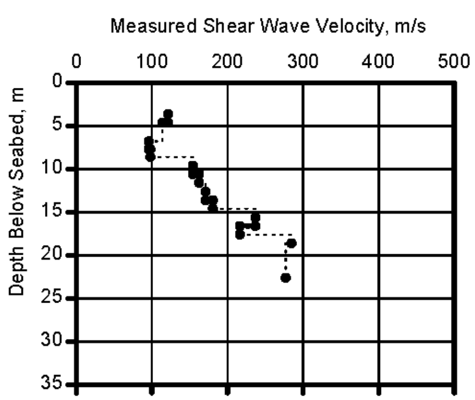

Figure 3 SCPT Test Results of Location 1

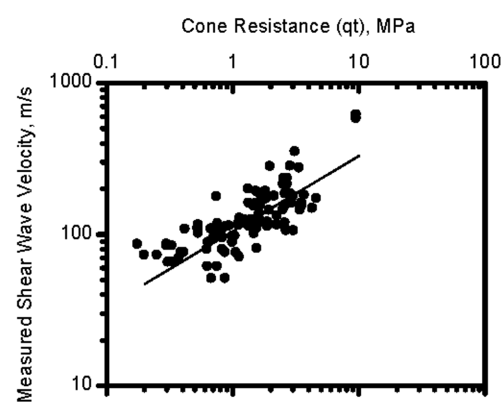

Figure 4 Shear Wave Velocity vs Cone Resistance

Figure 4 shows shear wave velocity increase linearly with increase of cone resistance. The relationship of these two parameters was further examined using the methods proposed by Hegazy & Mayne (1995) and Mayne & Rix (1993) as follows:

4.1 *Hegazy & Mayne (1995) Method*

The effects of four independent parameters q_c, f_s, void ratio (e_o) and $\sigma_{vo'}$ collected from different sites has been correlated with V_s. Simple regression analysis has revealed the following:

Clay: $V_s = 14.13 q_t^{0.359} e_o^{-0.473}$ (1)

Sand: $V_s = 13.18 q_t^{0.192} \sigma_{vo'}^{0.179}$ (2)

For all Types of Soils:

$V_s = (10.1 \mathrm{Log} q_t - 11.4)^{1.67} (f_s / q_t \times 100)^{0.3}$ (3)

where V_s is in m/s and q_t ,$\sigma_{vo'}$ and f_s are in kPa.

Since e_o and $\sigma_{vo'}$ are not possible to obtain without laboratory test results, another set of equations has been developed as follows:

Clay: $V_s = 3.18 q_t^{0.549} f_s^{0.025}$ (4)

Sand: $V_s = 12.02 q_t^{0.319} f_s^{-0.0466}$ (5)

where V_s is in m/s and q_t and f_s are in kPa.

4.2 *Mayne & Rix (1993) Method*

The effects of three independent parameters q_c, f_s, and e_o collected from different sites comprising clay soils has been correlated with Vs. Simple regression analysis has revealed the following:

Clay: $V_s = 9.44 q_t^{0.435} e_o^{-0.532}$ (6)

where V_s is in m/s and q_t is in kPa.

Since e_o is likely not to be known beforehand while conducting in-situ tests with CPTs, the following equation has been developed independent of e_o values.

Clay: $V_s = 1.75 q_t^{0.627}$ (7)

where V_s is in m/s and q_t is in kPa.

4.3 *Case Studies*

CPTU results and SCPT results available from the geothermally/seismic active site were examined. Shear wave velocities were computed from the CPTU results based on the equations (3), (4) and (5) proposed by Hegazy & Mayne (1995) and the equation (7) proposed by Mayne & Rix (1993). The computed shear wave velocities were plotted against the measured shear wave velocities from seismic cone test results and are presented in Figure 5.

A number of Soil classification charts based on the CPTU Results are available in the literature. These classification charts presents the soil behavior type. Robertson (1990) suggests these charts are still global in nature and should be used as a guide to define soil behavior type based on CPTU data. The boundaries between the soil behavior type zones in the Robertson (1990) Charts are approximated as concentric circles, and the radius of each circle is defined as soil behavior type index (I_c). The soil behavior type index can be computed using the following Equation 8.

$$I_c = ((3.47 - LogQ_t)^2 + (LogF_r + 1.22)^2)^{0.5} \qquad (8)$$

Where

$$Q_t = \frac{(q_t - \sigma_v)/\sigma_{atm}}{(\sigma_v'/\sigma_{atm})^n} \qquad (9)$$

$$F_R = f_s/(q_t - \sigma_v) \times 100 \qquad (10)$$

where
σ_{atm} = 1 bar = 100 kPa and
q_t ,σ_v, σ_v' and f_s are in kPa.

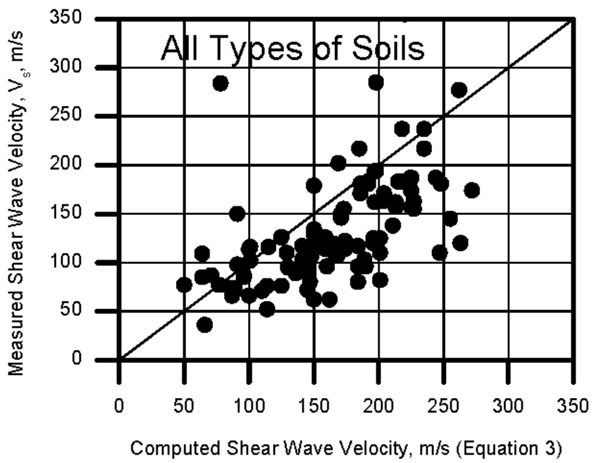

Figure 5a. Measured & Computed V_s
(Eqn. 3) $V_s = (10.1x\text{Log } q_t - 11.4)^{1.67} x (f_s/q_t\ x100)^{0.3}$

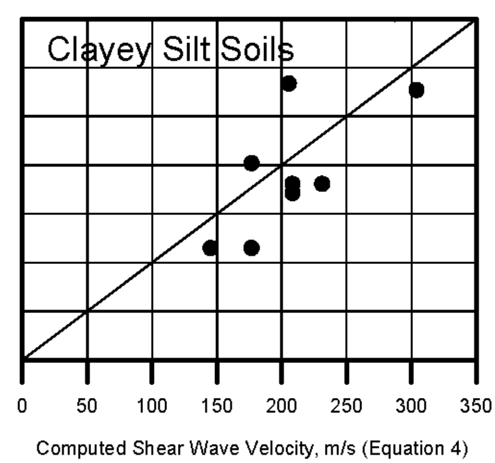

Figure 5b Measured & Computed V_s
(Eqn. 4) $V_s = 3.18xq_t^{0.549}xf_s^{0.025}$

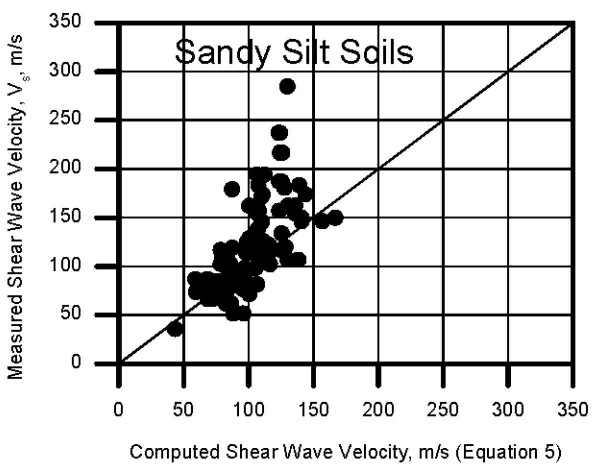

Figure 5c Measured & Computed V_s
(Eqn. 5) $V_s = 12.02xq_t^{0.319}xf_s^{-0.0466}$

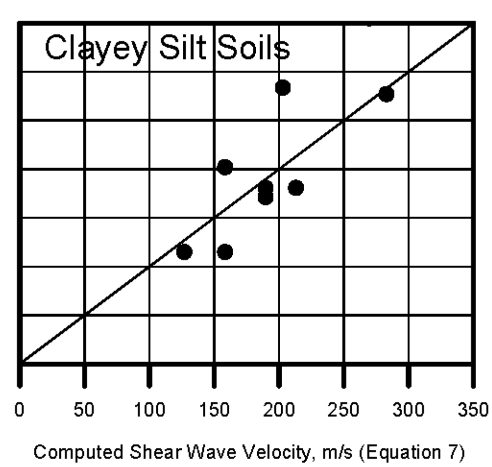

Figure 5d Measured & Computed V_s
(Eqn. 7) $V_s = 1.75xq_t^{0.627}$

For the computation of I_c, Robertson and Wride (1998) suggests an iterative method. Assume n=1 and compute I_c and if the computed I_c is less than 1.64, use n=0.5. If the computed I_c is greater than 3.3, use n=1. If the computed I_c is between 1.64 and 3.3, then re-compute value of n using the Equation 11 given below:

$$n = (I_c - 1.64) \times 3 + 0.5 \quad (11)$$

The computed I_c values are presented in Figure 6.

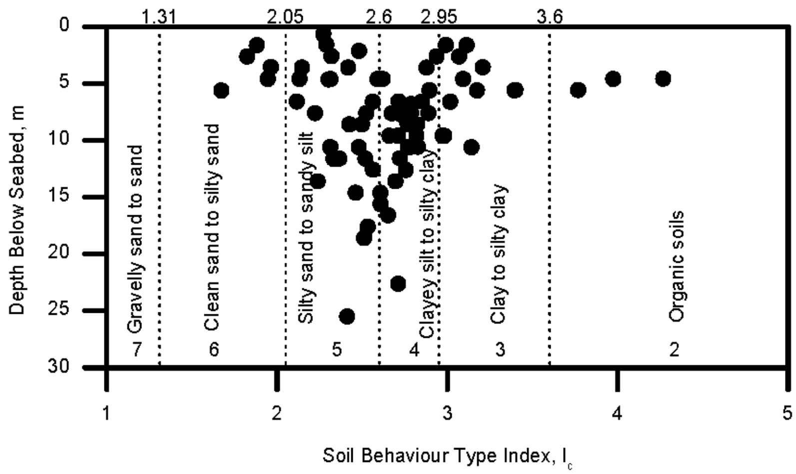

Figure 6. Computed Soil Behavior Index Values

The soil classified under different zones in Figure 6, is similar to soil classification zones described by Robertson (1990) Charts. The majority of the soil is being classified as silty sand / sandy silt and clayey silt / silty clay. This classification is very well comparable with the laboratory classification test results and visual observations made from the adjoining soil borehole samples.

The soil behavior type index (I_c) versus V_s/q_t values were plotted and presented in Figure 7. The measured shear velocities (V_s) and the cone resistance (q_t) values presented in Figure 4 were used to compute V_s/q_t values.

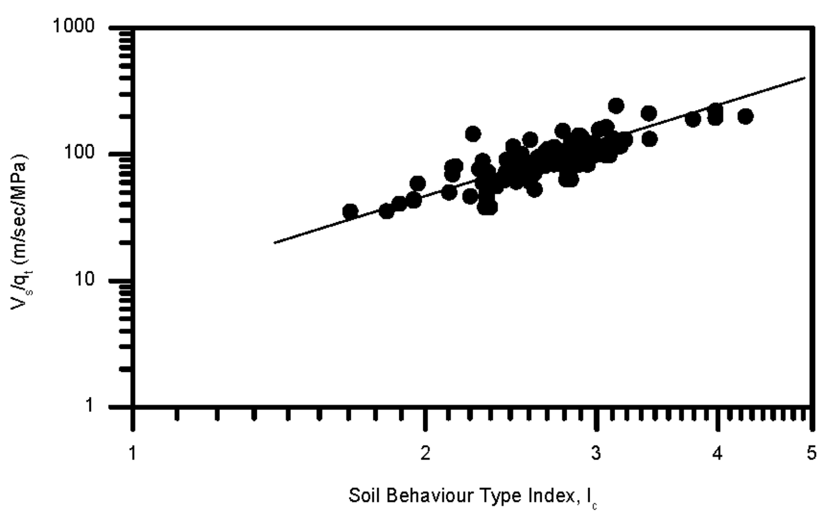

Figure 7. V_s/q_t versus Soil Behavior Type Index

Figure 7 shows shear wave velocities increase linearly with the increase of cone resistance, indicating a relationship between the shear wave velocity and the cone resistance.

5 EVALUATION OF CYCLIC STRESS RATIOS

Loose saturated sands when subjected to strains or shocks tend to reduce in volume. As a result the pore pressure may increase while decreasing the effective stress within the soil mass. When the pore pressure is equal to effective stress, the sand is said to be in the state of liquefaction. Cyclic loads such as earthquakes can build pore pressures in saturated sands leading to possible liquefaction.

Cyclic strength has been defined as the cyclic stress ratio (CSR) required for the developed pore water pressure to become equal to the initial confining pressure under a certain number of shear stress application. CSR is computed by normalizing the cyclic shear stress by the initial effective vertical stress. A simplified method to estimate CSR from cyclic triaxial tests results was developed by Seed and Idriss (1971) with the Equation 12 given below:

$$CSR = \tau_{av}/\sigma_{vo}^{'} = 0.65(a_{max}/g)(\sigma_{vo}/\sigma_{vo}^{'})r_d \quad (12)$$

where

τ_{av} = average cyclic shear stress, in kPa

$\sigma_{vo}^{'}$ = effective vertical overburden stress, kPa

a_{max} = maximum horizontal acceleration in m/s^2

g = 9.81 m/s^2 acceleration due to gravity

σ_{vo} = total vertical overburden stress, in kPa

r_d = 1.0 – 0.00765z ; if z < 9.15m

r_d = 1.174-0.0267z ; if 9.15 < z < 23m

z = depth in meters

There are only 5 cyclic triaxial tests results available from the study site and they are all being carried out on soil samples obtained between 2.0 and 4.5m below seabed. The measured CSR values are in the range of 0.13 to 0.19.

The liquefaction resistance is defined as the cyclic resistance ratio (CRR) for an earthquake magnitude of 7.5 Richter scale. Potential liquefaction is then evaluated with the $CRR_{7.5}$/CSR ratio. In general terms, a factor of safety greater than one (FOS > 1) indicates that the liquefaction resistance exceeds the earthquake loading, and therefore liquefaction may not occur.

$$FOS = CRR_{7.5}/CSR \quad (13)$$

The $CRR_{7.5}$ can be computed based on the CPTU results using the following equations (after Robertson and Wride, 1998):

$$CRR_{7.5} = 0.833((q_{c1N})_{cs}/1000) + 0.05 \quad ;\text{if } (q_{c1N})_{cs} < 50 \quad (14)$$

$$CRR_{7.5} = 93((q_{c1N})_{cs}/1000)^3 + 0.08 \quad ;\text{if } 50 \leq (q_{c1N})_{cs} \leq 160 \quad (15)$$

where

$(q_{c1N})_{cs}$ = normalized cone resistance of clean sand

$$(q_{c1N})_{cs} = K_c(q_{c1N}) \quad (16)$$

where

K_c = correction factor

$(q_{c1N}) = Q_t$ = normalized cone resistance

$K_c = 1.0$; if $I_c \leq 1.64$
$K_c = (-0.403I_c^4 + 5.58I_c^3 - 21.63I_c^2 + 33.75I_c - 17.88)$; if $I_c > 1.64$
q_{c1N} that is equal to Q_t, is estimated using the Equation 9.

The computed $CRR_{7.5}$ values from CPTU results are presented in Figure 8 along with the CSR values computed from the Cyclic Triaxial Test results.

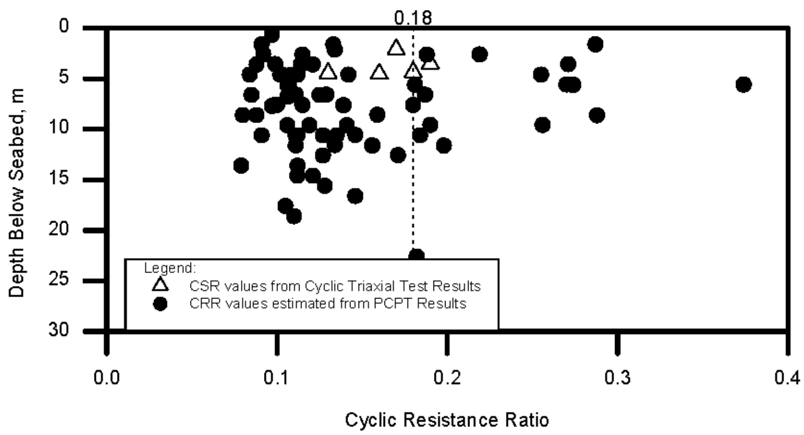

Figure 8. Cyclic Resistance Ratio Values from PCPT Results

The majority of the estimated $CRR_{7.5}$ values are lower than 0.18.

6 TEMPERATURE CONE PENETROMETER TESTS

The soil temperature was measured at 5 borehole locations using temperature cone penetrometers. Due to presence of geothermal activity at site the seabed temperature was around 40^{o}c and steep temperature gradients were encountered below ground. The highest measured temperature at this site is about 129^{o}c at 24m below seabed. The temperature of the soil samples from adjoining boreholes were measured using thermometers when the soil sample tubes were brought over to the drill floor from the borehole. Temperature measurements from CPTs were comparable with the manually measured temperatures. It indicates that the CPT cones used at this site can withstand extraordinarily high temperatures and the measured data are reliable. The TCPT data comprise only soil temperature readings and the basic soil strength measurements since pore pressure readings are not available. Figure 9 presents the measured temperature profiles from the TCPT data.

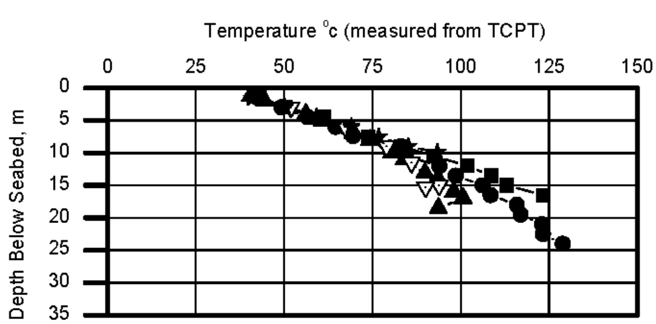

Figure 9 Temperature Profiles from TCPT Data

7 CONCLUSIONS

The downhole tools used for the in-situ tests were found to be reliable in sub soils where the temperature was very high. This paper illustrates various soil property computations with CPTU results. The summary of the computations are as follows:

a. Computation of shear wave velocities for granular and cohesive materials: The results are comparable when they were computed independently as in Equations 4, 5 & 7. The computed shear wave velocities according to Equation 3, that is applicable for All Types of Soil, indicate that some results are scattered.
b. Computation of soil behavior type index: The soil classifications based on laboratory test results and the computed soil behavior index values are comparable.
c. Computation of cyclic resistance ratio: The majority of the computed CRR values were within the range of 0.1 and 0.18. The CSR values computed from cyclic triaxial test results indicate they are in the range of 0.13 and 0.19.
   Further research is recommended to evaluate whether any temperature factor should be incorporated into Equations 14 and 15.
d. V_s/q_t versus CPT soil behavior type index (I_c) profile indicates a relationship between the shear wave velocity and the cone resistance.

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