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Thomas G. Thomann
Woodward-Clyde Consultants, Wayne, NJ

Roman D. Hryciw
University of Michigan, Ann Arbor, MI

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Thomas G. Thomann
Staff Engineer, Woodward-Clyde Consultants, Wayne, NJ

Roman D. Hryciw
Assistant Professor of Civil Engineering, University of Michigan, Ann Arbor, MI

SYNOPSIS: Results from seismic cone penetration and dilatometer tests performed in a cohesionless soil are presented and compared with previously published correlations for estimating the elastic shear modulus. Poor correlations were obtained between the elastic shear modulus, the dilatometer modulus and the cone tip resistance; however, somewhat better estimates of the elastic shear modulus were obtained using the coefficient of lateral stress and total unit weight empirically determined from the dilatometer.

INTRODUCTION

The elastic shear modulus (G^e) of cohesionless soils is primarily a function of the void ratio (e) and the effective confining stresses in the direction of particle motion (σ_i') and wave propagation (σ_j') - variables which can be determined from cone penetration (CPT) and dilatometer (DMT) tests. Therefore, empirical correlations between G^e and penetration tests may offer a viable alternative to costly laboratory or in-situ seismic tests when estimates of G^e are required.

Correlations between penetration tests and G^e have previously been developed. Jamiolkowski, et.al. (1988) related G^e to the DMT modulus, E_D, and Baldi, et.al. (1989) related G^e to cone tip resistance, q_c. Both approaches were based primarily on laboratory tests in sands. However, in-situ tests have often revealed the shortcomings of these correlations in predicting G^e (Sully and Campanella, 1989; Hryciw, 1990). Better predictions may be obtained when the coefficient of lateral stress (K_o) and the total unit weight (γ_t) from the DMT are used in an empirical correlation for G^e (Hryciw, 1990). The present paper compares the results from seismic cone penetration tests (SCPT) and DMT tests performed at a site in northern Michigan with the approaches suggested by Jamiolkowski, et.al. (1988), Baldi, et.al. (1989), and Hryciw (1990).

BACKGROUND

DMT calibration chamber tests, resonant column tests and in-situ cross hole tests performed in Ticino sand and Po River sand by Jamiolkowski, et.al. (1988) indicate that the ratio of G^e to E_D, commonly termed R_g, decreases with increasing relative density. Baldi, et.al. (1989) developed a correlation between G^e/q_c and $q_c/(\sigma_{vo}')^{0.5}$ based on CPT calibration chamber tests and resonant column and cross hole tests.

Empirical correlations for determining G^e of cohesionless soils from the void ratio and confining stress have existed for nearly three decades. The original equation proposed by Hardin and Richart (1963), has undergone minor modifications since its inception. The most recent version by Hardin and Blandford (1989) takes the form :

$$G^e{}_{ij} = \frac{\mathrm{OCR}^k}{F(e)} \; \frac{S_{ij}\, P_a{}^{(1-n)}}{2(1+\nu)} \; (\sigma_i'\sigma_j')^{n/2} \qquad (1)$$

where OCR (overconsolidation ratio) is the ratio of the maximum historical mean effective stress to the current mean effective principal stress, k is a function of the plasticity index (PI) (k = 0 for cohesionless soils), S_{ij} is the stiffness coefficient in the ij plane, P_a is the atmospheric pressure, $F(e)$ is a void ratio function = $0.3 + 0.7(e)^2$, and ν is the Poisson's ratio. In seismic down hole testing, σ_i' is the effective horizontal stress (σ_h') and σ_j' is the effective vertical stress (σ_v'). Hardin and Blandford (1989) suggest using S_{ij} = 1400, n = 0.5, and ν = 0.1. The obvious disadvantage of equation (1) is that not only must the void ratio be known but the vertical as well as the horizontal stresses must be evaluated.

Recognizing that the DMT offered the ability to determine the soil density as well as the horizontal stress, Hryciw (1990) developed an empirical equation similar to equation (1).

The model is based on results from nine different test programs where DMT and in-situ shear wave measurements were performed. The soils ranged from clays with dilatometer indices (I_D) of 0.1 to sands with I_D's as high as 8. The proposed equation takes the form:

$$G^e = \frac{530}{(\sigma_v'/p_a)^{0.25}} \; \frac{(\gamma_t/\gamma_w)-1}{2.7-(\gamma_t/\gamma_w)} \; K_o^{0.25}(\sigma_v' p_a)^{0.5} \qquad (2)$$

where γ_t = DMT-based total unit weight and γ_w = unit weight of water.

Hryciw (1990) also showed that when a range of soil types is considered, very little correlation exists between R_g and the horizontal stress index (K_D) of the DMT. Hryciw postulated that the poor correlations may be due to differences in the strain level when measuring G^e and E_D.

SITE CHARACTERISTICS

The in-situ tests were performed at the University of Michigan Biological Station in Pellston, Michigan. The soil conditions consisted of a medium to fine, subangular, light brown sand. The grain size distribution is shown in Figure 1. The Unified Soil Classification was SP - poorly graded sand. The specific gravity (G_s) was 2.65 and the maximum and minimum void ratios (e_{max} and e_{min}) were 0.82 and 0.54, respectively.

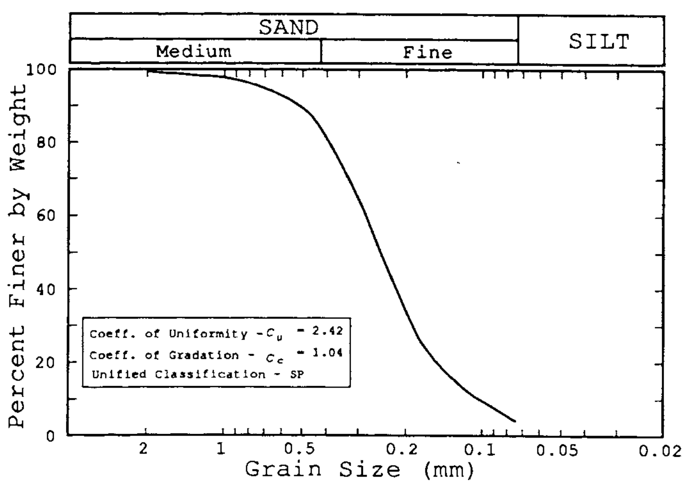

Figure 1. Grain Size Distribution of Douglas Lake Sand.

The grain size distribution of the soil, as observed from several augered holes, was consistent to a depth of 9.1 m. One CPT test outside the study area indicated similar soil to a depth of 21.3 m. The water table was never encountered during augering or penetration testing; however, the moisture content was approximately 4%.

Two sets of SCPT and DMT tests were performed at a distance of 3.35 m apart. At each location, the SCPT and DMT tests were performed approximately 0.61 m away from each other. The q_c, E_D, and DMT horizontal stress index (K_D) from the two sets of tests are shown in Figure 2. The friction ratios from the CPT tests were generally less than 1%. I_D ranged from 2 to 4. Complete results of the SCPT and DMT tests are given by Thomann (1990).

DETERMINATION OF K_O AND D_R

Comparisons between the results of this study and the previously mentioned correlations require determination of the relative density, dry unit weight, and confining stresses. These values were determined by empirical correlations developed for the SCPT and DMT.

Marchetti and Crapps (1981) found that the density increased as E_D increased. They developed an empirical chart relating E_D and I_D to the soil type and density. This chart was used for determining γ_D and σ_v' at the Douglas Lake site for use in equation (2).

Several relationships exist for determining K_o of cohesionless soils from results of the DMT test. Schmertmann (1983) developed a semi-empirical equation for estimating K_o based on K_D and the thrust necessary to drive the DMT blade. Extensive calibration chamber tests performed by Baldi, et.al. (1986) found that K_o for natural, predominantly quartz, uncemented sand was best predicted by using :

$$K_o = 0.376 + 0.095\,K_D - 0.00461\,q_c/\sigma_v' \qquad (3)$$

The K_o values at the Douglas Lake site, as determined by equation (3), are shown in Figure 3.

Results from 228 calibration chamber tests performed by Jamiolkowski, et.al. (1988) on both normally consolidated and overconsolidated Ticino Sand samples revealed a strong relationship between D_r, q_c, and the mean effective confining stress (σ_o'):

$$D_r = \frac{1}{2.93}\ \ln\left(\frac{q_c}{205(\sigma_o')^{0.51}}\right) \qquad (4)$$

The relative density at the Douglas Lake site is shown in Figure 3. The results of equations (3) and (4) were used to estimate G^e via

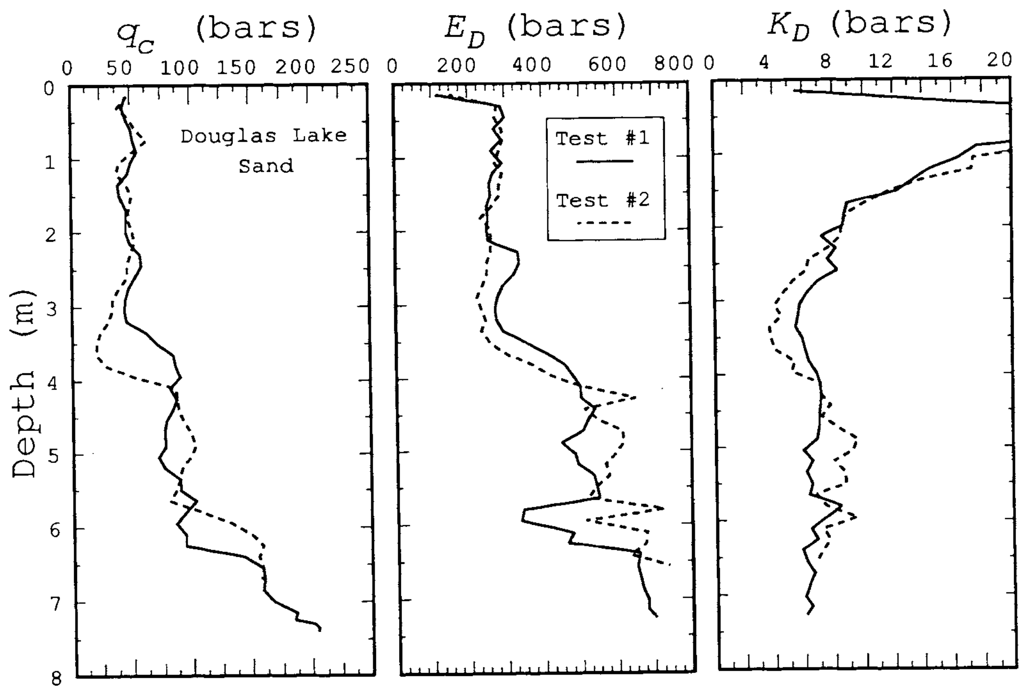

Figure 2. q_c, E_D, and K_D from SCPT and DMT tests in Douglas Lake Sand.

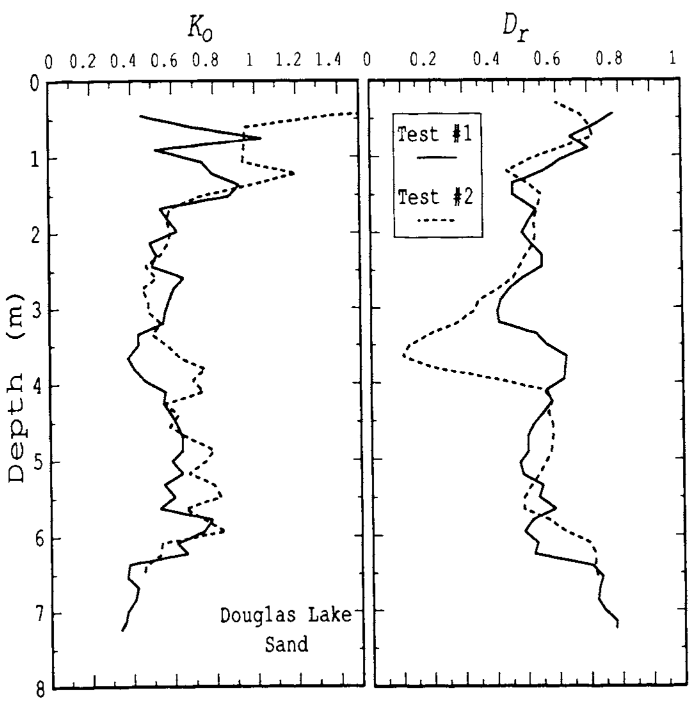

Figure 3. K_o and D_r from equations (3) and (4) in Douglas Lake Sand.

equation (1). The empirical total unit weights from the DMT were not used for determining D_r because they were found to be unreasonable.

G^e CORRELATIONS FROM DMT AND SCPT RESULTS

Essentially no correlation was found between R_g and D_r, as shown in Figure 4. This figure also contains results from Sully and Campanella (1989) which also do not coincide with results of Jamiolkowski, et.al. (1988). Very little relationship was also found between the Douglas Lake results and the correlation proposed by Baldi, et.al. (1989) (Figure 5).

The poor correlations may be due to the difference in strain level between seismic and penetration tests. The strains associated with G^e are much lower than those imposed when determining E_D or measuring q_c.

Equations (1) and (2) yield somewhat better estimates of G^e (Figure 6). Although both equations predict similar values for G^e, equation (2) does not require CPT, e_{max} and e_{min} tests to be performed. Therefore, the DMT-based approach for determining G^e is more desirable.

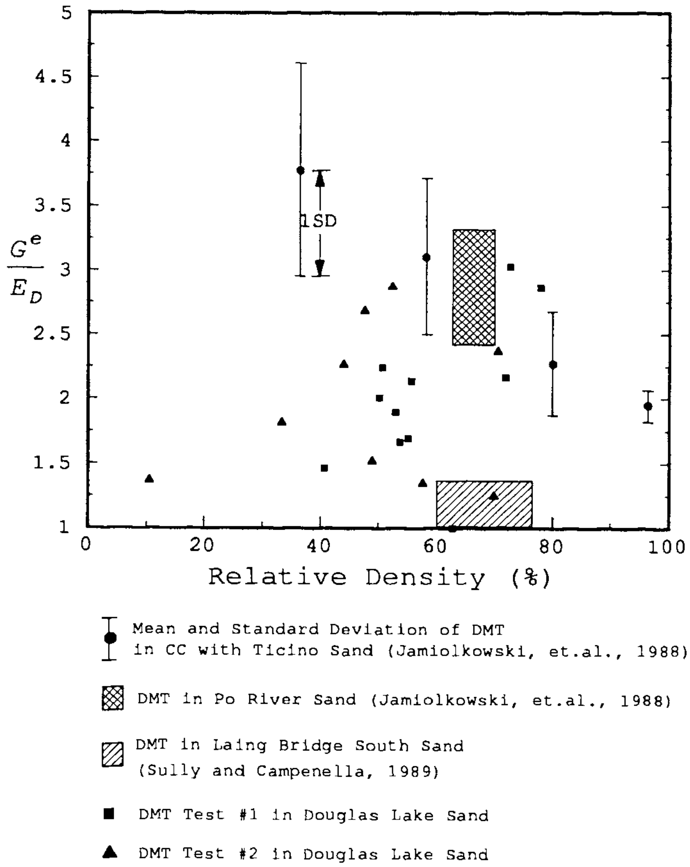

Figure 4. Correlation between R_g and D_r

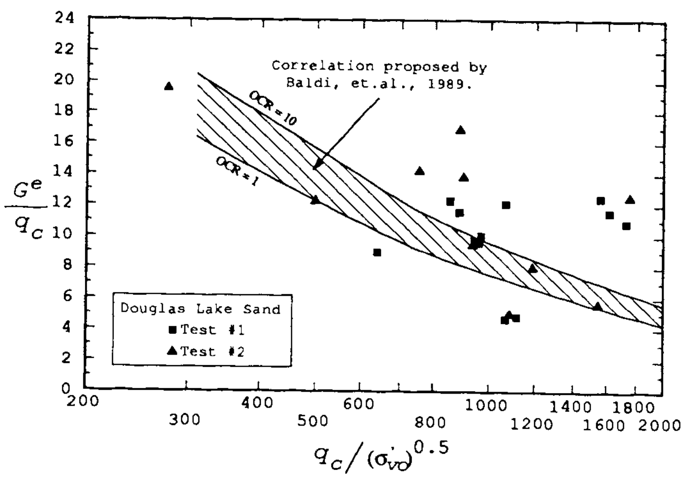

Figure 5. Correlation between G^e and q_c.

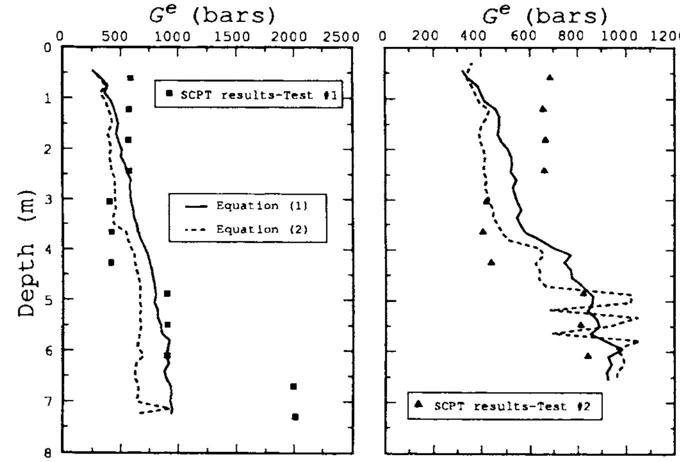

Figure 6. Estimated G^e by equations (1) and (2).

CONCLUSIONS

The results from SCPT and DMT tests performed in a cohesionless soil were compared with previously published correlations for estimating G^e. Poor correlations were obtained when equations relating G^e to E_D and q_c were used. The poor correlations may be due to differences in strain magnitude between seismic and penetration tests. The most accurate and efficient method for empirically determining G^e from penetration tests appears to be one that utilizes the DMT-based γ_t and K_o.

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