콘관입시험을 이용한 지반강도의 상세평가해석

Advanced Evaluation of In-situ Strength using CPT results

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SYNOPSIS : 콘관입시험(CPT)은 의사정적상태로 수행되는 현장시험방법으로서, 각종 기초구조물의 설계와 더불어 지반조사를 위한 대표적 방법으로 널리 적용되고 있다. 본 논문에서는 콘관입시험결과를 이용하여 사질토 지반에 있어서 지반강도의 상세평가법을 제안하고자 한다. 사질토의 강도는 상대밀도와 응력상태에 따라 변하는 상태의존적 성질을 나타내고 있으나, 이러한 역학적 성질은 실험실 내에서만 측정이 가능한 상태이며, 현장강도의 경험식이나, 대표강도의 평가만이 제안되어 있는 실정이다. 따라서 본 연구에서는 대표적 현장시험방법인 CPT를 이용하여 다이러턴시 특성 평가가 가능하며, 다양한 지반특성치가 반영될 수 있는 현장강도의 상세평가법을 제안하고자 한다. 이를 위해 실내삼축압축시험을 통해 얻어진 강도특성과 역학특성치들을 분석하였으며, 이를 토대로 수정 다일러턴시 평가법을 제안하였다. 제안된 방법의 검증을 위해 가압토조를 이용한 콘관입시험을 수행하였으며, 측정값과의 비교분석을 수행하였다.

Key words : strength, granular soils, cone penetration tests, dilatancy, friction angle

1. Introduction

Evaluation of strength is a challenging task for geotechnical engineers, primarily due to complex soil constitutions, non-homogeneity, and non-linearity of soil behavior. For sandy soils, the challenge is even greater, as strength is highly state-dependent and undisturbed soil sampling is not an economically and practically feasible option. As a result, various empirical correlations based on in-situ test results, such as SPT blow count N_{SPT} from the standard penetration test (SPT) or cone resistance q_c from the cone penetration test (CPT), have been proposed.

There have been several methods for the estimation of the shear strength in sands using CPT results, defining direct correlations between q_c and the peak friction angle $_p$. While these have provided useful tools for the interpretation of CPT measurements, further investigation is still necessary as no specific consideration of the state-dependent dilatancy and soil constitution were addressed in detail.

In the present study, methods to estimate the in-situ strength and dilatancy characteristics of sandy soils are proposed based on CPT results. A series of laboratory test results obtained for various soil conditions are used in the analysis and investigation. For each soil and stress condition, cone penetration analysis is performed and used to develop the CPT-based methodology for in-situ evaluation of dilatancy. Results from calibration chamber tests are used for verification.

2. Shear Strength and Dilatancy of Sandy Soils

The peak friction angle ϕ_p of sands is a stress- and density-dependent variable (Bolton 1986). The critical-state friction angle ϕ_c is on the other hand an intrinsic soil variable, independent of stress state, history, and density, and thus can be uniquely obtained even using completely disturbed samples. In order to quantify the dilatancy of sandy soils, Bolton (1986) proposed the following relationship based on experimental test results:

$$\phi'_p = \phi'_c + R_D \cdot I_R \tag{1}$$

where R_D = dilatancy ratio = 3 and 5 for triaxial and plane-strain conditions, respectively. The dilatancy index I_R is given by:

$$I_R = I_D\left[Q - \ln\left(\frac{100\sigma'_{mp}}{p_A}\right)\right] - R \tag{2}$$

where I_D = relative density as a number between 0 and 1 p_A = reference stress = 100 kPa σ_{mp}= mean effective stress at peak strength (in the same units as p_A) and Q and R = intrinsic soil variables. According to Bolton (1986), values of Q and R are equal to 10 and 1 for clean quartz sands, respectively. As Bolton's dilatancy relationship of Eqs. (1) and (2) reflects effects of both relative density and confining stress, it has been widely used and adopted for strength evaluation of sands experimentally and analytically.

As indicated by Eq. (2), state variables that control the dilatancy of sands are D_R and σ_{mp}. D_R is a state soil variable that is uniquely defined for given soil conditions. As σ_{mp} represents the mobilized mean effective stress at peak, it depends on a number of factors, including initial vertical and horizontal effective stresses, D_R, and other state and intrinsic soil variables. For laboratory test conditions, where stress and soil conditions are known for a given confining stress $_c$, $_{mp}$ can be easily determined. Field evaluation of $_{mp}$, however,is difficult due to unknown stress states mobilized upon loading. For this reason, Eqs. (1) and (2) have not been fully applied for field evaluation of strength.

3. Experiments

In order to evaluate strength of sandy soils, triaxial (TX) test results from Salgado et al. (2000) and Lee et al. (2004) were adopted. Test soils in both cases were Ottawa sand containing different amount of non-plastic silts in 0 to 20% range by weight. Other detailed test procedure and properties of Ottawa sand can be found in Salgado et al. (2000) and Lee et al. (2004). Additional triaxial and fundamental property tests were performedin this study using Jumunjin sand. Relative densities in range of 45 – 90% and confining stresses in range of 50 – 400 kPa were considered in triaxial tests to characterize state-dependent shear strength of Jumunjin sand.

Calibration chamber CPTs were also performed for characterizing CPT-stregnth correlation. The calibration chamber used in this study was made of steel and had a diameter and height equal to 77.5 and 125 cm, respectively. Inside the chamber, two rubber membranes were attached on the bottom and lateral sides. Through these membranes, compressed air pressure was supplied for achieving a desired stress state of the calibration chamber specimen. Fig. 1 shows details of the calibration chamber and cone penetrometer used in this study.

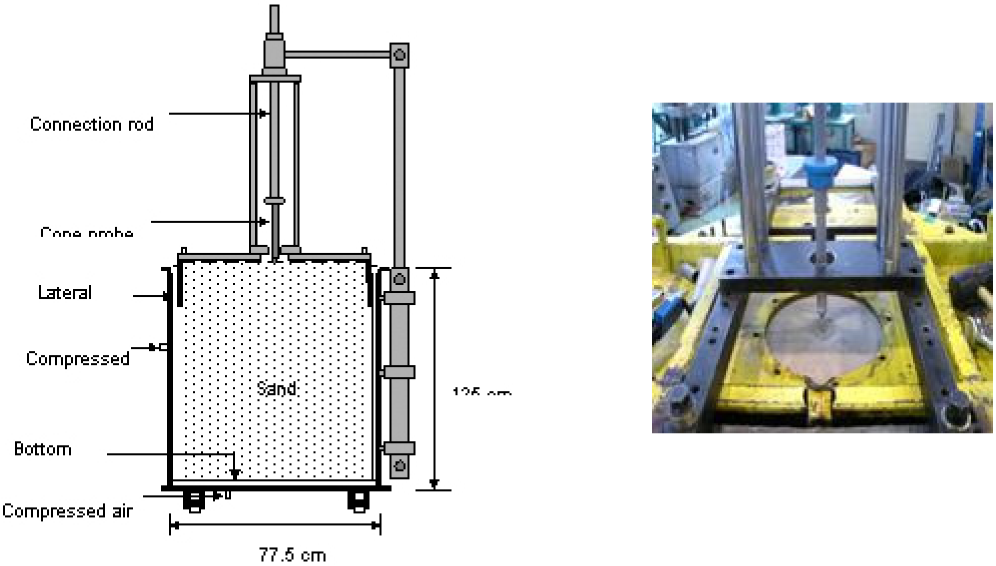


Fig. 1. Calibration chamber cone penetration tests

4. Modified Dilatancy Model

The confining stress in TX tests is routinely determined from in-situ vertical and horizontal stresses at a certain target depth, typically given as the mean effective stress equal to $(\sigma_{v0}+2\sigma_{h0})/3$. Based on this procedure, a series of in-situ stresses, equivalent to σ_c adopted in TX tests, were obtained. For each in-situ stress state and D_R, values of q_c were obtained from the cone penetration analysis using the program CONPOINT, which has been widely examined and validated (Salgado and Randolph 2001).

Fig. 2(a) shows relationships between the mean effective stress at peak σ_{mp}, measured from TX tests, and the cone resistance q_c obtained from CONPOINT. As can be seen in Fig. 2(a), the correlation appears to be fairly unique for all the soil conditions considered in this study. Correlations in Fig. 2(a) can be given by

$$\ln\frac{\sigma'_{mp}}{\sigma'_{h0}} = \alpha \cdot \left(\ln\frac{q_c}{\sigma'_{h0}} \right)^{\beta} \tag{3}$$

where σ'_{mp} = peak mean effective stress; σ'_{h0} = in-situ horizontal effective stress; and α and β = correlation parameters given in Fig. 2(b). Based on results in Fig. 6 I_R given by Eq. (2) can be rewritten as:

$$I_{R,CPT} = I_D\left[Q - \alpha \cdot \left(\ln\frac{q_c}{\sigma'_{h0}} \right)^{\beta} - \ln\left(\frac{100\sigma'_{h0}}{p_A} \right) \right] - R \tag{4}$$

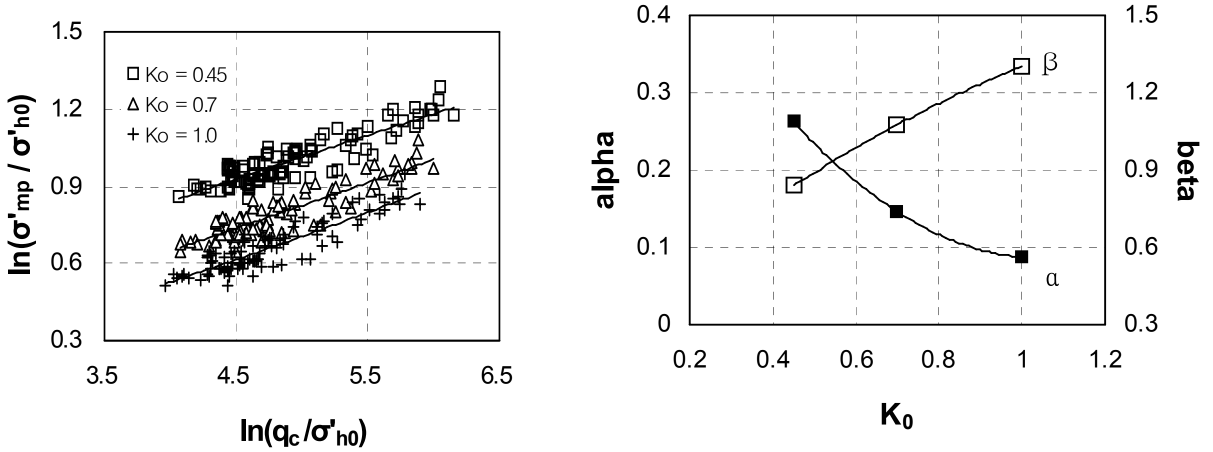


Fig. 2 Modified dilatacy relationship (a) σ'_{mp} versus q_c correlaion and (b) vaues of α and β

Both q_c and ${}_{mp}$ are primarily governed by D_R and the confining stress. Based on the assumption of similar dependency of q_c and ${}_{mp}$ on D_R and ${}_{h0}$, the Bolton's dilatancy index and ${}_{p}$ equations of Eqs. (2) and (1) may further be modified as a sole function of q_c as follows:

$$I_{R,CPT*} = I_D\left[Q_{CPT} - \ln\left(\frac{100q_c}{p_A}\right)\right] - R_{CPT} \tag{5}$$

where $I_{R,CPT*}$= simplified dilatacy index in terms of q_c Q_{CPT} and R_{CPT}= intrinsic soil variables that are analogous to Q and R in Eq. (2). In Eq. (9), values of Q_{CPT} and R_{CPT} are different from those of Q and R in Eq. (2), due to numerical differences between q_c and σ'_{mp}. In order to obtain values of Q_{CPT} and R_{CPT}, a regression analysis was performed using TX test results. Fig. 3 shows values of Q, R, Q_{CPT} and R_{CPT}, as a function of silt content, obtained from the regression analysis. From Fig. 3, values of Q_{CPT} in case s_{co} = 0, 2, 5, 10, 15 and 20% cases were found to be 14.0, 15.4, 14.0, 14.3, 11.8 and 12.1, while values of R_{CPT} were 1.0, -0.12, -0.12, -0.01, 0.01 and 0.12 respectively.

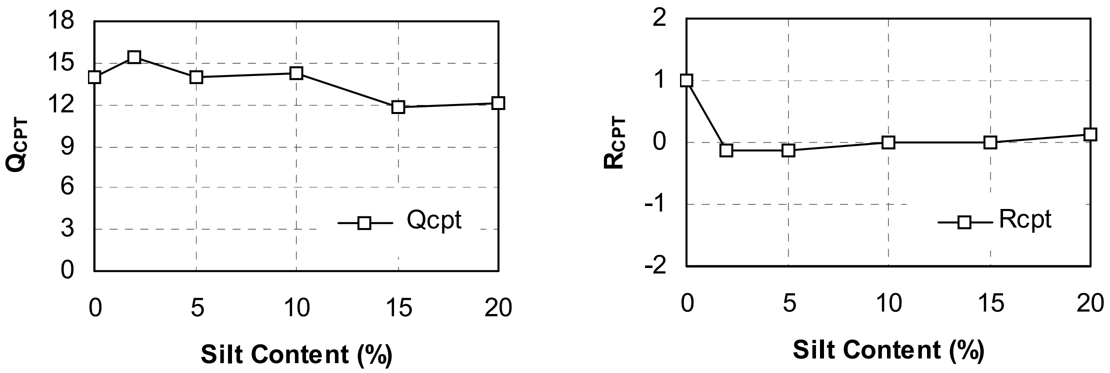
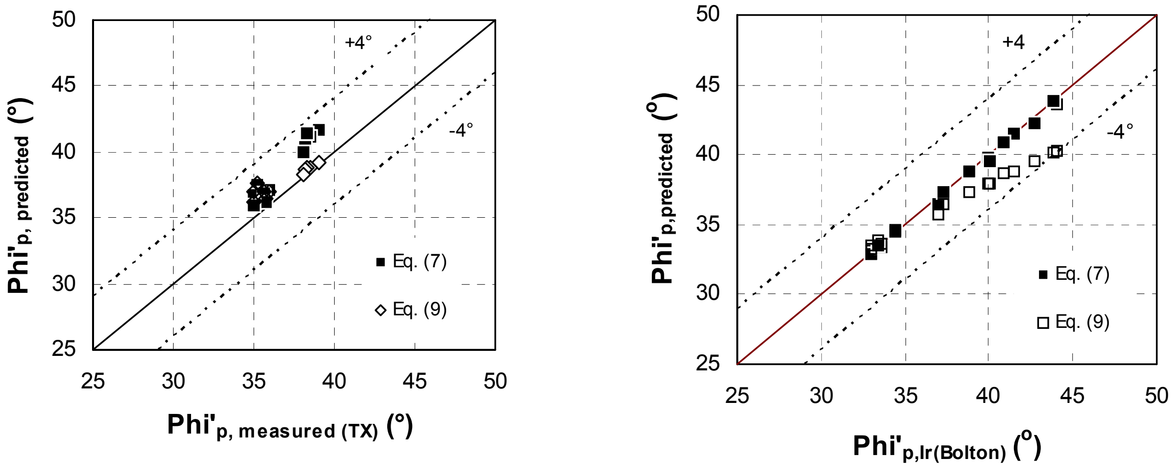


Fig. 3 Values of correlation parameter Q_{CPT} and R_{CPT}

5. Verification

In order to verify the proposed methods, calibration chamber cone penetration tests using Jumunjin sand and from literature were adopted and used in comparison and verification. A total of 11 calibration chamber CPTs using Jumunjin sand were performed at different relative densities and stress states. For cases from literature, 15 calibration chamber CPT results by Houlsby and Hitchman (1988) were adopted in this comparison.

Fig. 4(a) shows measured and predicted values of ϕ'_p for each calibration chamber test case using Jumunjin sand. It is observed that predicted results from Eqs. (4) and (5) show reasonable agreement with those measured from TX tests. Fig. 4(b) shows compared results for calibration chamber tests by Houlsby and Hitchman (1988). As Houlsby and Hitchman (1988) suggested and no TX test results were available, reference values of ϕ'_p adopted in the comparison were those obtained from Bolton's dilatancy relationship. It is observed that the modified dilatancy equation of Eq. (4) produces virtually the same results as those from Bolton's relationship. The simplified dilatacny equation of Eq. (9), on the other hand, tends to underestimate ϕ'_p as soil becomes more dilatant. Nonetheless, both methods appear to give satisfactory prediction of stregnth in practical purpose.



(a)　　　　　　　　　　(b)

Fig. 4 Measured and predicted values of ϕ'_p

6. Conclusion

In this study, methodology for the field application of the dilatancy equation based on CPT cone resistance q_c is investigated for sandy soils containing fines. Results from a series of laboratory tests and cone penetration analysis were used for the development of CPT-based methods of strength and dilatancy evaluation for sands. Based on empirical correlations between q_c and $_{mp}$for a given TX and equivalent field stress state, a modified dilatancy index $I_{R,CPT}$ in terms of q_c was proposed and investigated for different K_0 conditions. Results from both modified and original dilatancy indexes showed close agreements for soils at all the silt contents and K_0 values considered. As both q_c and $_{mp}$ are primarily governed by D_R and the confining stress with similar dependency, simplified dilatancy index $I_{R,CPT*}$ in terms of q_c was proposed as well.

For verifying the proposed methods, calibration chamber CPT results were adopted and used for comparison. Various soil conditions were used in both tests. It was observed that both methods of modified and simplified dilatancy relationships produce results reasonably close to those measured from triaxial tests and estimated from Bolton's original dilatancy relationship.

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