Asian Institute of Technology 40th Year Conference, New Frontiers & Challenges, Nov. 1999

NUMERICAL SIMULATION OF CPT TIP RESISTANCE IN LAYERED SOIL

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ABSTRACT

The paper presents the results of a numerical modeling for cone penetration in layered soil. Experimental studies show that as the tip advances into different soil layers, it senses the effects of an approaching layer. The cone penetration tip resistance is influenced by the soil properties ahead and behind the tip. The interface distance over which the tip resistance senses the effect of a soil layer is reported to be 5 to10 times the cone diameter.

With the present analysis, the complete process of cone penetration is modeled as the cone starts to penetrate the soil from the ground surface to any deeper layers below the ground. This capability of the program enables the analysis of penetration in layered soil to be modeled in a realistic way. The commercial computer program FLAC is used for this analysis.

INTRODUCTION

Cone penetration analysis has been the subject of research for more than three decades. To tackle this boundary value problem, many different procedures are suggested. Bearing capacity theory [Meyerhof (1961), Durgunoglu and Mitchell (1975)], cavity expansion theory [Vesic (1972), Yu and Houlsby (1991), Salgado et al. (1997), Shuttle and Jefferies (1998)], strain path method [Baligh (1985), Teh and Houlsby (1991)], and finite element analysis [van den Berg et al. (1996)] were used to analyze the penetration process. Yu and Mitchell (1998) present a comprehensive review of different methods in the analysis of cone resistance.

Though there have been a number of papers in the literature presenting solutions for cone penetration, the aspect of penetration in layered soil has been inadequately addressed.

In this Paper, a new approach for cone penetration is discussed. To validate the reliability of this approach the experimental results from calibration chamber tests in sand are compared with the numerical values obtained with the present approach. Thereafter, the results of numerical analysis in layered soil are discussed. This new modeling technique can be used to analyze the penetration in layered soil in a realistic way. The commercial computer code FLAC (1998) has been used for this analysis.

CONSTITUTIVE LAW

The Mohr-Coulomb elasto-plastic model was chosen for this problem. The values of stresses in close proximity to the cone tip are very much higher than those in the far field, and it is argued that the model parameters will therefore be different in the near and far field. Hence, in simulating the calibration chamber tests, the Mohr-Coulomb soil parameters are considered to be stress dependent.

The stress dependent relations for shear and bulk modulus used in the Mohr-Coulomb soil model are:

$$G = K_G P_A \left(\frac{\sigma'_m}{P_A}\right)^n \qquad [1]$$

$$B = K_B P_A \left(\frac{\sigma'_m}{P_A}\right)^m \qquad [2]$$

In the above relations, σ'_m is the mean effective stress, P_A is the atmospheric pressure, equal to 1 kg/cm^2 =98.1 kPa, m and n are constants which are both chosen to be 0.6, and K_G and K_B are constants that mainly depend on the relative density of the sand in the calibration chamber. The parameters used for K_G and K_B are shown in Table 1. These values are in the range of values reported by Byrne et al. (1987).

Drained shear strength parameters of Ticino sands used in the calibration chamber tests were found from triaxial tests carried out by ENEL/ISMES in Italy.

Baldi et al. (1986) have summarized the results of these tests in terms of the curvilinear formula given by Baligh (1975):

$$\tau_{ff} = \sigma'_{ff}\left[\tan\phi'_0 + \tan\alpha\left(\frac{1}{2.3} - \log_{10}\frac{\sigma'_{ff}}{P_A}\right)\right] \qquad [3]$$

Where τ_{ff} = shear stress on the failure surface at failure, σ'_{ff} = effective normal stress on the failure surface at failure, α = angle which describes the curvature of the failure envelope, and ϕ'_0 = secant angle of friction at σ'_{ff} = 2.72 P_A.

Table 1 also shows the values of ϕ'_0 and α as obtained by specimens of three different classes of relative density.

Table 1. Parameters used for deformation and shear strength of Ticino sand.

D_r %	K_G	K_B	ϕ'_0 (deg)	α (deg)
45	195	325	38.2	4.2
65	230	385	40.2	6.5
85	290	480	42.9	8.1

D_r = average relative density of the tested specimens, at the end of consolidation.

The dilational characteristics of the sand was given by the following relationship which relates the dilation angle to the developed friction angle and constant volume friction angle:

$$\sin\varphi = \sin\phi_f - \sin\phi_{cv} \qquad [4]$$

Parameters in the above relation are defined as: φ = dilation angle, ϕ_f = friction angle at failure, ϕ_{cv} = constant volume friction angle for Ticino sand, assumed equal to 34.8 degrees as described by Salgado et al (1997).

AN APPROACH IN CONE PENETRATION MODELING

The present analysis models the penetration process in a realistic way, in the sense that the penetration starts at the top of the grid (ground surface) and progresses into the grid (ground), and finally can end at any desired depth in the grid meaning that the modeling process is realistically simulating the cone moving downward in the ground. The axisymmetric configuration is used for this approach.

Since cone penetration is basically a large strain phenomenon, the soil under the cone tip undergoes a severe deformation pattern; and it is necessary to use the large strain option in the analysis to better simulate the process.

In order to physically simulate penetration in this approach, the soil elements located along the cone path are pushed away. The grid points associated with these soil elements are given a vertical downward as well as a horizontal displacement.

COMPARISON OF NUMERICAL RESULTS WITH EXPERIMENTAL VALUES IN A CALIBRATION CHAMBER

The numerical results for sands are compared with the experimental values obtained from penetration tests in Enel Cris calibration chamber in Italy. The test results together with the properties of sand used and type of boundary condition for each test are given in Lunne et al. (1997).

Figure 1 shows the predicted values of tip resistance versus the experimental values for a series of tests with BC1 type boundary condition for normally consolidated as well as over-consolidated Ticino sand. In this type of boundary condition, constant stresses are applied in the horizontal as well as the vertical directions in the calibration chamber. For these series of tests, the relative density ranged from 55% to 92%, and the vertical stress in the chamber

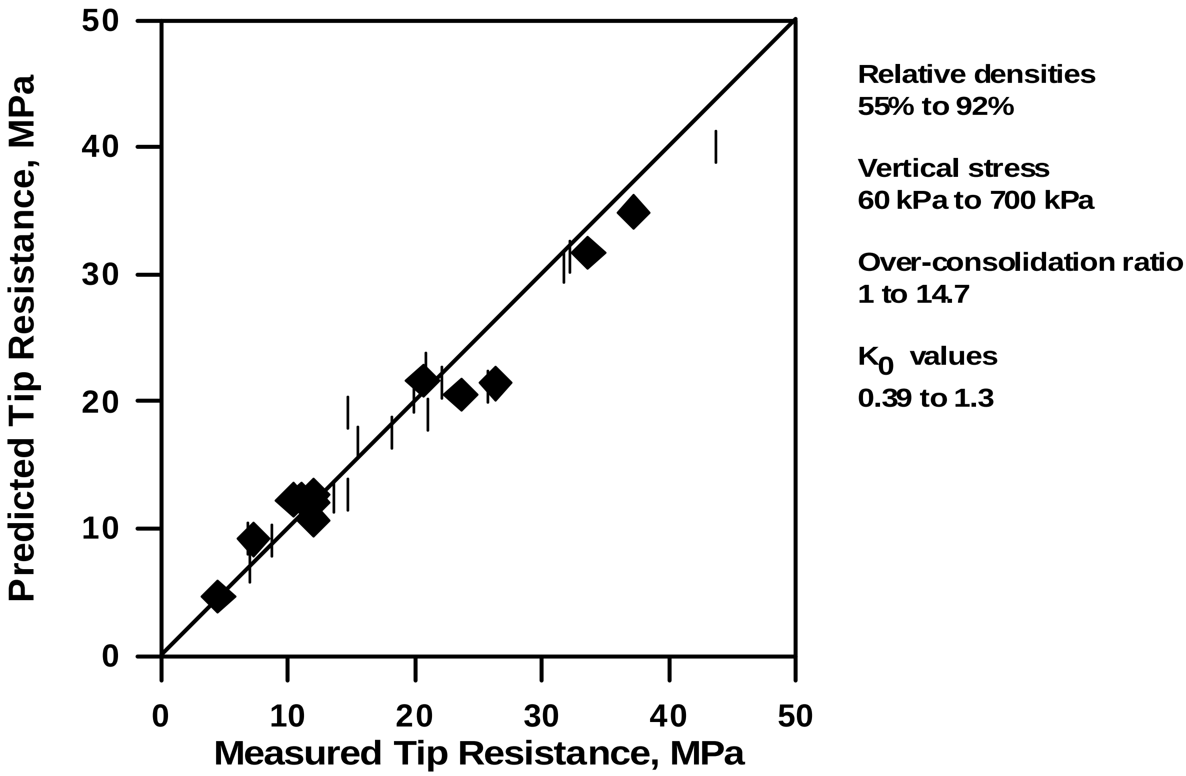

Figure 1. Agreement of predicted and measured tip resistance.

ranged from about 60 kPa to 700 kPa. The k_0 values were in the range 0.39 to 1.3, and the OCR values ranged from 1 for normally consolidated sand to 14.7 for over-consolidated sand. The points in this figure are close to the line with a slope of 45 degrees, indicating that the predicted values obtained from numerical analysis are in good agreement with the experimental values obtained in calibration chamber testing. It is also noted that for values of tip resistance more than 35 MPa, the numerical procedure systematically underpredicts the tip resistance values. These points correspond to experiments in which confinement stresses were high. The underprediction may be due to parameters in the model describing the dilatancy characteristics of the sand.

ANALYSIS FOR LAYERED SOILS

In the sections to follow, the results of numerical analyses showing the effects of soil layering on penetration resistance are discussed. The results of tip resistance for a loose sand over dense sand and for a dense sand over loose sand as well as the results of tip resistance for a medium with two different soils, i.e., sand and clay are presented.

LOOSE SAND OVER DENSE SAND

Figure 2 shows the result of the numerical analysis of tip resistance for a drained cone

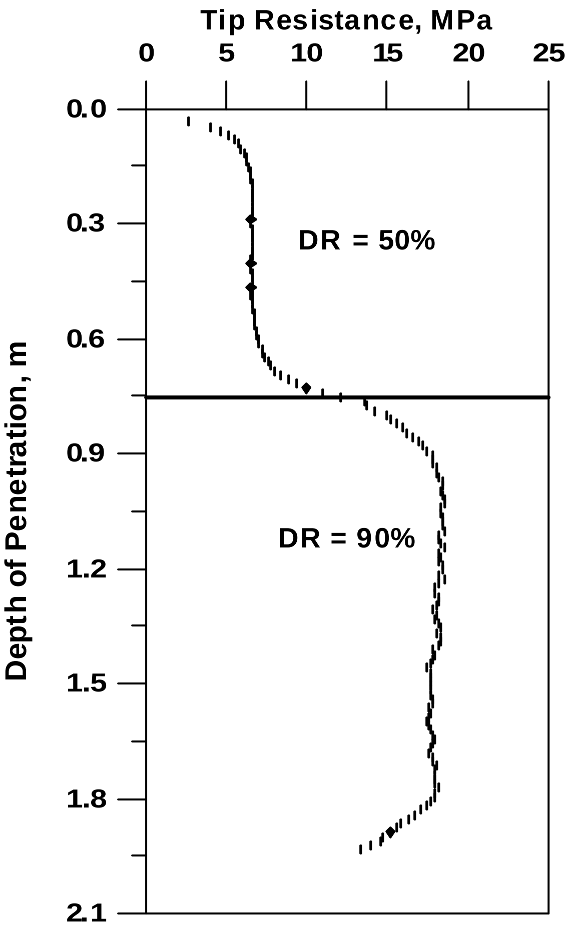

Fig. 2: Result of drained cone penetration analysis for loose sand overlying dense sand. (10 cm^2 cone, 60° tip, σ'_v = 100 kPa, k_0 = 0.5)

penetration in sand with two different relative densities. The loose sand has a relative density of 50% and the dense sand below has a relative density of 90%. The in-situ effective vertical stress is 100 kPa, and k_0 is 0.5. The height of the loose top layer is 0.75 m. The figure shows that as the cone tip approaches the dense layer the tip resistance increases. This increase in tip resistance occurs at a distance of 0.25 m above the dense layer. In other words, the tip senses the effect of the approaching layer 0.25 m ahead. This interface distance is approximately 7 times the cone diameter. Chamber studies show that the tip senses an interface distance of 5 to 10 cone diameters ahead and behind the tip [Campanella et al. (1995)]. The calculated value is well within the experimental range.

While the cone is inside the dense sand, but close to the loose layer above, its tip resistance is affected by the presence of the layer above. It takes a distance of 0.18 m for the cone tip to be influenced solely by the dense sand. The distance that the cone senses the top layer behind is about 5 cone diameters, which is again in agreement with experimental results.

The figure also shows that as the cone is approaching the bottom stress boundary, the tip resistance starts to decrease. This indicates that the cone in dense sand is beginning to pick up the effects of the bottom boundary, which is located at a depth of 2.3 meters, i.e., it is sensing the bottom boundary at a distance of about 0.5 m.

DENSE SAND OVER LOOSE SAND

The result of the analysis for a dense sand layer over loose sand is shown in Figure 3. The

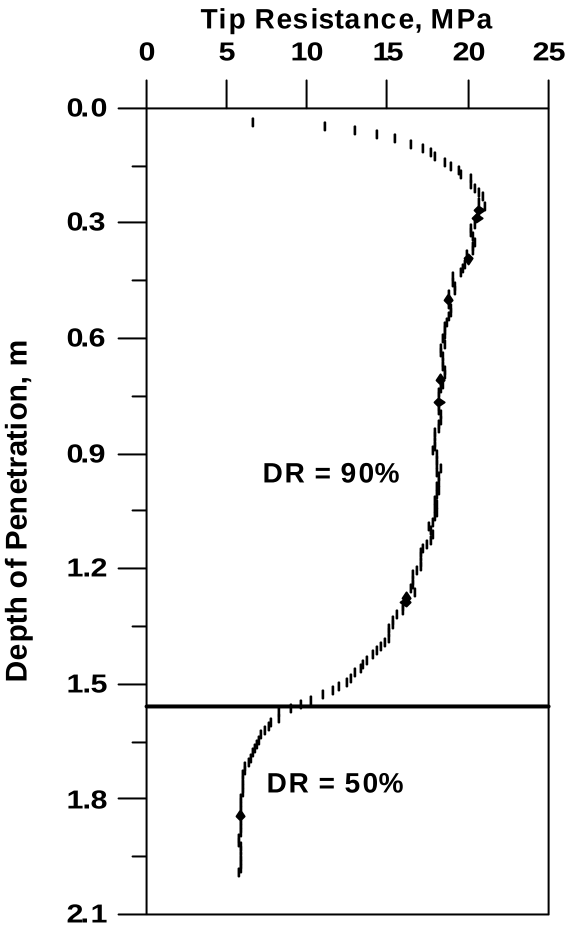

Fig. 3: Result of drained cone penetration analysis for dense sand overlying loose sand.
(10 cm^2 cone, 60° tip, σ'_v = 100 kPa, k_0 = 0.5)

same values of in-situ vertical and horizontal stresses are used for this analysis. The height of the dense sand is 1.56 m. This larger height was chosen to allow the cone in the dense layer to reach its tip resistance without the influence of the soil layer below. Once the cone approaches the bottom loose layer, as the figure shows, its tip resistance gradually decreases. The cone in the dense sand senses the effect of the approaching loose sand at a distance of 0.56 m, which is about 16 times the cone diameter. This value of interface distance is larger than the value obtained for the previous case. This may be because the dense sand, due to its higher stiffness, can project forward its influence over a wider zone [Mitchell and Brandon (1998)]. Thus, although the relative stiffness is the same for both cases, the sensing distance is much greater when a dense sand overlies a looser sand.

As the cone penetrates further into the loose sand, the effect of the overlying dense layer decreases; at a distance of 0.17 m the effect of top dense layer vanishes, and the tip resistance is only affected by the layer in which the cone is penetrating.

Comparing this figure with Fig. 2, it is seen that the cone resistance is not yet affected by the presence of the bottom boundary. For this analysis the location of the bottom boundary is the same as for Fig. 2. This further indicates that the interface distance depends on soil stiffness.

Figure 3 shows that the tip resistance near the start of penetration (top of the grid) is higher, and then it gradually drops. The larger values obtained in the analysis occur as a result of restraining the top boundary in order to simulate the rigid top platen used in most experimental calibration chambers today.

SAND ON CLAY

Figure 4 shows the numerical analysis of penetration in a sand layer above a clay. The sand has a relative density of 70%, and the clay has an undrained shear strength of 30 kPa. The values of in-situ effective vertical and horizontal stresses are 100 kPa and 50 kPa respectively, which are the same values chosen for the previous analysis. The sand layer covers the top 1.74 m of the grid. While the cone is in sand, its tip resistance is not affected by the clay layer until it reaches a distance of 0.45 m from the clay layer below. This distance is about 12.5 times the cone diameter. However, the tip resistance in the clay layer is only slightly affected by the sand layer above. It needs only a penetration of less than 0.05 m for the cone in clay, or about 1.7 cone diameters, to be solely dependent on the clay layer itself, and not the sand layer above. This demonstrates that the radius of the zone that affects the cone is much less for clay than for sand.

CONCLUSION

A new approach for cone penetration is presented in this paper. The numerical prediction of cone tip resistance obtained by this new approach is in good agreement with the experimental values in the calibration chamber with type BC1 boundary condition and a wide range of relative densities, vertical as well as horizontal stresses, and OCR ratios. The results of penetration in layered soil also give values of interface distance that are in good agreement with experimental values. The interface distance for a dense layer over loose one is larger than for a loose layer over a dense layer even though all parameters are the same. This may be because the dense sand, due to its higher stiffness, can project forward its influence over a wider zone. Based on these results, it can be concluded that for accurate characterization of soil stratigraphy and evaluation of engineering properties, the influence of layering on cone values must be considered in terms of the magnitude of interface distances both behind and ahead of the cone tip.

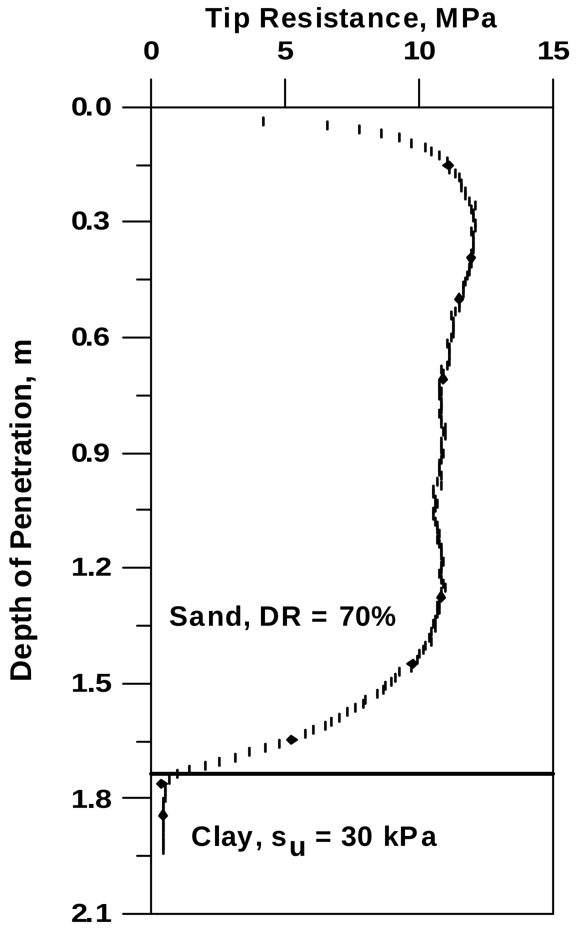

Fig. 4: Result of drained cone penetration analysis in sand into undrained clay.
(10 cm^2 cone, 60° tip, σ'_v = 100 kPa, k_0 = 0.5)

ACKNOWLEDGEMENTS

The authors are grateful to the Natural Sciences and Engineering Research Council of Canada (NSERC) for their financial support.

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