#155

Pan Am 99
BRAZIL

Simulation of Cone Penetration with FLAC

M. M. Ahmadi, P. M. Byrne and R. G. Campanella
Dept. of Civil Engineering, University of British Columbia, Vancouver, Canada

ABSTRACT: The cone penetration test (CPT) has been used for decades in in-situ geotechnical engineering practice. Over these years, there has been a great demand for validated correlations between cone resistance and engineering properties of soil. The correlations for sands are mostly obtained from experiments in calibration chamber tests with specified boundary conditions. The correlations for clays are mostly from laboratory tests on undisturbed samples. In this Paper, different attempts to simulate the cone penetration process using the commercial code FLAC are discussed.

In one approach, the cone is placed in a predetermined location in the grid, and is given a downward vertical deformation. In this attempt, the analytical result shows that with continued penetration, larger stresses around the cone are obtained. This is especially true for sand, and it is obviously unacceptable.

In another approach, the complete process of cone penetration was modeled as the cone starts to penetrate the soil from the ground surface to deeper layers. Based on the second approach, numerical results are compared with experimental values from calibration chamber tests in sands. It is shown that the second approach gives numerical values of tip resistance that are in good agreement with calibration chamber test results.

KEYWORDS: cone penetration analysis, tip resistance, calibration chamber, Mohr-Coulomb model.

1. 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 was one of the first methods used to predict the cone factor in clay [e.g. Meyerhof (1961) and Durgunoglu and Mitchell (1975)]. In the bearing capacity approach the effect of soil compressibility is neglected. To account for this shortcoming, cavity expansion theory was investigated by Vesic (1972), Yu and Houlsby (1991) and most recently by Salgado et al. (1997) and Shuttle and Jefferies (1998). A promising approach called "Strain Path Method" was suggested by Baligh (1985), and the results of analysis of penetration in clayey material based on this approach was presented by Teh and Houlsby (1991). Among other researchers, van den Berg (1998) implemented an Eulerian Finite Element analysis for both clay and sand. Yu and Mitchell (1998) present a comprehensive review of different methods in the analysis of cone resistance.

In this Paper, two different approaches for cone penetration are presented. The first approach in cone penetration modeling gives unreliable values for cone tip resistance. This is especially true for penetration analysis in sandy material. The second approach, however, gives values of cone tip resistance that are reasonable for both sands and clays. To verify the reliability of the second approach, the experimental results from calibration chamber tests were compared with the numerical analysis used in the second approach. Finally, the constitutive law used for these two approaches is discussed. The commercial computer code FLAC (1998) has been used for these two approaches.

2 FIRST APPROACH IN CONE PENETRATION

The method used in the first approach is basically a generalization of the method that has successfully been used for predicting the failure load of shallow foundations. Griffiths (1982) and de Borst and Vermeer (1984) have presented the result of their Finite Element analysis of cone penetration in clay. Their methodology is basically similar to this approach. In this approach, the variation of stresses versus displacements at points close to the cone tip is monitored. The collapse (or failure) load is reached when the stresses remain constant with continued increase in displacement.

In this first approach the cone is placed at the mid depth of an axisymmetric grid. The mesh close to the cone tip is more congested with elements so that the variation of actions around cone could be monitored with higher accuracy. The axisymmetric option is necessary for this problem to reduce the number of elements in the solution. In this model, the cone is separated from the surrounding soil by interface elements. Only small strain analysis could be investigated in this approach. The implementation of large strain option resulted in discontinuation of program execution due to the "Bad Geometry" message in FLAC. This message simply means that the execution cannot proceed due to the large distortion in the elements.

To simulate the penetration process, the points associated with the cone in the grid are given a downward movement. This results in an increase in the stresses below and around the cone tip. Figure 1 shows the variation of cone factor, $N_c = (q_c - \sigma_v)/s_u$, with respect to the penetration depth for an analysis in clayey material. In the above relation, q_c is the tip resistance, σ_v is the vertical in-situ stress, and s_u is the undrained shear strength of clay. As is shown in Figure 1, the cone factor, N_c, seems to reach a constant value of about 12.5 with continued penetration. This value of cone factor is in the range reported in the literature and obtained during practical cone penetration testing in clay. It should be noted that although the slope of the curve diminishes with increase in penetration of tip, the curve does not become completely flat.

However, it has been argued that this rather flat curve is acceptable for practical purposes. The situation is not so for penetration in sand. Figure 2 shows the variation of cone tip resistance versus the penetration of tip.

It is implied in this figure that tip resistance is still increasing after a penetration of 0.2 meters, and no failure load can be distinguished. This general trend for sand is unacceptable.

To overcome the difficulties associated with this approach, a different procedure for modeling was pursued, which will be discussed next.

3 SECOND APPROACH IN CONE PENETRATION MODELING

In this approach the process of cone penetration is modeled in a more realistic way. The modeling imitates the penetration 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 cone is realistically moving downward in the ground. The axisymmetric configuration is also 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 simulate better the process. Unlike the first approach, a large strain analysis could be performed in this second approach.

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

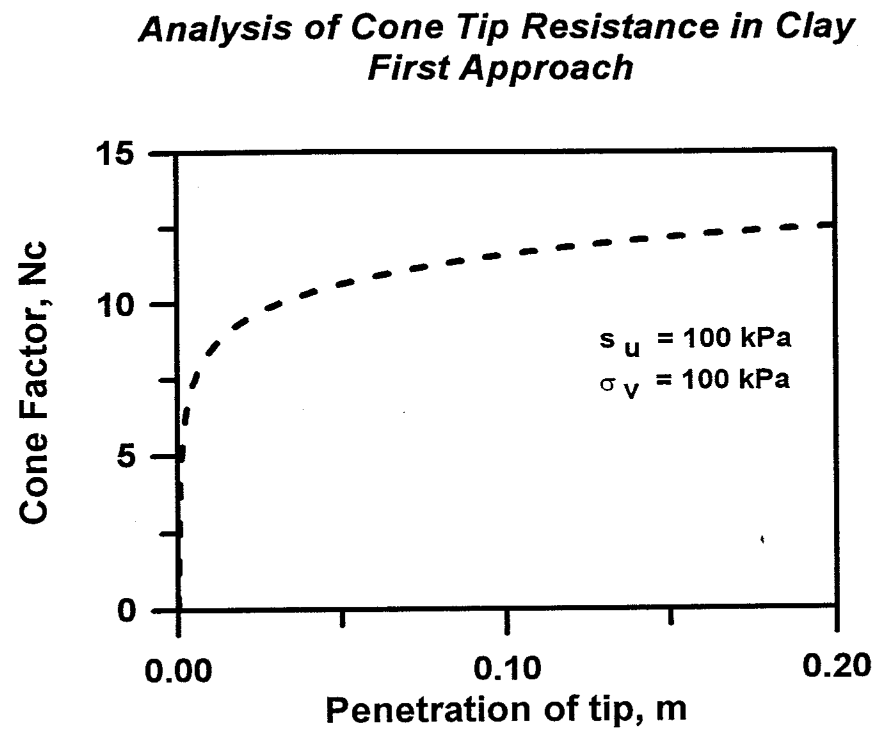

Figure. 1. A reasonable variation of cone factor with penetration of tip (clay soil).

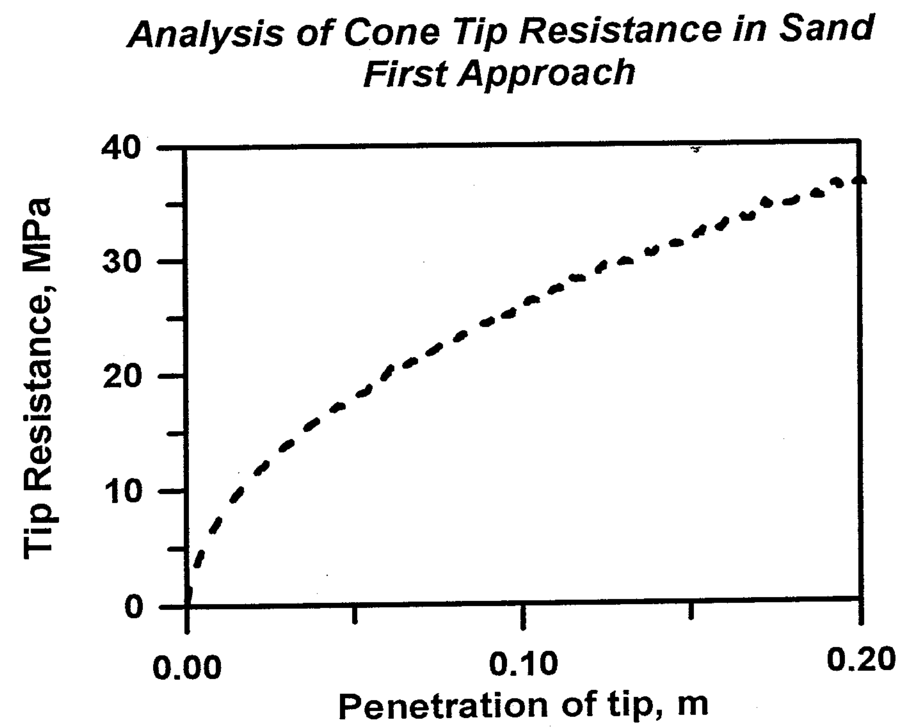

Figure 2. Unacceptable variation of tip resistance with penetration of tip (sandy soil).

downward as well as a horizontal displacement.

3.1 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 3 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 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 the numerical analysis based on this second approach 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.

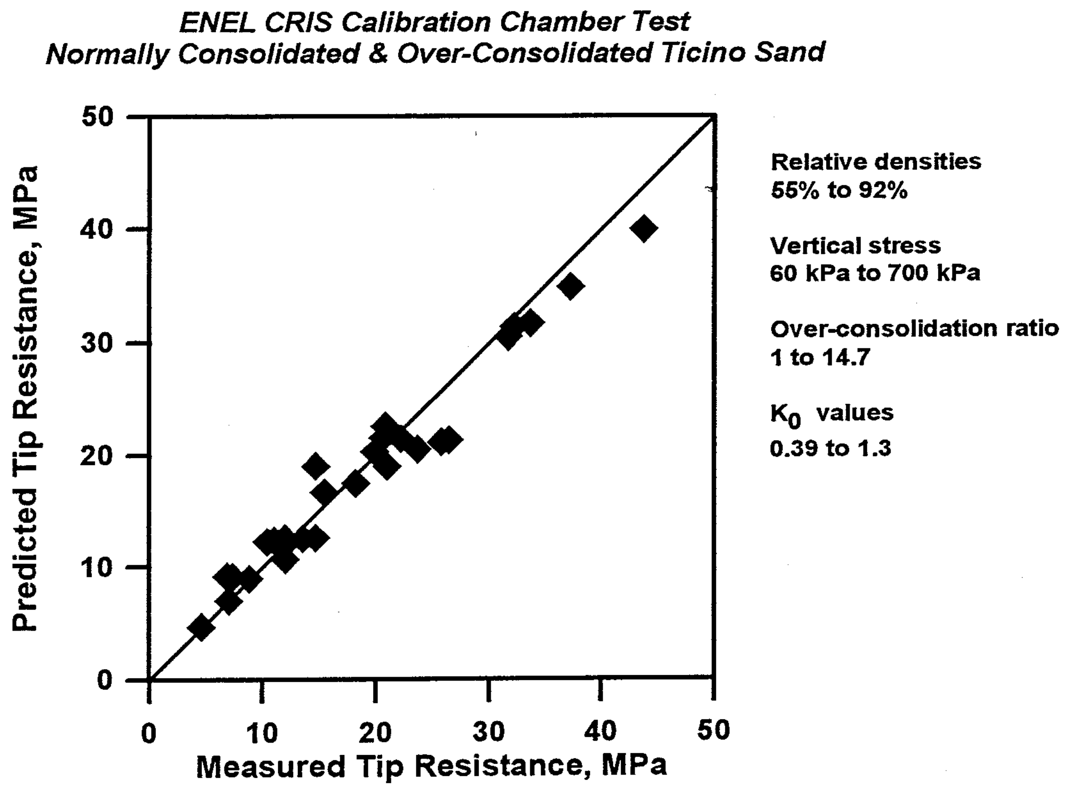

Figure 3. Agreement of predicted and measured tip resistance (second approach).

4 CONSTITUTIVE LAW

The Mohr-Coulomb elasto-plastic model was chosen for this problem. The values of stresses in close proximity of the cone tip are very much different from the far field. In this regard, it can be argued that the model parameters will be different in the near and far field. 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 in the Mohr-Coulomb soil model are:

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

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

In the above relations σ'_m is the mean effective stress, P_A is the reference pressure, assumed 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 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):

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

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

Table 1. Shear strength of Ticino sand.

D_r %	ϕ'_0 (deg)	α (deg)	R^2 (-1)
45	38.2	4.2	0.67
65	40.2	6.5	0.78
85	42.9	8.1	0.89

D_r = average relative density of the tested specimens, at the end of consolidation;
R^2= correlation coefficient.

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:

$$\sin\varphi = \sin\phi_d - \sin\phi_{cv} \tag{4}$$

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

5 CONCLUSION

Two different approaches for cone penetration in both clayey and sandy material are presented. The first approach does not provide a reliable answer especially for penetration in sand. The numerical prediction of cone tip resistance obtained by the second 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. A Mohr-Coulomb elasto-plastic soil model with stress dependent parameters was used in the numerical analysis.

ACKNOWLEDGEMENTS

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

REFERENCES

Baldi, G., Belotti, R., Ghionna, N., Jamiolkowski, M., and Pasqualini, E. (1986). "Interpretation of CPT's and CPTU's, 2nd part, drained penetration of sands." *Proc. 4th Int. Geotechnical Seminar*, Singapore.

Baligh, M.M. (1975). "Theory of deep site static cone penetration resistance." Report R75-56, No. 517, Dept. of Civil Eng., Massachusetts Institute of Technology, Cambridge, Massachusetts.

Baligh, M.M. (1985). "Strain path method." ASCE Journal of Geotechnical Engineering, vol. 111, No. 9, 1108-1136.

van den Berg, P., de Borst, R., and Huetink, H. (1996). "An Eulerian finite element model for penetration in layered soil." *Int. Journal for Numerical and Analytical Methods in Geomechanics*, 20, 865-886.

de Borst, R. and Vermeer, P.A. (1984). "Possibilities and limitations of finite elements for limit analysis." *Geotechnique*, 34(2), 199-210.

Byrne, P.M., Cheung H., and Yan, L. (1987) "Soil parameters for deformation analysis of sand masses." *Canadian Geotechnical Journal*, 24(3), 366-376.

Durgunoglu, H.T. and Mitchell, J.K. (1975). "Static penetration resistance of soils. I: Analysis." *Proc. ASCE Specialty Conference on In-Situ Measurement of Soil Properties*, ASCE, New York, Vol. 1, 151-171.

FLAC User Manual, Vol. 1-3, version 3.4, (1998), Itasca Consulting Group Inc., USA.

Griffiths, D.V. (1982). "Elasto-plastic analysis of deep foundations in cohesive soil." *Int. Journal for Numerical and Analytical Methods in Geomechanics*, 6, 211-218.

Lunne, T., Robertson, P.K., and Powell J.M. (1997). *"Cone Penetration Testing in Geotechnical Practice"*, Blackie Academic & Professional.

Meyerhoff, G.G. (1961). "The ultimate bearing capacity of wedge shaped foundations." *Proc. 5th Int. Conf. on Soil Mechanics and Foundation Engineering*, Paris, Vol. 2, 103-109.

Salgado, R., Mitchell, J.K., and Jamiolkowski, M. (1997). "Cavity expansion and penetration resistance in sand." *Journal of Geotechnical and Geoenvironmental Engineering, ASCE*, 123(4), 344-354.

Shuttle, D. and Jefferies, M. (1998). "Dimensionless and unbiased CPT interpretation in sand." *Int. Journal for Numerical and Analytical Methods in Geomechanics*, 22, 351-391.

Yu, H.S. and Mitchell J.K. (1998). "Analysis of cone resistance: review of methods." *Journal of Geotechnical and Geoenvironmental Engineering, ASCE*, 124(2), 140-147.

Vesic, A.S. (1972). "Expansion of cavities in infinite soil mass." *Journal of Soil Mechanics and Foundation Div., ASCE*, 98(3), 265-290.