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**INTERPRETATION OF CONE FACTOR IN UNDRAINED SOILS USING FULL-PENETRATION
FINITE ELEMENT ANALYSIS**

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ABSTRACT

The cone penetration test is widely used to interpret the undrained shear strength of soil. The interpretation methods are commonly based on bearing capacity, cavity expansion or strain path methods. In this paper, the cone penetration test is analysed using the large deformation finite element method and contact mechanics. The full penetration process from the ground surface is modelled. The 3-parameter Tresca model is used to represent the undrained soil behaviour in a total stress analysis, to derive a cone factor equation for interpretation of the undrained shear strength. This equation is then compared with existing solutions in the literature as well as experimental data. The complexities due to more sophisticated soil models are elaborated in an effective stress analysis using the Modified Cam Clay model.

KEYWORDS

CPT, cone factor, undrained shear strength, finite element method, large deformation

INTRODUCTION

The cone penetration test (CPT) is one of the most common methods used to characterise *in situ* soil properties. Amongst other soil properties, the undrained shear strength of clayey soils can be estimated from CPT data in conjunction with laboratory or field tests. Extensive research has been carried out to interpret the undrained shear strength from CPTs in cohesive soils, for example those methods based on bearing capacity theories (e.g. Meyerhof, 1961), those based on cavity expansion theories (Ladanyi 1963, Vesic 1972, Carter *et al.* 1986, Yu 1993), and those empirically based (e.g. Lunne *et al.* 1997). Teh and Houlsby (1991) combined the strain path of Baligh (1985) with finite element analysis to model the steady-

state cone penetration in a von Mises material and proposed an alternative equation for the cone factor. In these interpretation methods, the cone penetration is assumed to be analogical either to a deep footing problem, a cylindrical or spherical cavity expansion problem, or a prescribed strain path problem. The recent advances in modelling penetration problems using large deformation finite element method seem to provide an interesting alternative to study the cone penetration problem (Kioussis et al. 1988, van den Berg 1994, Sheng et al. 1997, 2005, Hu and Randolph 1998, Susila and Hryciw 2003, Abu-Farsakh et al. 2003, Huang et al., 2004, Weiher and Davis 2004, Wei et al. 2005, Tolooiyan and Gavin 2011, Obrzud et al. 2011). In particular, some of these studies involve the simulation of the full cone penetration process from the ground surface (Sheng et al., 1997 2005; Huang et al. 2004; Obrzud et al. 2011), hence alleviating the difficulty of considering the complex stress and strain fields around the penetrating cone or the need of assuming appropriate strain paths in the soil.

In this paper, the cone penetration test in undrained soils is analysed using the large deformation finite element method and contact mechanics. The full penetration process from the ground surface is modelled. The simple Tresca model is first used to represent the undrained soil behaviour in a total stress analysis, to derive a cone factor equation for interpretation of the undrained shear strength. This equation is further elaborated in the effective stress analysis of cone penetration in a Modified Cam Clay soil. All analyses in this study are carried out using the code ABAQUS Standard 6.10.

FULL-PENETRATION FINITE ELEMENT MODELLING OF CPT

The cone penetration test in this paper is modelled using large deformation finite element analysis. The cone is treated as an elastic material with a Young's modulus of 200 GPa and Poisson's ratio of 0.3. The undrained soil behaviour is idealised by the Tresca model, characterised by three soil parameters: Young's modulus (E), Poisson's ratio (ν) and undrained shear strength (c_u). The cone, which was initially hung over the ground surface, is inserted into a desired depth in the soil by prescribing the vertical displacement at the top end of the cone shaft. The cone-soil interaction is modelled using contact constraints (Wriggers 2002; Sheng et al. 2007), with a simple Coulomb friction law for the interface. In this study, the interface friction angle is assumed to be zero, i.e. a smooth cone. The cone resistance is

Figure 1. The finite element mesh used in the analyses (6-noded triangle elements).

computed from the vertical reaction force at the top end of the shaft divided by the cross section area of the shaft. A standard cone with a cone angle of 60° and a shaft cross section area of 10 cm^2 is used in this study. The soil domain is 11.2 shaft radii wide and 112 radii deep. Both the soil and the cone are discretised into 6-noded axis-symmetric triangle elements.

The finite element mesh used to obtain all the results in this paper is shown in Figure 1. As shown in Sheng et al. (2009), the computed cone resistance is affected by the size of the mesh,

Soil Properties: $c_u=10\text{kPa}$, $E=1000\text{kPa}$, $\nu=0.4$

(a) Fine initial mesh with cone embedded in the soil

(b) Predicted cone resistances

Figure 2. Sensitivity analysis using a finer mesh.

98 but converges to a finite value as the mesh is continuously refined. Finer meshes are prone to
 99 numerical breakdown due to mesh distortion and convergence problem. The mesh shown in
 100 Figure 1a is possibly the finest one that can result in successful analysis of the penetration
 101 process, without any numerical enhancement such as mesh refinement. The finer mesh
 102 shown in Figure 2a can also lead to successful analysis, but the cone has to be embedded into
 103 the soil initially and the Arbitrary Lagrangian-Eulerian (ALE) method has to be applied at
 104 certain time steps to avoid mesh distortion. Even with the ALE method (Nazem et al. 2006;
 105 Sheng et al. 2009), the optimal frequency for mesh update can only be found out by trial and
 106 error and the analysis broke down at small penetration depths (~ 5 shaft radius for the mesh in
 107 Figure 2a). The computed cone resistance using the finer mesh is about 5% lower than the
 108 one obtained using the coarse mesh in Figure 1.

Figure 3. Effect of Poisson's ratio.

The mesh size effect is somewhat offset by using a Poisson's ratio of 0.4 for the undrained soil. Figure 3 plots the computed steady state cone resistance against Poisson's ratio, with $c_u=10$ kPa and $G/c_u=35.7$. The cone resistance increases about 10% as Poisson's ratio increases from 0.4 to 0.48. Griffiths (1982) also noted similar effect of Poisson's ratio when studying the base capacity of axi-symmetric deep foundations. However, larger Poisson's ratios tend to cause numerical breakdown, particularly for high rigidity indices. Therefore, the Poisson ratio is set to a constant of 0.4 in this study, assuming that its effect on the cone resistance is somewhat offset by the mesh size effect. This is clearly a limitation of the finite element analysis presented here, but the use of a larger Poisson's ratio would not significantly change the main conclusions of this study.

The computed cone resistance shown in Figure 1b or Figure 2b reaches a steady state value at the depth of about 10 shaft radii, in the weightless Tresca soil. This steady state resistance is used in the analysis of the cone factor in the following sections.

COMPARISON AGAINST EXISTING SOLUTIONS

Justification of the use of the cone factor

It is a common practice to link the undrained shear strength with the cone resistance through a so-called cone factor:

Figure 4. Justification of the use the cone factor (varying σ_{v0}).

$$129 \quad \frac{q_t - \sigma_{v0}}{c_u} = N_{kt} \left(\frac{G}{c_u} \right) \quad (1)$$

130 where q_t is the cone resistance, σ_{v0} the initial vertical stress at the depth of the cone, c_u the
 131 undrained shear strength, G the shear modulus in the soil constitutive model and is usually set
 132 to the value at 50% peak shear stress (Yu and Mitchell 1998), and N_{kt} the cone factor which is
 133 usually assumed to be a function of the rigidity index (G/c_u). The use of such a cone factor
 134 implies that

- 135 1. For the same soil (c_u and G), the net cone resistance ($q_t - \sigma_{v0}$) is constant and
 136 independent of the initial vertical stress;
- 137 2. For proportional variation of the initial vertical stress (σ_{v0}), the undrained shear strength
 138 (c_u) and the shear modulus (G), the cone factor (N_{kt}) should remain constant and hence
 139 the cone resistance (q_t) should vary in proportion.

140 The above implications can be verified in the finite element analysis. In Figure 4, the
 141 computed cone factor is plotted for fixed c_u and G values, but varying initial vertical stress
 142 applied at the ground surface. The oscillation in the computed cone resistance is mainly due
 143 to the node-to-segment discretisation of the cone-soil contact. This oscillation, which was

Figure 5. Justification of the use the cone factor (varying c_u).

noted as early as in Simo and Meschke (1993), was discussed in detail in Sheng et al. (2006) and Fischer et al. (2007). Neglecting the oscillation, the differences in the computed steady state cone resistances are insignificant, indicating that the net cone resistance ($q_t - \sigma_{v0}$) is independent of the initial vertical stress. In Figure 5, the computed net cone resistance ($q_t - \sigma_{v0}$) is plotted for different sets of c_u and G values. The steady state cone factor (N_{kt}) derived from Figure 5 is around 10.5 for all c_u values. It is clear that the cone factor is more or less constant as long as σ_{v0} , c_u and G vary in proportion. Therefore, the results from the finite element analyses support the use of a cone factor in a Tresca type material.

Comparison against existing solutions

There are a number of equations in the literature that links the cone resistance to the undrained shear strength of the soil (see Yu and Mitchell 1998, Schnaid, 2009 for review). Some commonly used ones are listed below:

Vesic (1972):
$$N_{kt} = 3.9 + 1.33 \ln \left(\frac{G}{c_u} \right)$$

Baligh (1975):
$$N_{kt} = 12 + \ln \left(\frac{G}{c_u} \right)$$

Figure 6. Computed cone resistance (FP-FEM) in comparison with existing solutions.

158 Teh & Houlsby (1991):
$$N_{kt} = \left(1 + \ln \left(\frac{G}{c_u} \right) \right) \left(1.67 + \frac{G}{1500c_u} \right)$$

159 Yu (1993):
$$N_{kt} = 4.18 + 1.155 \ln \left(\frac{\sqrt{3} G}{2 c_u} \right)$$

160 The original equation proposed by Teh and Houlsby (1991) includes terms due to surface
 161 roughness and initial stress anisotropy. These terms are dropped here for the smooth cone.
 162 The equations by Vesic (1975), Baligh (1975) and Yu (1993) are all based on cavity
 163 expansion theories, while the equation by Teh and Houlsby (1991) is based on strain-path
 164 finite element analysis.

165

166 The cone factor based on the full-penetration finite element analysis (FP-FEM) is compared
 167 with the above existing solutions in Figure 6. The computed cone factor varies between 10
 168 and 20 for the rigidity index between 35 and 450. These N_{kt} values are significantly higher
 169 than those by Vesic (1975), Teh and Houlsby (1991) and Yu (1993), but smaller than that by
 170 Baligh (1975) for low rigidity indices. They are also higher than the numerical cone factors
 171 reported by Lu et al. (2004) and Liyanapthirana (2009), where the studied cones were initially
 172 embedded in the soil. One possible reason for the difference with other results reported in the

Figure 7. Cone resistance at small penetration depths.

literature is that the nodes beneath the cone tip ($r=0$) are allowed to move in the radial as well vertical direction in the full penetration analysis. Test runs indicate that fixing the horizontal moments of the nodes at $r=0$ would lead to a much lower cone resistance, which is somewhat against intuition. Another possible reason is the uncertainty involved in determining the steady state cone resistance at small penetration depths. Figure 7 replots the computed cone resistance shown in Figure 1b at small depths (2.5~5 radii). The cone resistance seems to have reached a steady state value at all the three depths, being 63 kPa, 68 kPa and 91 kPa for 2.5, 3.0 and 5.0 radii, respectively. However, the actual steady state value is around 103 kPa and is reached at the depth of ~ 10 radii. The depth where the steady state value is reached also changes with the material properties. In addition, the cone resistance values shown in Figure 1b and Figure 7 are taken at every 100 time steps in the finite element analysis, due to the large amount of data. Recording the data more frequently will lead to more oscillation for the curves shown in Figure 7.

The high N_{kt} values reported in this study may also be caused by the effects of the altered stress field due to full penetration. The results in Figure 6 seem to indicate that the stress disturbance effects due to cone penetration are more pronounced in soils with higher rigid indices. The computed cone factor is not linearly related to the logarithmic rigidity index as

Figure 8. Equation (2) compared with the FP-FEM and Teh & Houlsby solution.

191 in the cavity expansion solutions (Figure 6b). The trend of the computed cone factor is very
 192 similar to the solution of Teh and Houlsby (1991). A slightly modified version of Teh and
 193 Houlsby (1991) would lead to almost identical N_{kt} values as the full-penetration finite
 194 element analysis (Figure 8):

$$195 \quad N_{kt} = \left(1 + \ln \left(\frac{G}{c_u} \right) \right) \left(2.3 + \frac{G}{1000c_u} \right) \quad (2)$$

196 We also note equation (2) contains only one soil parameter: the rigidity index. This
 197 parameter is usually calibrated using laboratory or field data, see the section on comparison
 198 against experimental data. It is possible to extend the equation to include other parameters
 199 such as initial stress anisotropy, cone-soil interface friction, soil over-consolidation ratio
 200 (OCR), soil friction angle, etc. The cost is then additional unknowns are introduced into the
 201 interpretation equation. Furthermore, the stress anisotropy caused by the penetration of the
 202 cone is already included in the FP-FEM results. Parameters like OCR and soil friction angle
 203 are indirectly related to the undrained shear strength and their effects on the cone factor are
 204 partly included in equation (2), see the discussion in the following section. The cone-soil
 205 interface friction is likely to increase the computed cone factor somewhat, but is generally
 206 less significant in saturated clay than in sand. Including this friction will not only cause

numerical convergence problems in the FP-FEM, but also create problems to the contact kinematics (the Coulomb friction law will violate the undrained shear strength of the soil).

CONE PENETRATION IN MODIFIED CAM CLAY

Modified Cam Clay parameters

While the analysis of the cone penetration in the three-parameter Tresca soil is relatively straightforward, the undrained shear strength in reality is not a constant for one soil. It depends on soil initial conditions (e.g. water content or over-consolidation ratio), stress path and failure modes. A somewhat better constitutive model that can quantify some characteristics of undrained shear strength is perhaps the Modified Cam Clay (MCC) model. In the MCC model, the undrained shear strength is not a model parameter, rather an outcome of stress path. Wroth (1984) first derived the undrained shear strength under triaxial stress states using critical state parameters and the over-consolidation ratio:

$$c_u = \frac{M}{2} \left(\frac{OCR}{2} \right)^{\lambda - \kappa / \lambda} \sigma'_{v0} \quad (3)$$

where M is the slope of the critical state line and is assumed to be a constant, OCR is the over-consolidation ratio, λ and κ are the slope of the normal compression line and the unloading-reloading line respectively, and σ'_{v0} is the initial vertical effective stress in the soil. To model the cone penetration in an MCC soil, we need to invoke coupled displacement and pore pressure analysis. The material properties in the Modified Cam Clay model are listed in Table 1.

Table 1. Parameters adopted for the MCC model

e_0	γ_t (kN/m ³)	ν	κ	λ	M	K_0	k (m/s)	p_c (kPa)
2.0	15.5	0.333	0.01~0.1	0.1~0.5	0.8~1.2	0.5	10^{-8}	40, 70, 100
Note: e_0 : initial void ratio; ν : Poisson's ratio; γ_t : total unit weight; λ : slope of normal compression line in $e - \ln p'$ plot (p' is the effective mean stress); κ : slope of unloading-reloading line in $e - \ln p'$ plot; M : stress ratio at failure, q_f/p' (q_f is the deviator stress at failure); K_0 : at-rest earth pressure coefficient; k : hydraulic conductivity; and p_c : the size of the initial yield locus.								

The finite element mesh is the similar to that shown in Figure 1a, except every two triangles are replaced by one 8-noded quadrilateral element. The 8-noded quadrilateral elements with

4 integration points seem to be more robust for the cone penetration in MCC soils. Chai et al. (2012) carried out similar ABAQUS analyses to study the dissipation curve in MCC soils. The top boundary of the soil domain is assumed to be fully drained (zero pore water pressure). The left, right and bottom boundaries are assumed to be undrained (zero flux). The soil is fully saturated with an initially hydrostatic pore water pressure distribution. The rate of cone penetration is 2 cm/sec.

Cone factor and undrained shear strength

In the case of the Modified Cam Clay model, the initial vertical and horizontal stresses, overconsolidation ratio, undrained shear strength and shear modulus all vary with depth. The obvious question is: does a steady state cone resistance still exist in such a soil. Figure 8a shows the variations of undrained shear strength, OCR and initial vertical stress over the depth. The undrained shear strength values were obtained from equation (3). The computed net cone resistance is plotted in Figure 9b. It is clear that the cone resistance reaches a

Figure 9. Undrained shear strength, net cone resistance and cone factor over the depth.

Figure 10. Variation of cone factor with M ($\lambda=0.3$, $\kappa=0.05$, $\sigma_{v0}=50\text{kPa}$, $p_c=70\text{kPa}$).

‘steady state’ value at the depth of 5-10 shaft radii, and the steady state cone resistance then increases linearly with depth. The computed cone factor (FP-FEM) is compared with Teh and Houlsby’s solution and equation (2). Equation (2) seems to give very good prediction of the numerical cone factor from the full-penetration finite element analysis. The material parameters used to obtain the results in Figure 8 are: $M=1.0$, $\lambda=0.3$, $\kappa=0.05$. An initial vertical stress of 50kPa was applied at the ground surface. The initial isotropic yielding stress (p_c) is set to 70kPa. Because the yield surface in the MCC model is a circle in the deviatoric plane, a value of $M=1$ corresponds a soil friction angle of 25.4° in triaxial compression or 36.9° in triaxial extension. All other parameters are given in Table 1.

The computed cone factor varies with the failure stress ratio (M) and OCR while all other parameters are fixed. Figures 10 and 11 show the variation of N_{kt} with M and OCR , respectively, according to the full-penetration finite element analysis (FP-FEM), Teh and Houlsby (1991) and equation (2). The N_{kt} values were taken at the penetration depth of 84 shaft radii. In Figure 10, the computed cone factor by the FP-FEM is somewhere between those predicted by Equation (2) and Teh and Houlsby (1991). In Figure 11, the FP-FEM solution is very close to equation (2) for OCR between 1-1.8. Overall, equation (2) seems to capture the cone factor in the Modified Cam Clay soil reasonably well.

Figure 11. Variation of cone factor with OCR ($M=1.0$, $\lambda=0.3$, $\kappa=0.05$).

Another complication for the MCC model is that the computed cone factor varies with the soil compressibility (λ), even though all other parameters are fixed. The theoretical undrained shear strength defined by equation (3) does not change with λ , as long as the ratio $(\lambda - \kappa) / \lambda$ remains constant. Figure 12 shows that the computed cone factor varies with λ and equation (2) overestimates the numerical cone factor by about 8% when the soil is very soft ($\lambda=0.5$). The variation of N_{kt} with λ is usually not reflected in most cone factor equations in the literature, not at least directly.

In summary, the interpretation of cone penetration in the Modified Cam Clay is more complex. The undrained shear strength of the soil may vary with a number of factors such as OCR , soil compressibility, soil friction angle, etc. The inter-relation of these factors makes the interpretation even more difficult. Overall, equation (2) seems to provide reasonable approximation of the numerical cone factor. The trend of the cone factor variation with the critical state friction angle (M) and over-consolidation ratio (OCR) is also reasonably captured by equation (2), even though the equation does not directly involve these parameters.

COMPARISON AGAINST EXPERIMENTAL DATA

The undrained shear strength cannot be determined from cone penetration test alone. It is common to calibrate the cone factor against field vane test or laboratory triaxial test. One

Figure 12. Variation of cone factor with λ ($M=1.0$, $(\lambda-\kappa)/\lambda=0.833$, $\sigma_{v0}=50\text{kPa}$, $p_c=70\text{kPa}$).

exception is perhaps the interpretation method proposed by Larsson and Åhnberg (2005) where the cone factor is calibrated against the liquid limit of the soil:

$$N_{kt} = 13.4 + 6.65w_L \quad (4)$$

where w_L is the liquid limit of the soil.

Luke (1996) provided chamber test data of cone resistance in clayey soils, as well as triaxial data of undrained shear strength and rigidity index. The liquid limits of the soils tested were also given in Luke (1996). These data can be used to derive the cone factor and are compared with equation (2) in Figure 13. The experimental N_{kt} values are scattered between 7~25. Equation (2) gives reasonable estimation of the cone factor at low rigidity indices, but somewhat over-estimates the cone factor at high rigidity indices when compared with the values derived from the triaxial undrained shear strength data. On the other hand, the cone factor values predicted by equation (2) are significantly lower than those based on the measured liquid limits according to Larsson and Åhnberg (2005).

Kurup et al. (1994) provided chamber test data of cone resistance in kaolinite and sand mixtures, together with some c_u data from undrained triaxial compression tests. The cone factors derived from the test data are compared with equation (2) in Table 2. The experimental data are less scattered and compare remarkably well with the theoretical values

Figure 13. Comparison of Equation (2) against experimental data (the data by Luke (1996) is based on triaxial tests; the data by Larsson & Åhnberg (2005) is based on liquid limits).

according to equation (2), except for sample No. 4 where the two differ by a factor of 3. It should be noted that the undrained shear strength and the rigidity index for sample No.4 in Kurup et al. (1994) were not measured from triaxial tests and only estimated values were given in Kurup et al. (1994).

Table 2. Comparison against experimental data by Kurup et al. (1994).

Sample	c_u (kPa)	I_r	N_{kt} (Kurup et al.,1994)	N_{kt} (Equation 2)
No. 1	80	100	13	13.45
No. 1	80	100	13	13.45
No. 2	40	150	15	14.73
No. 2	40	150	16	14.73
No. 2	40	150	14	14.73
No. 3	60	267	16	16.91
No. 3	60	267	16	16.91
No. 3	60	267	14	16.91
No. 4	65	567	7	21.04
No. 4	65	567	7	21.04
No. 4	65	567	6	21.04

More recently, Low (2009) provided extensive experimental data of various penetrometers in soft soils. Laboratory and vane shear data of the undrained shear strength were also provided. Figure 13 shows the experimental N_{kt} values compared with equation (2). The experimental N_{kt} values were obtained using the average undrained shear strength values from triaxial and simple shear tests. The rigidity indices in Figure 14 were based on the scant shear modulus at 50% of maximum shear or deviator stresses. The experimental N_{kt} values from 6 test sites are

Figure 14. Comparison of Equation (2) against experimental data of Low (2009).

scattered between 7~17, but the average values and the trend seem to be well captured by equation (2).

In summary, Equation (2) seems to give a reasonable representation of the experimental cone factors estimated from chamber, field and laboratory tests.

CONCLUDING REMARKS

Numerical modelling of the cone penetration test can generate information that is very useful for interpreting the test results. One key feature of the finite element analysis used in this paper is that the full penetration process is modelled, alleviating the difficulty in considering the effects of stress disturbance due to cone penetration. The greatest challenge in such a full penetration analysis is to obtain a convergent solution. Great care must be taken in selecting time step size, element size and solution strategy control. In this study, the commercial code ABAQUS was used to obtain all the results. It is the authors' intention to make the data files available for interested readers.

The cone resistance values obtained from the full-penetration finite element analysis are, while qualitatively comparable with, generally higher than most small-strain theoretical solutions in the literature. The numerical analysis also confirms that the net cone resistance

normalised by the undrained shear strength of a Tresca soil, or the so-called cone factor, is dependent on the rigidity index of the soil. Such a dependency makes the direct interpretation of undrained shear strength from cone penetration tests difficult. The cone factor expressed in terms of rigidity index seems to be applicable to more complex soil models such as the Modified Cam Clay model, but the interpretation for the undrained shear strength becomes even more complex as the number of model parameters increases.

The cone factor equation proposed by Teh and Houlsby (1991) was modified based on the full-penetration finite element analysis. The modified equation seems to capture the numerical cone factor reasonably well. This equation was also compared against experimental data from laboratory chamber tests and field measurements and it seems to give reasonable prediction of the experimental data for a range of soils.

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