Effect of penetration rate on piezocone tests in soft clay

D.S. Liyanapathirana
School of Engineering, University of Western Sydney, Locked Bag 1797, Penrith South DC, NSW 1797, Australia

ABSTRACT
This paper presents a numerical study based on the finite element method investigating the effect of the penetration rate on piezocone resistance in saturated soft clays. The numerical model used for the finite element analysis has the ability to simulate penetration from the ground surface and piezocone tests are simulated at various penetration rates investigating the change in drainage conditions from undrained to fully drained for soils with different over consolidation ratios. Backbone curves depicting normalised penetration rate against tip resistance and excess pore pressure are developed capturing the transition from undrained to drained penetration. Numerical results are compared with cavity expansion theory and centrifuge model tests carried out at varying rates and subsequent dissipation tests. A correlation between cone resistance and drainage conditions has been established. The excess pore pressure generation at the cone tip is larger than that at the shoulder and is resulting from changes in octahedral normal stress in both normally and over consolidated soils.

RÉSUMÉ
Quand des sols contaminés sont chauffés, la pression de vapeur des composés chimiques organiques augmente. L'augmentation de la volatilité aide à l'élimination des contaminants par des méthodes conventionnelles telles l'extraction par vapeur et l'extraction à phases multiples. On peut chauffer le sol par des méthodes électriques in-situ en utilisant le chauffage par conduction conventionnel, le chauffage électrique par résistance ou le chauffage par électromagnétisme.

1 INTRODUCTION

The piezocone penetration test is widely used in geotechnical engineering practice to investigate the engineering properties of in-situ soils. The main advantage of the test is that it can be used to obtain continuous profiles of soil properties with depth under existing stresses and boundary conditions in the field. In recent years the usage of piezocone in geotechnical site investigations has been increased and used for construction control, assessment of ground improvement effectiveness, assessment of contaminant transport through soil and bearing capacity evaluations for deep foundations (Voyiadjis and Song, 2003).

The interpretation of piezocone results is not straight forward as number of variables such as location of the filter element, rate of penetration, type of soil and the design of the cone influence the measured data. According to the ASTM standard and the International Reference Testing Procedure (IRTP) the standard penetration rate for performing the test is 20±5 mm/s (Kim et al., 2008). This rate is given irrespective of the soil type. If the test is performed in sand with the standard rate, fully drained conditions prevail. On the other hand in clay soils, fully undrained conditions prevail if the test is performed with standard rate. If the soil type consists of a mixture of clay and sand, then partially drained conditions may prevail during penetration. Therefore it is important to investigate the influence of penetration rate on the measured penetration resistance during a piezocone penetration test.

If the penetration rate is sufficiently low for a particular soil, soil ahead of the advancing cone consolidates developing larger stiffness and shear strength than it would have under undrained conditions. Hence the measured cone resistance, q_c, reaches the maximum value in a particular soil type when the piezocone penetration rate is sufficiently low and allows soil to consolidate reaching fully drained conditions.

In recent years number of researchers have shown the significance of piezocone penetration rate on the measured penetration resistance and the excess pore pressure generation around the advancing penetrometers using calibration chamber tests (Kim, 2004, Kim et al., 2008), centrifuge tests (Randolph and Hope, 2004), field tests (Kim et al., 2008) and cavity expansion theory (Silva et al., 2006)).

In this research, the finite element software ABAQUS/Standard is used to simulate the penetration of a piezocone from the ground surface while maintaining a high quality mesh throughout the analysis. The numerical model is validated using the centrifuge data from Randolph and Hope (2004). Effect of penetration rate on measured penetration resistance, pore pressure generation and pore pressure dissipation are investigated for both normally and over consolidated soils. Finally a correlation between piezocone resistance and drainage conditions has been established based on the numerical results and excess pore pressure dissipation around the cone tip and shoulder are discussed for both normally consolidated and over consolidated soils.

2 NUMERICAL MODEL

A numerical procedure based on the coupled theory of nonlinear porous media has been used to simulate the piezocone penetration in soft clay. Numerical simulations have been carried out using the ABAQUS/Standard. Piezocone penetration has been carried out similar to the cone penetration simulated by Huang et al. (2004) in cohesionless soils and pile jacking simulated by Liyanapathirana (2008) in soft clay.

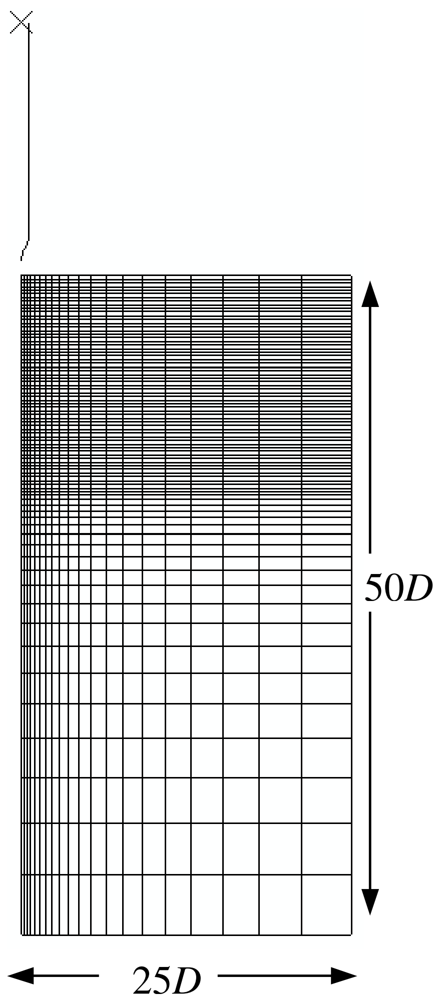

Figure 1. Finite element mesh.

The finite element discretisation of the soil domain is shown in Figure 1, where the soil is modelled using eight-node axisymmetric elements with pore pressure degrees of freedom at four corner nodes. Constitutive behaviour of the soil is modelled using the Modified Cam Clay model (Roscoe and Burland, 1968). Soil domain has been extended 25*D* in radial direction away from the piezocone wall and 50*D* away from the piezocone tip in the vertical direction at the commencement of the penetration. This will avoid the influence of boundaries on numerical predictions. Piezocone is modelled using an analytical rigid surface ignoring any deformations of the piezocone during the penetration.

The piezocone-soil interface is modelled using the contact algorithm in ABAQUS/Standard, where the piezocone is defined as the master surface and the soil surface in contact with the piezocone is defined as the slave surface. The contact formulation will not allow slave surface to penetrate into the master surface but the master surface can penetrate the slave surface. The friction between the two surfaces is modelled using an isotropic Coulomb model, where a friction coefficient and a stress limit is introduced to control the finite-sliding between the master and slave surfaces. The shear stress transmitted between the two surfaces is computed by multiplying the normal force across the interface by the friction coefficient between the surfaces. The limiting shear stress, τ_{max}, at the piezocone-soil interface is a function of undrained shear strength of the soil, s_u, and is given by,

$$\tau_{\max} = \frac{2s_u}{\sqrt{3}} \qquad [1]$$

The contact between master and slave surfaces are established using a contact search algorithm as described by Liyanapathirana (2009). When the contact between piezocone-soil interface is smooth, the total vertical reaction at the reference point is equal to the vertical force applied over the conical tip of the piezocone and it can be used to compute the tip resistance, q_c. In other cases, where the friction coefficient is non-zero, vertical and horizontal components of contact shear and normal forces can be used to evaluate q_c and shaft resistance, f_s.

The numerical model has the ability to simulate piezocone penetration starting from the ground surface. Figure 2 shows the deformed mesh adjacent to the piezocone after a penetration of 13.5*D* and it shows clearly that the mesh distortions are not significant even though the depth of penetration is significantly large compared to the diameter of the piezocone.

3 VALIDATION OF THE NUMERICAL MODEL

In this section, the numerical model is validated using the centrifuge model piezocone penetration tests in normally consolidated kaoline reported by Randolph and Hope (2004). The piezocone used for the tests has a diameter of 10 mm. The cone resistance profiles were measured over a depth of 15 m after a steady penetration of 3 mm/s. In addition, they carried out dissipation tests after a steady penetration rate of 1 mm/s. The acceleration of the centrifuge during all tests was 100g. The properties of the kaoline used for the centrifuge tests are given in Table 1.

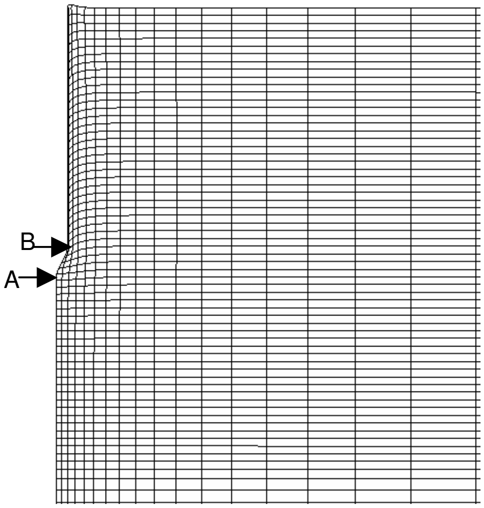

Figure 2. Deformed Finite element mesh at penetration of 13.5*D*.

Figure 3 shows the penetration resistance from centrifuge tests reported by Randolph and Hope (2004) with the finite element results at 5 m, 7 m, 10 m, 12 m and 15 m depths over the soil deposit. Although numerical predictions are slightly higher than centrifuge results, the overall agreement between centrifuge data and finite element results are good. In Figure 4, the excess pore pressure generation behind the cone at point B (in Figure 2) is shown at different depths. Results from centrifuge tests, cavity expansion theory by Silva et al. (2006) and finite element model used in this study are shown. Cavity expansion results slightly deviates from centrifuge data at deeper depths but the finite element results agree well with the centrifuge data further confirming that the numerical model has the ability to simulate piezocone penetration.

Table 1. Properties of kaoline.

Property	Value
Soil particle density, G_s	2.6
Slope of critical state line, M	0.92
Slope of normal consolidation line, λ	0.205
Slope of swelling line, κ	0.044
Permeability, k (m/s)	$1.5\text{x}10^{-9}$
Critical state voids ratio at 1 kPa, e_{cs}	2.14
Coefficient of earth pressure at rest, K_o	0.69

Figure 5 shows the radial effective stress obtained from the cavity expansion theory results given by Silva et al. (2006) and the finite element model at point B. These values are obtained at the end of steady state penetration at each depth. Although cavity expansion approach assumes plane strain conditions and the analysis is carried out for a surface of unit thickness in the axial direction assuming only radial soil movement, the radial effective stresses from both cavity expansion theory and finite element results agree well.

Figures 6 (a) to (d) show the pore pressure dissipation data reported by Randolph and Hope (2004) based on centrifuge tests carried out at a penetration of 1 mm/s, cavity expansion theory results by Silva et al. (2006) and finite element results from this study. At all 4 depths considered for the analysis, finite element and cavity expansion theory results agree well. During the dissipation phase, excess pore pressures monotonically decays to zero and in Section 6 dissipation curves for normally consolidated and over consolidated soils will be discussed in detail.

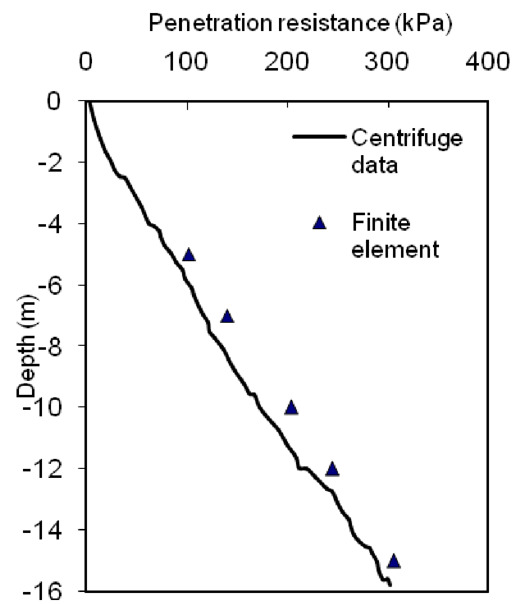

Figure 3. Comparison of undrained penetration resistance from finite element analysis and centrifuge tests.

At 5 m depth, centrifuge test data agree with theoretical predictions. At other depths, predicted dissipation curves follow the form of the model test results but there is a time lag. During consolidation, void ratio of the soil will decrease and this will result a decrease in the permeability of the soil. This effect has not been taken into account in the numerical simulation. Hence, the numerical simulation shows rapid pore pressure dissipation compared to the centrifuge model test.

4 INFLUENCE OF PENETRATION RATE ON CONE RESISTANCE

In this section the influence of penetration rate on cone resistance, q_c, will be investigated for normally consolidated kaoline. Figure 7 shows the variation of penetration resistance with penetration rate for normally concolidated kaoline when the penetration rate decreases from the standard penetration rate of 20 mm/s to 10^{-4}

mm/s. For the soil properties used for this analysis, penetration resistance has increased from 305 kPa to 420 kPa showing an increase in penetration resistance by 40%.

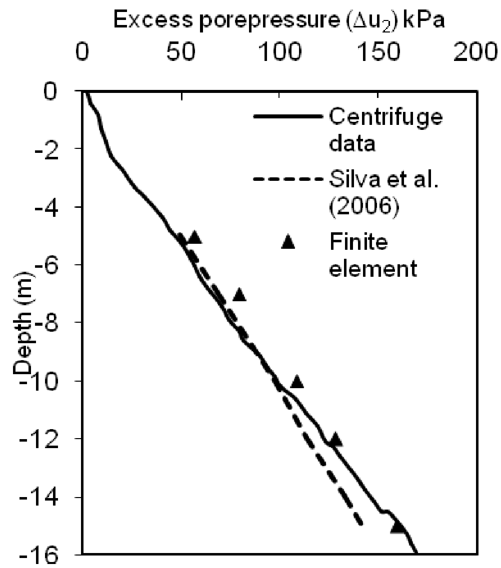

Figure 4. Comparison of excess pore pressures at the shoulder of the penetrometer from centrifuge tests, cavity expansion thery and finite element analysis.

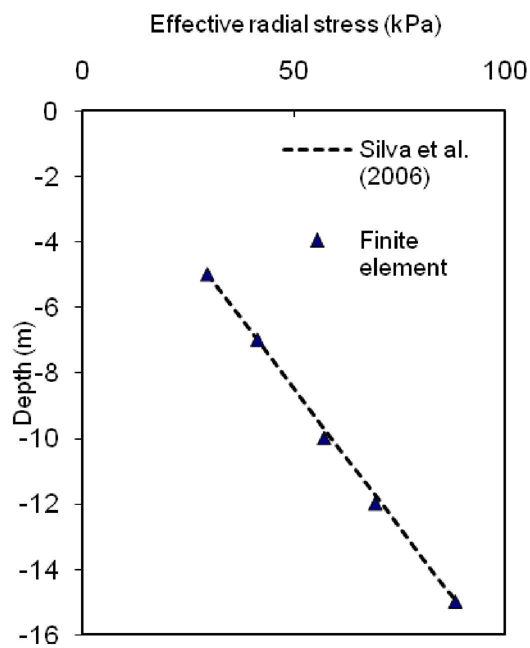

Figure 5. Effective radial stress from Cavilty expansion theory and finite element analysis.

Figure 8 shows the excess pore pressure generated behind the cone at the end of cone penetration up to 13.5D. The dimensionless penetration rate, V, in Figure 8 is given by (House et al., 2001),

$$V = \frac{vD}{c_v} \qquad [2]$$

Where v is the penetration rate, D is the diameter of the piezocone and c_v is the coefficient of consolidation. For the normally consolidated kaoline, c_v is 2.6 m^2/yr for the kaoline used for the simulations (House et al., 2001). When V = 0.2, generated excess pore pressure is negligible and essentially the piezocone penetrates under fully drained conditions. The penetration rate corresponding to $V = 0.2$ is 1.6×10^{-3} mm/s.

Figure 9 shows the variation of normalised penetration resistance with dimensionless penetration rate, V. The dimensionless penetration resistance is computed using q_{ref}, which is the penetration resistance under undrained conditions. This curve is known as the backbone resistance curve. The q_c/q_{ref} relevant for drained penetration obtained in this study are lower than those reported by Randolph and Hope (2004). This may be due to the smooth interface assumed between the piezocone and the soil. They observed q_c/q_{ref} values up to 2.5 when the penetration rate is corresponding to fully drained conditions. According to the finite element analysis this ratio reaches only up to 1.4. This needs to be further investigated by considering different friction coefficients at the interface between piezocone and the soil.

Both Figures 8 and 9 show that the transition between undrained to partially drained penetration occurs when V is around 30. The transition from partially drained to drained occurs when V is around 0.2. The piezocone penetration data presented by Kim et al. (2008) shows that V corresponding to transition from undrained to partially drained obtained from a backbone resistance curve such as Figure 9 and from a graph of $\Delta u/\Delta u_{max}$ vs V are different. It should be noted that in the the backbone resistance curves presented by Kim et el. (2008), penetration resistance is normalised using vertical stress, σ_v. The calibration chamber results presented by them shows that partially drained penetration occurs in the range $0.05 \leq V \leq 10$ based on the normalised excess pore pressure variation with V and $0.05 \leq V \leq 1$ based on the normalised penetration resistance backbone curve. The reason for this difference may be the influence of strain rate effects in field situations. With the increase in penetration rate, drainage condition surrounding the piezocone moves towards undrained conditions, which tends to lower the penetration resistance. On the other hand, strain rate effects tend to increase the penetration resistance (Liyanapathirana, 2009). Therefore in a field situation, pore pressure measurements will be a better indication to identify the velocity corresponding to transition between undrained and partially drained penetration or partially drained and drained penetration. In the numerical model presented in this paper, strain rate effects are not incorporated. Therefore both q_c/q_{ref} and $\Delta u/\Delta u_{max}$ give the same V for the transition points between undrained and partially drained penetration and drained and partially drained penetration.

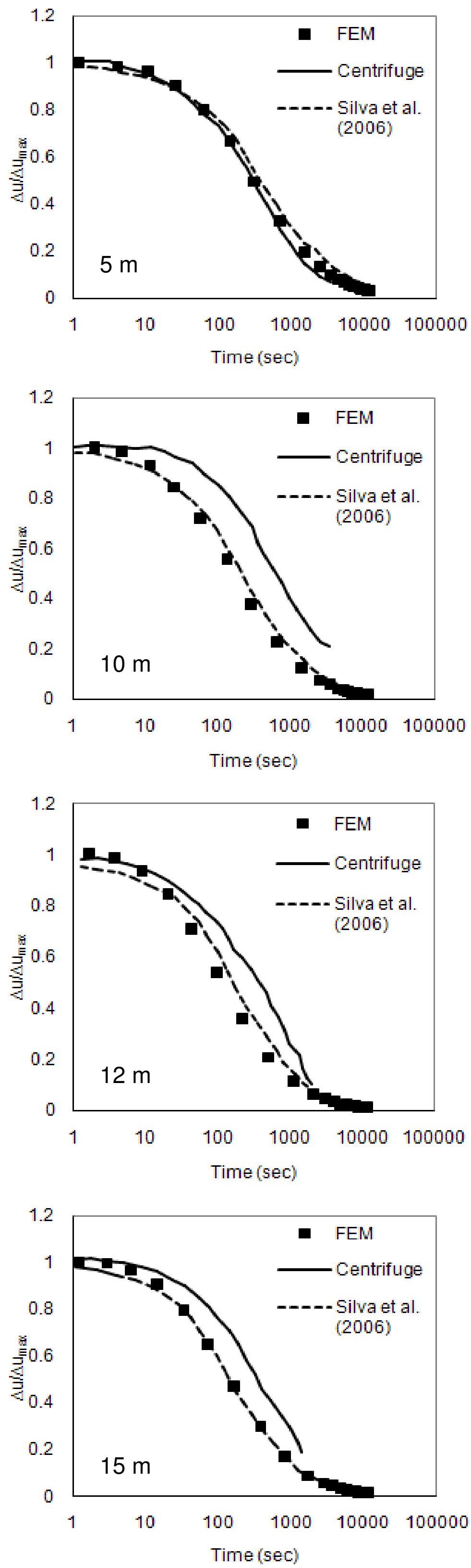

Figure 6. Pedicted and measured pore pressure dissipation curves at different depths.

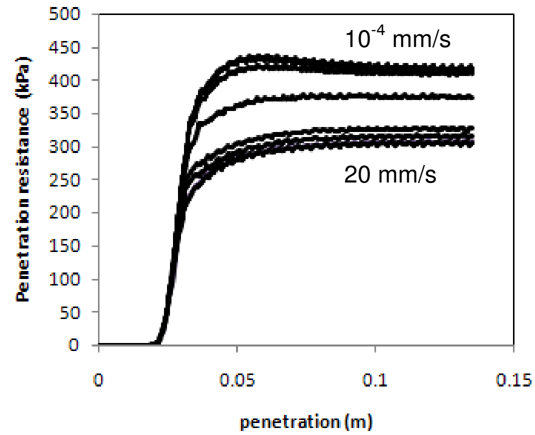

Figure 7. Variation of penetration resistance with penetration rate.

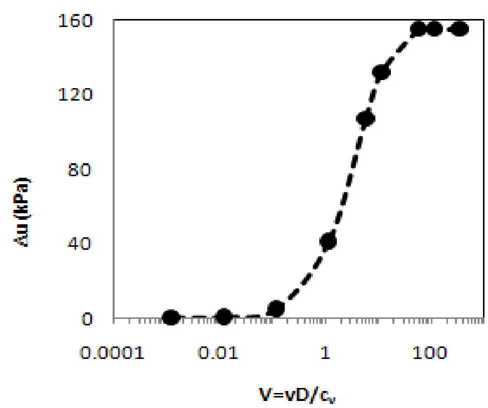

Figure 8. Variation of normalised excess pore pressure at the shoulder with normalised penetration rate at the end of steady state penetration (13.5D).

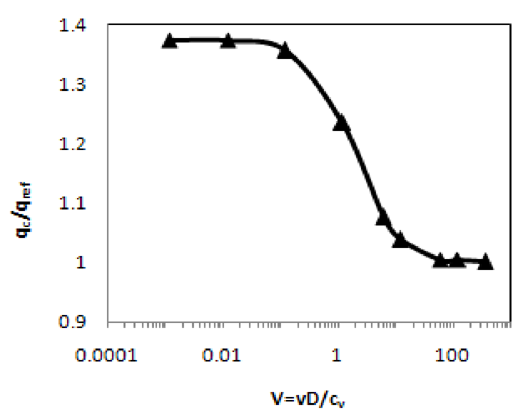

Figure 9. Variation of normalised penetration resistance with normalised penetration rate.

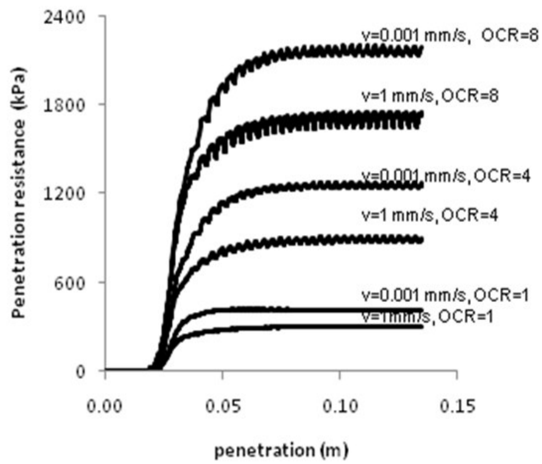

Figure 10.Variation of drained (v = 1 mm/s) and undrained (v = 1 mm/s) penetration resistance with OCR.

Results similar to this can be used to obtain limiting values of c_v for penetration to take place under drained or undrained conditions for a given set of values for cone diameter and rate of penetration. The V = 30 for the transition from undrained to partially drained corresponds to a c_v of 2.4×10^{-5} m^2/s for the standard penetration rate and a diameter of 35.77 mm. Therefore undrained q_c is expected with a standard cone at the standard penetration rate of of 20 mm/s when soils have c_v values less than 2.4×10^{-5} m^2/s for the kaolin used in this analysis.

The V = 0.2 for the transition from partially drained to fully drained corresponds to a c_v of 3.6×10^{-3} m^2/s for a standard cone at the standard penetration rate. Therefore fully drained penetration expected when c_v is greater than 3.6×10^{-3} m^2/s for the kaoline used for the finite element simulations.

5 EFFECT OF OVER CONSOLIDATION RATIO ON RATE EFFECTS

Figure 10 shows that variation of drained (v = 0.001 mm/s) and undrained (v = 1 mm/s) penetration resistance at different over consolidation ratios. It can be clearly seen that with the increase in OCR, both drained and undrained penetration resistances increase significantly.

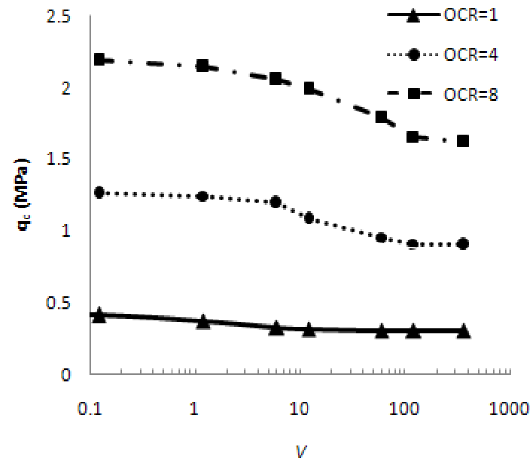

Figure 11. Variation of penetration resistance with OCR and penetration rate.

Figure 11 summarises the values of penetration resistance obtained for a range of penetration rates varying from 0.001 mm/s to 3 mm/s. This figure clearly shows that the transition from undrained to partially drained penetration and partially drained to fully drained penetration occurs at the same V for both normally consolidated and over consolidated kaoline.

6 DISSIPATION OF EXCESS PORE PRESSURE

Figures 12 and 13 present the dissipation curves for the excess pore pressure developed at the cone tip and the shoulder when v = 1 mm/s (undrained) and v = 0.05 mm/s (partially drained) respectively. The undrained penetration in normally consolidated soil shows identical dissipation curves at both tip and shoulder. At the end of piezocone penetration, excess pore pressure developed at the tip is slightly higher than that at the shoulder as shown in Figure 12 (a).

When the cone penetrates under partially drained conditions in normally consolidated clay or when the soil is overconsolidated for both undrained and partially drained penetration of the piezocone, the excess pore pressures developed at the cone tip are significantly higher than those around the shoulder. Also in over consolidated soils, at the end of piezocone penetration, pore pressures starts to increase around the shoulder during the dissipation stage as illustrated in Figures 12 (b) and 13 (b). This behaviour has been explained by Kim et al. (2007).

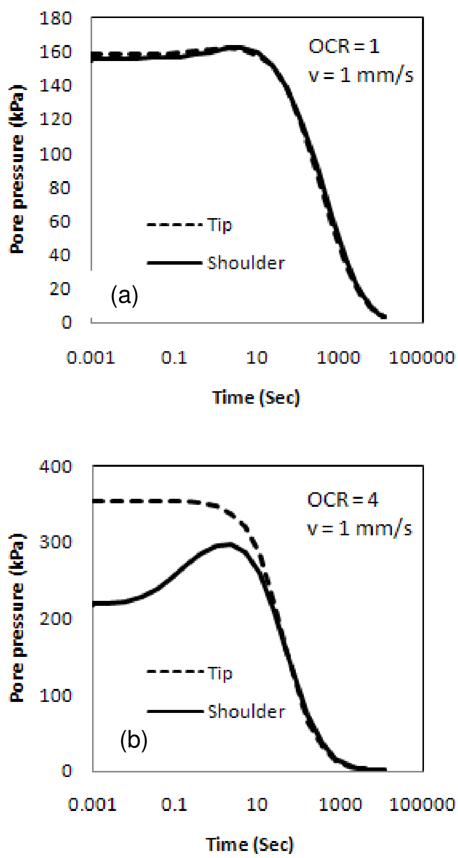

Figure 12. Excess pore pressure dissipation at the end of undrained penetration at a rate of 1 mm/s.

The total amount of excess pore pressure developed during piezocone penetration is the sum of Δu_{oct} due to changes in octahedral normal stress and Δu_{shear} due to changes in octahedral shear stress (Burns and Mayne, 1999). Due to the high compressive stresses induced during penetration, excess pore pressures induced by the normal stress is larger than that of the shear induced pore pressure under the piezocone tip and decays over a long time span. The shear induced excess pore pressures show different dissipation behaviours in normally consolidated and overconsolidated clays. In normally consolidated clays, the shear induced excess pore pressures are considerably lower in magnitude than normal induced excess pore pressures and decays much more rapidly (Burns and Mayne, 1999). In overconsolidated clays, the shear induced excess pore pressures are negative and increase to zero over a short period.

According to Figures 12 (a) and 13 (b), in normally consolidated soil, excess pore pressures decays monotonically at both tip and shoulder. Hence it can be concluded that octahedral normal stresses dominate around both tip and shoulder. In overconsolidated soils, excess pore pressures monotonically decrease only at the cone tip showing that octahedral normal stresses dominate only around the cone tip. However at the shoulder, octahedral shear stresses dominates and hence resulting excess pore pressure distribution during the dissipation phase increases to a peak showing dilatory behaviour and then starts to decay back to the hydrostatic pore pressure.

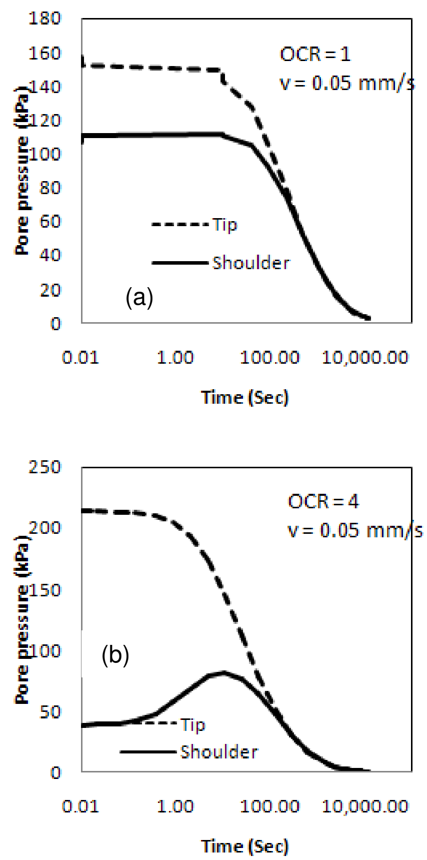

Figure 13. Excess pore pressure dissipation at the end of undrained penetration at a rate of 1 mm/s.

7 CONCLUSION

A numerical model based on the coupled theory of nonlinear porous media within a critical state framework has been outlined to simulate piezocone penetration in soft clay. The model has the ability to simulate piezocone penetration from the ground surface. Predictions from the numerical model closely agrees with the centrifuge model test data carried out in kaoline and cavity expansion theory. The model has been used to evaluate and quantify the influence of penetration rate on penetration resistance in both normally consolidated and overconsolidated clay. It was observed that with the decrease in penetration rate, drainage condition around the piezocone changes from undrained to drained increasing the penetration resistance. The transition from undrained to partially drained condition occurred when $V \approx 30$ and transition from partially drained to fully drained occurred when $V \approx 0.2$. Based on these limiting V values, we can obtain the limiting values of c_v for fully drained or fully undrained penetration for a given cone and a penetration rate.

Although with the increase in OCR, penetration resistance increases significantly irrespective of penetration rate, the limiting V values do not change with OCR.

The excess pore pressure dissipation curves infer that around the cone tip excess pore pressure generation is due to changes in octahedral normal stresses for both normally consolidated and overconsolidated soils. Also excess pore pressures at cone tip is higher than those at the shoulder. At the shoulder, pore pressure generation is dominated by octahedral normal stresses only when the soil is normally consolidated. In overconsolidated soils, induced excess pore pressures are dominated by changes in octahedral shear stresses. Therefore in the dissipation phase, pore pressure increases up to a certain level and then starts to decrease monotonically.

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REFERENCES

Burns, S.E. and Mayne, P.W. (1999). “Pore pressure dissipation behaviour surrounding driven piles and cone penetrometers.” *Transportation Research Record 1675*, Paper No. 99-0423, pp. 17-23.

House, A.R., Oliveira, J.R.M.S. and Randolph, M.F. (2001). “Evaluating the coefficient of consolidation using penetration tests.” *International Journal of Physical Modelling in Geotechnics*, Vol. 3, pp. 17-26.

Huang, W., Sheng, D., Sloan, S.W. and Yu, H.S. (2004). “Finite element analysis of cone penetration in cohesionless soil.” *Computers and Geotechnics*, Vol. 31, pp. 517-528.

Kim, D.K. (2004). “Effect of penetration rate and filter location on piezocone test results.” *KSCE Journal of Civil Engineering*, Vol. 8, No. 3, pp. 273-279.

Kim, K., Prezzi, M., Salgado, R. And Lee, W. (2008). “Effect of penetration rate on cone penetration resistance in saturated clayey soils.” *Journal of Geotechnical and Geoenvironmental Engineering*, ASCE, Vol. 134, No. 8, pp. 1142-1153.

Kim, T., Kim, N., Tumay, T. And Lee, W. (2007). “Spatial distribution of excess pore water pressure due to piezocone penetration in overconsolidated clay.” *Journal of Geotechnical and Geoenvironmental Engineering*, ASCE, Vol. 133, No. 6, pp. 674-683.

Liyanapathirana, D.S. (2008). “A numerical model for predicting pile setup in clay.” *GEOCONGRESS 2008: Characterisation, Monitoring and Modelling of Geosystems, Geotechnical Special Publication N0. 179*, pp. 710-717.

Liyanapathirana, D.S. (2009). “Arbitrary Lagrangian Eulerian based finite element analysis of cone penetration in soft clay.” *Computers and Geotechnics*, Vol. 35, No 5, pp. 851-860.

Randolph, M.F. and Hope, S.N. (2004). “Effect of cone velocity on cone resistance and excess pore pressure.” *Proceedings of the Engineering Practice and the Performance of soft deposits*, Osaka, pp. 147-152.

Roscoe, K.H. and Burland, J.B. (1968). “On the generalised stress-strain behaviour of wet clay.” *Engineering Plasticity*, Heyman, J. And Leckie, F.A. (eds), Cambridge University Press, Cambridge, pp. 535-609.

Silva, M.F., White, D.J. and Bolton, M.D. (2006). “An analytical study of the effect of penetration rate on piezocone tests in clay.” *International Journal for Numerical and Analytical Methods in Geomechanics*. Vol. 30, pp. 501-527.

Voyiadjis, G.Z. and Song, C.R. (2003). “Determination of hydraulic conductivity using piezocone penetration test. *International Journal of Geomechanics*, ASCE, Vol. 3, No. 2, pp. 217-224.