Limited-Tension Analysis of Socketed Piles

L'Analyse à Tension Limité des Pieux Forés

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SYNOPSIS Conventional elastic analyses of socketed piles produce plausible load-deformation curves and distributions of side shear stress, even though tensile stresses may be present in the surrounding medium and at the pile face. This paper presents finite element solutions, for a range of pile geometries and material properties, treating the rock or soil as a no-tension or limited-tension material. Results are given for both elastic and plastic stress ranges, with associated and non-associated flow rules.

INTRODUCTION

Existing solutions for pile load-deformation behaviour are based mainly on linear elasticity with no checks made on radial stresses at the pile - rock or soil interface or the signs of principal stresses in the surrounding medium. Several authors, including Coates and Yu (1970) and Osterberg and Gill (1973) have mentioned the existence of tensile zones in finite element solutions, but usually it has been assumed that these have an insignificant effect on pile behaviour or that overburden stresses will suppress them. Recent FE analyses by the authors have shown the existence of tensile principal stresses in the radial plane and also in the circumferential direction, and these agree qualitatively with crack patterns observed in field and model tests on socketed piles.

The distribution of shear stress down the shaft of a pile will also depend on the radial normal stress, and conventional analytic methods will show shear stresses even in regions where tensile stresses would cause a gap to open over a section of a straight shafted pile. In this paper the effects of redistributing tensile stresses are examined theoretically using a limited tension elasto-plastic finite element analysis.

METHOD OF ANALYSIS

The finite element method was used with 4-noded isoparametric elements and the programme details and performance are essentially as described in Donald et al. (1980), except for a provision to limit tensile stresses. This provision basically follows Zienkiewicz et al. (1968) and will not be described in the paper. However, it should be mentioned that in the plastic range the stresses were corrected for tension before being checked for yielding due to shear.

A constant stiffness approach was adopted as it has computational advantages, and only an additional 5-10 iterations were required over conventional analyses for convergence.

MATERIAL PROPERTIES

In all analyses the modulus of the pile, E_c was taken as 30 GPa with a Poisson's ratio, $\nu_c=0.25$. The pile is assumed to be always elastic with a tensile strength, σ_t equal to 2.5 MPa.

The rock mass is assumed to behave as a Mohr-Coulomb material with or without a tension cut-off. Yielding in shear follows a non-associated flow rule unless otherwise stated. The rock mass Poisson's ratio was taken as $\nu_m = 0.25$ for most calculations, with any departures indicated on the Figures.

ELASTIC ANALYSES

These have been performed for various pile length /diameter (embedment) ratios, L/D, and pile to rock mass modular ratios, E_c/E_m. No initial stresses in the rock mass have been assumed, and for the tension cut-off analyses the tensile strengths have been taken to be negligible. The no-tension analyses will thus give lower bound solutions.

In Figure 1 the normal stress, σ_r, at the pile rock interface (without a tension cut-off) has been plotted as a ratio of the applied stress, q. It can be seen that the depth at which σ_r is first tensile decreases with decreasing L/D. With a higher ν_m this zone of tensile normal stresses increases as shown in Figure 2 and indicated by Donald et al. (1980). Principal tensile stresses also occur in the rock mass adjacent to the interface.

If the pile-rock interface elements cannot sustain a tensile stress and the rock mass has only limited tensile strength or is jointed, it is necessary for these tensile stresses to be redistributed to a state of equilibrium. Figures 3 and 4 illustrate the effects of this stress redistribution. The increase in pile settlement for an analysis with a tension cut-off can be 12% to 35% of a conventional (without tension cut-off) analysis, depending on E_c/E_m and L/D.

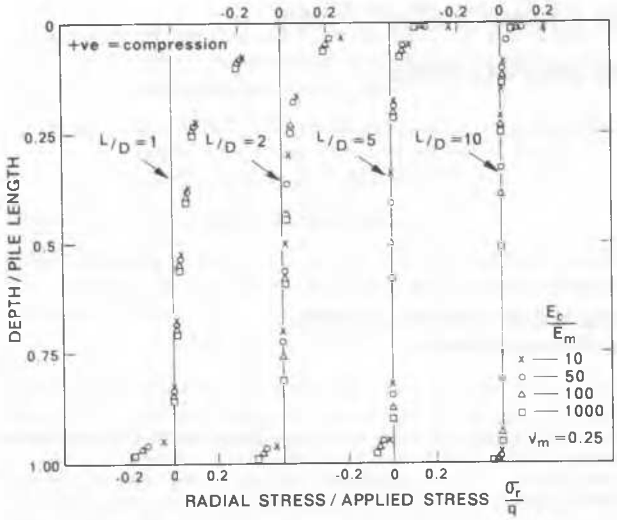

Fig. 1 Normal Stress at Pile-Rock Interface

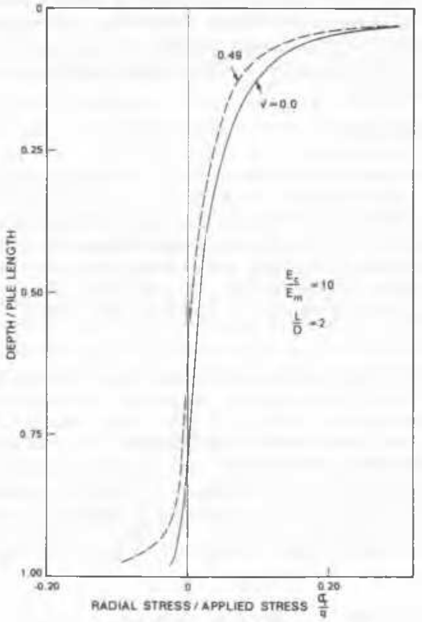

Fig. 2 Effect of ν_m on Tensile Stress

The percentage of load taken by the base also increased in all cases when tensile stresses were eliminated. These results present the limiting case for an asperity-free pile/rock interface with zero tensile strength. For rough sockets (i.e. with asperities) some intermediate condition would be applicable, and for sockets under a high overburden pressure net tensile stresses could not occur.

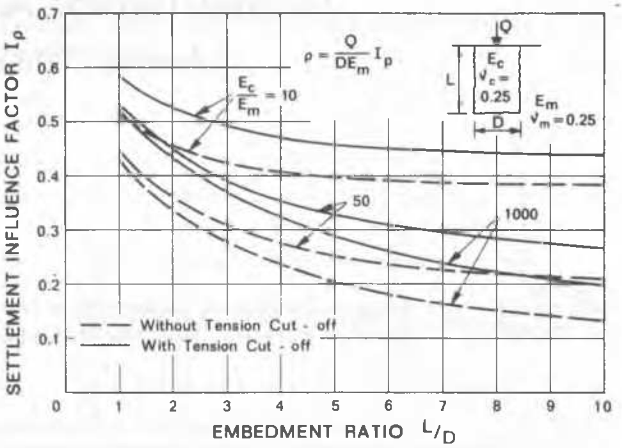

Fig. 3 Settlement Influence Factors

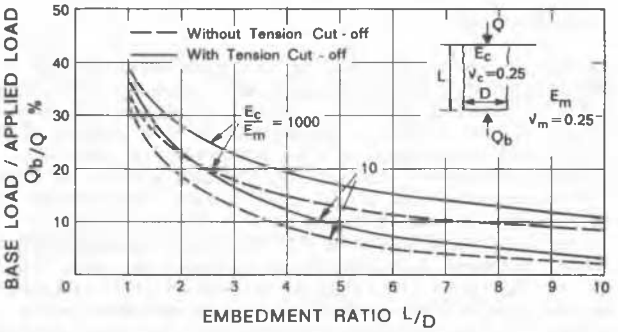

Fig. 4 Base Load Percentage

ELASTO-PLASTIC ANALYSES

Three hypothetical piles of 1m dia. and L/D ratios of 10, 5 and 2 (Piles 1, 2 and 3 respectively), socketed into Melbourne mudstone of various degrees of weathering (Johnston et al. (1980)), were analysed for the effects of limited tension in the plastic range. Field evidence has shown that many working sockets are stressed beyond the elastic limit.

Highly to completely weathered rock - $\phi=0$ case

Melbourne mudstone classified under this weathering zone has properties much like a silty clay or a clayey silt. When a significant proportion of the load supported by a pile socketed in such a rock mass is live loading, undrained behaviour predominates with a small to zero angle of friction (Chiu and Johnston, (1980)). The material has negligible tensile strength, and initial geostatic gravity stresses have been assumed.

An examination of Figure 5 shows little difference in the load settlement behaviour of Pile 1 analysed with both approaches up to an applied surface stress of about 25 times the undrained cohesion of the material. This is not surprising as the assumed gravity stresses are sufficient to suppress most of the induced tensile stresses. However, at higher loads the result of assigning zero tensile strength to the rock was to increase the settlement of the pile. When yielded zones were plotted (Fig. 6) it was evident that the limited tension condition had resulted in a greater number of elements failing - hence the increase in settlement.

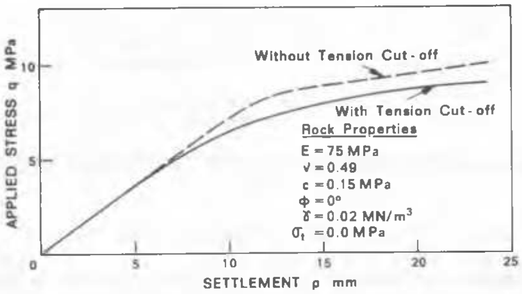

Fig. 5 Load-Settlement Curves, Pile 1, L/D = 10

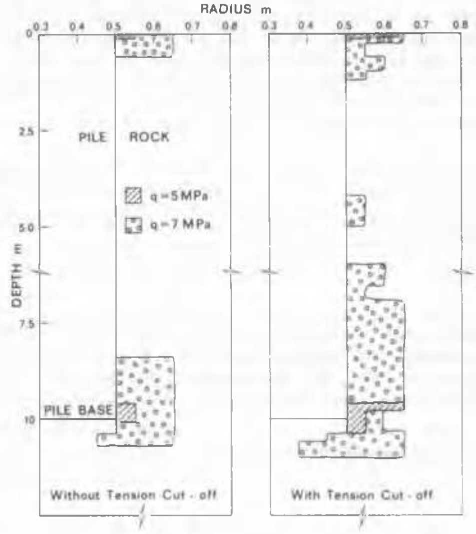

Fig. 6 Yielded Zones - Pile 1

It is worth noting that the above analyses are also relevant to piles socketed in clay as the E_m used is in the stiff clay range.

Slightly to moderately weathered rock - c-ϕ case

Two piles were analysed assuming c-ϕ behaviour. Pile 2 is in moderately weathered mudstone while Pile 3 is in the slightly weathered zone rock. They represent piles carrying a large percentage of dead load and as explained in Chiu and Johnston (1980) drained parameters are applicable. Initial stresses have been assumed as in Pile 1 and tensile strengths, σ_t, were taken to be one-tenth the unconfined compressive strengths. Results are presented in Figures 7-11. Figure 7 shows principal tensile stresses for Pile 3 if a tension cut-off is not provided. It can be observed that the greatest tension occurs near the pile-rock interface extending across up to 2 pile diameters. Tensile stresses in the rock below the pile base are less widespread.

Figures 8 and 9 shows the load settlement response of Piles 2 and 3 with and without tension cut-off analysis and for both non-associated and associated flow rules. For the non-associated flow rule the behaviour is the reverse of that observed for the ϕ=0 case, and the redistribution of tensile stresses shifts the Mohr stress circle to the compressive side which, for a c-ϕ material, implies an increase in shear strength and a reduction in the extent of failed zones. In the elastic range however, the limited tension analysis gives increased deformations as previously. Analyses for the same piles, but using an associated flow rule with its large implied dilation in the plastic range, give much stiffer behaviour over the whole load range, with the limited tension case giving slightly larger deformations. The dilation during shear naturally suppresses the development of tensile stresses, as is reflected in the graphs.

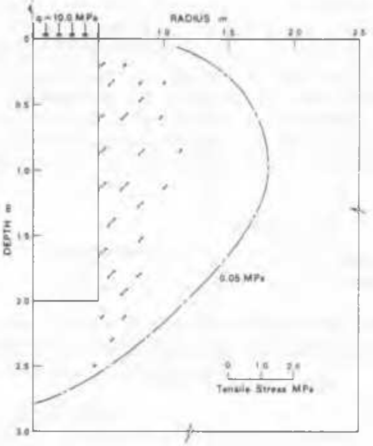

Fig. 7 Principal Tensile Stresses-Pile 3,L/D=2

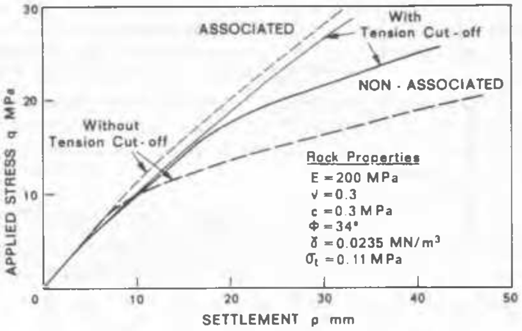

Fig. 8 Load-Settlement Curves, Pile 2,L/D=5

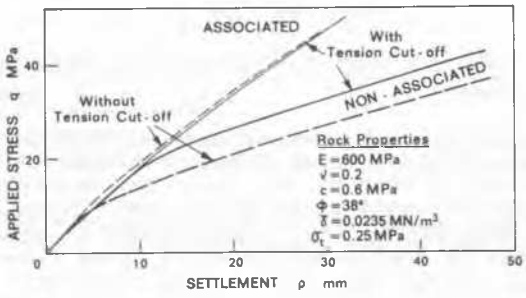

Fig. 9 Load-Settlement Curves, Pile 3,L/D=2

Figures 10 and 11 show the distributions of radial and shear stress on the shaft for Pile 3. Radial stresses for the analysis with tension cut-off, and either flow rule, are more in agreement with measured field stresses (Williams et al., (1980)) than analyses permitting tension, and the shear stress distribution is more uniform over the central section of the pile. As shown in Fig. 12 the percentage of the total load carried by the base is reduced in the plastic range by the inclusion of limited tension in the analysis, and for an associated flow rule it remains almost constant.

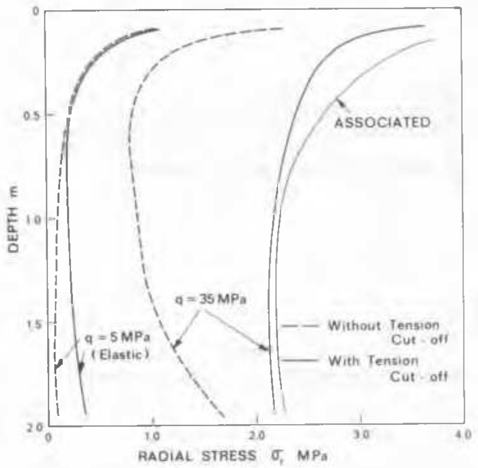

Fig. 10 Radial Stress on Shaft, Pile 3

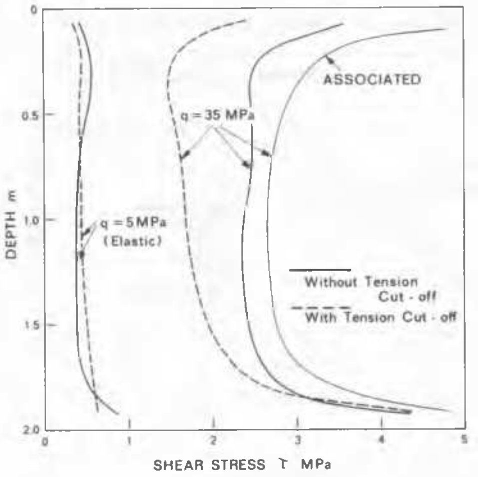

Fig. 11 Shear Stress on Shaft, Pile 3

CONCLUSIONS

The inclusion of a limit on tensile strength of the rock or soil medium in pile analyses has a significant effect on both the elastic and plastic load deformation curves, the differences being more marked when a non-associated flow rule is used in the plastic range.

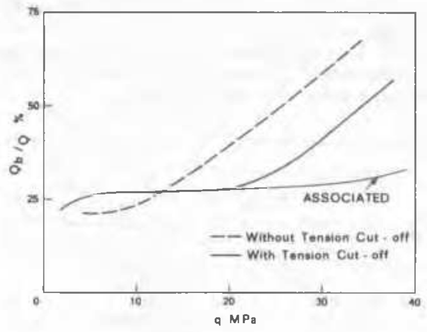

Fig. 12 Base Load Percentage, Pile 3

The curves presented are for the limiting cases of associated and non-associated flow rules, and asperity-free pile-medium interface. Real situations will require an assessment of the relevance of the flow rules to measured shear dilation behaviour, and for some circumstances, such as elastic behaviour at low overburden stress levels where radial tensile stresses can occur at the pile-rock interface, the influence of interface roughness could be significant.

Solutions in the elastic range have been presented in chart form for settlement influence factor and base load percentage as functions of embedment ratio, $\frac{L}{D}$.

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