

The behaviour of a propped retaining wall: results of a numerical experiment

D. M. POTTS and A. B. FOURIE (1984). *Géotechnique* **34**, No. 3, 383–404

Dr J. S. Steenfelt, *Technical University of Denmark*

In their Paper the Authors conclude that the results of their numerical experiment have serious practical implications for walls embedded in stiff overconsolidated clays with high K_0 values. However, the calculations (published) are based on only one set of strength parameters, $c' = 0$ and $\phi' = 25^\circ$ (plane strain parameters?) which seems to be incompatible with the modelling of stiff overconsolidated clays with varying K_0 values.

In the Writer's opinion the wide range of design values for the prop force and the maximum wall bending moment together with the maximum depth of excavation/retained height

could equally well be attributed to a variation in the strength parameters c' , ϕ' . For increasing values of K_0 and overconsolidation ratio increasing values of c' would be expected and thus it seems a rather strong limitation to consider only $c' = 0$, unless only the ultimate critical state is modelled.

Using the Danish design method for retaining walls (Dansk Ingeniørforening, 1978; Brinch Hansen, 1953) the design values of the prop force F , the maximum wall bending moment $M_{\max}$ at depth x and the maximum depth of excavation/retained height h_2 have been calculated for ϕ' values of 25° , 27.5° and 30° and c' values of 0 and 10 kN/m^2 . The results are shown in Fig. 1 and

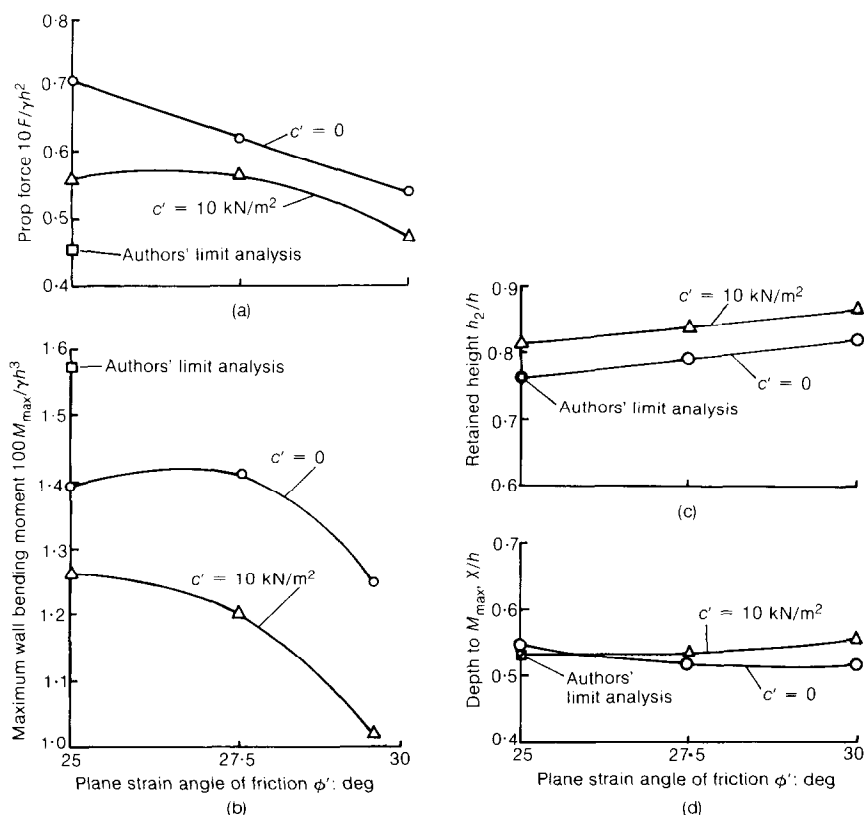


Fig. 1. Effect of strength parameters ϕ' and c' on the design values for the propped wall in the Authors' paper

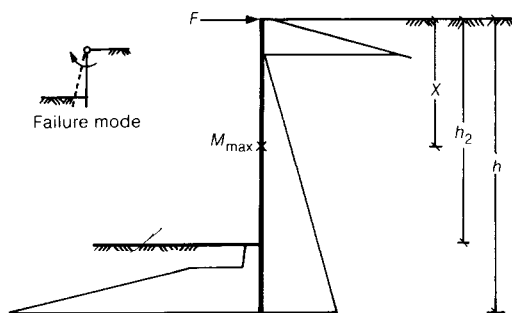


Fig. 2. Schematic earth pressure diagram for $\phi' = 25^\circ$ and $c' = 10 \text{ kN/m}^2$ using the Danish design method for the propped wall in the Authors' paper

the corresponding earth pressure diagram is shown schematically in Fig. 2.

The calculations are based on the same assumptions used in the Paper, i.e. $h = 20 \text{ m}$ and $\gamma = 20 \text{ kN/m}^3$, but for each set of strength parameters the maximum retained height h_2 is found corresponding to a safety factor $F_r = 1$. The difference between the present solution and the Authors' limit analysis solution is due to the difference in the applied earth pressure distributions (and earth pressure coefficients).

Accepting the Writer's values as the result of a similar numerical experiment, it is seen that the values of both c' and ϕ' significantly influence the design parameters. Since a range of ϕ' of $25\text{--}35^\circ$ and of c' of $0\text{--}50 \text{ kN/m}^2$ can be expected for stiff overconsolidated clays a wide range of design parameters will be encountered.

The results of the Authors' investigation have practical implications for diaphragm or secant pile walls in stiff overconsolidated clays in the sense that when high K_0 values are encountered this should be reflected by the applied design strength parameters.

The advantage of the type of calculation performed by the Authors lies primarily in the possibility of predicting the displacements and corresponding stresses under working conditions, which is impossible by limit analysis type of calculations.

REFERENCES

- Brinch Hansen, J. (1953). *Earth pressure calculation*. Copenhagen: Danish Technical Press.
- Dansk Ingeniørforenings (1978). *Code of practice for foundation engineering* (English translation). Danish Geotechnical Institute Bulletin No. 32, Copenhagen.

Authors' reply

As noted in the introduction to our Paper the objective was to investigate the effects of the method of wall construction (backfilled or excavated) and the magnitude of the initial hori-

zontal stress ($\sigma_h' = K_0 \sigma_v'$) on the behaviour of a propped retaining wall. The wall was 1 m thick and was assumed to be made of concrete and thus resembled a typical diaphragm or secant pile wall. To limit the number of variables and therefore to allow meaningful comparisons to be made only one set of soil strength parameters was used, namely $c' = 0$ and $\phi' = 25^\circ$. The effects of introducing a c' value of 10 kPa were also discussed, however.

One of the conclusions of the Paper was that for excavated walls in high K_0 soils the prop forces and bending moments from the finite element analyses greatly exceeded those predicted by a typical limit equilibrium design approach. It is this inability of the simple limit equilibrium calculation to predict the result of an idealized numerical experiment accurately, based on the same strength parameters, that is worrying and may have serious practical implications. If an approximate calculation procedure cannot predict a more refined one what chance has it of accurately predicting reality?

Dr Steenfelt has discussed the prediction of limiting values of prop force and bending moments and has indicated that differences will arise as a result of changes in strength parameters. Clearly this must be true and we appreciate that in overconsolidated soils, which usually are associated with high K_0 values, the magnitude of c' may be important. However, whatever strength parameters are selected the conclusion of our Paper still stands, namely that for excavated walls in high K_0 soils limit equilibrium type of calculations underestimate prop forces and bending moments.

A comparison of the values of prop force and bending moments given by Dr Steenfelt with those presented in Tables 1–3 of the Paper indicates that the former greatly underestimate those predicted for excavated walls in high K_0 soils.

In a recent Technical Note (Potts & Fourie, 1985) the effect of wall stiffness was considered. Several wall stiffnesses were investigated including those appropriate to sheet pile and concrete walls. It is shown that for flexible walls, of the sheet pile type, the finite element predictions indicate prop forces and bending moments that are smaller than those given by limit equilibrium type of calculations. These results are independent of the K_0 value and are in agreement with the experimental findings of Rowe (1952). Limit equilibrium type of approaches are therefore conservative for sheet pile walls. It is only for stiff excavated walls in high K_0 soils that our theoretical studies indicate that the simple limit equilibrium type of design approaches may not be conservative.

As noted by Dr Steenfelt the finite element approach is useful in predicting behaviour under working conditions. In addition, we believe that the approach is also a more accurate way of investigating limiting conditions. With limit equilibrium or simple plasticity solutions, assumptions about the magnitude and distribution of earth pressures or mechanisms of failure must be made. This is not so with the finite element method which predicts these as a result of the analyses. In a forthcoming paper (Potts & Fourie, 1986) the effects of different modes of wall deformation on the generation of earth pressures are investigated. It is concluded that the nature of the wall displacement has a large influence on the earth pressure distributions and consequently on prop forces and bending moments. These effects are not considered in conventional limit equi-

librium approaches. In addition the finite element method predicts realistic deflected wall profiles (if realistic wall stiffness values are used) as opposed to the simplified assumptions of rigid body rotation that are used in conventional analytical methods, e.g. the inset to Fig. 2 of Dr Steenfelt's discussion.

REFERENCES

- Potts, D. M. & Fourie, A. B. (1985). The effect of wall stiffness on the behaviour of a propped retaining wall. *Géotechnique* **35**, No. 3, 347–352.
- Potts, D. M. & Fourie, A. B. (1986). A numerical study of the effects of wall deformation on earth pressure. *Int. J. Numer. Analyt. Meth. Geomech.*, to be published.
- Rowe, P. W. (1952). Anchored sheet-pile walls. *Proc. Instn Civ. Engrs*, Part 1 **1**, 27–70.