International Conference on Structural and Foundation Failures
August 2-4, 2004, Singapore

AVOIDING EXCESSIVE DISPLACEMENTS: A NEW DESIGN APPROACH FOR RETAINING WALLS

A. S. Osman and M.D. Bolton
Department of Engineering, Cambridge University, UK,
aseko2@cam.ac.uk , mdb@eng.cam.ac.uk

Abstract

The Mobilisable Strength Design (MSD) method is a new design method based on the theory of plasticity and the concept of “mobilisable soil strength”. This method offers a simple unified design methodology, which can satisfy both safety and serviceability a single step of calculation. The possible use of the MSD method in the design of retaining walls supporting excavations in clay is explored and illustrated.

.

Keywords: Plasticity theory, Shallow foundations, Settlement.

Introduction

During recent decades, there has been an increasing use of retaining walls for basements and underground roads, for example. The need for construction of excavations in urban areas requires control of the surrounding ground surface since excessive ground movements damage adjacent properties. The characteristics of ground movements and wall deformations need to be understood in order to protect the adjacent properties. In the geotechnical design of retaining walls, limiting values of displacements or strains should be specified to define serviceability in terms of just acceptable conditions. Unnecessarily severe restrictions may lead to uneconomic design. Therefore, an accurate prediction of displacements under working conditions is required. The design criteria for retaining walls should also allow for the penetration depth to be sufficient for stability and for the wall to be strong enough to withstand the maximum bending moment.

In current design practice, there is a distinction between calculations for safety requirements and calculations for displacements. Plasticity theory is used in collapse calculations while elasticity theory is used to predict displacements. However, the stresses under working conditions are far from those obtained by plasticity theory, which predicts stresses at failure. The applications of elasticity theory are often complex and are based on an arbitrary equivalent modulus. Codes of practice do not deal with serviceability in any great depth (Simpson and Driscoll 1998). Factors of safety are introduced to make allowance for uncertainty in design values and to safeguard against deformation by factoring down the peak soil strength. However, there are different definitions and rules for selecting safety factors in design codes. Most of these definitions have shortcomings and fail to address the real nature of the soil, which always shows a non-linear and sometimes brittle response.

Therefore, there is a need for a simple unified design approach, which could relate successfully the real nature of serviceability and collapse limits to the soil behaviour. Bolton *et al.* (1990a, 1990b) and Osman and Bolton (2004) proposed a new approach based on the theory of plasticity accompanied by the introduction of the concept of “mobilisable soil strength”. The proposed design method treats a stress path in a representative soil zone as a curve of plastic soil strength mobilised as strains develop. Strains are entered into a simple plastic deformation mechanism to predict boundary displacements. Stresses are entered into simple equilibrium diagrams to demonstrate stability. Hence, the proposed Mobilisable Strength Design (MSD) method might satisfy both safety and serviceability in a single step of calculation.

The MSD method like any other design method idealises the soil behaviour. Therefore, the successful application of this method in design practice relies on the appropriate selection of simplified mechanisms and the identifications of representative soil elements. The possible use of MSD for retaining walls is the subject of this paper.

On the left hand side of Fig. 1 appears the familiar plastic equilibrium solution for cohesive material around retaining walls. On the right hand side of the figure is shown corresponding plastic deformation solutions. The 45° wedges around the cantilever wall contain soil all of which shear to the extent $\gamma_{mob} = 2\delta\theta$ when the wall rotates by small angle$\delta\theta$. The soil in far field is treated as rigid. The combination of the statical and kinematical approaches can offer a simple design method. The following assumptions can be made: active and passive zones in the two approaches correspond, and the mobilisation of a uniform plastic strength c_{mob} is consistent with the development of a uniform plastic shear strain γ_{mob}. If the wall height (D), the excavation height (H) and the bulk unit weight of soil are known, the required strength and pivot position can be determined by solving the equations of force and moment equilibrium. Fig. 2 shows normalised undrained shear stress ($c_{mob}/\gamma D$) mobilised for different excavation height ratios (H/D) for an embedded cantilever retaining wall to achieve equilibrium. The theoretical analysis shows that the maximum height of the pivot point to the total height of the wall (r/D) is less than 2.5%.

Design procedure

The following assumptions have been made in the MSD method:

- The soil is laterally homogenous and vertically consistent, although it may have a vertical profile of strength and stiffness dictated by variable overconsolidation ratios.

- The average shear stress induced in the zone of plastic deformation is deduced from a standard bearing capacity coefficient applied to estimated working loads.

- The displacements are controlled by the average soil stiffness in the zone of the deformation, through the assumption of a plastic deformation mechanism.

- A representative stress-strain curve for soil at mid-height of the retaining wall prior to excavation can be used to deduce the average shear strain in the zone of plastic deformation.

Fig. 3 illustrates a possible design procedure for a retaining wall supporting an excavation in clay.

Validation

Finite Element (FE) analysis

The validity of the assumptions used in the MSD method is determined by comparing its predictions with finite element results. The FE analysis was carried out using ABAQUS finite element software (Hibbit, Karlsson & Sorensen Inc. 2001). The FE mesh is shown in Fig. 4. In the finite element simulation, the Strain Dependent Modified Cam Clay (SDMCC) soil model (Dasari 1996) was used. This model can simulate the variation of stiffness with strain and the development of non-linearity inside the yield surface (Fig. 5), in addition to the effects of recent stress history. These behaviours are of prime importance in the modelling of retaining structures (Bolton and Sun 1991).

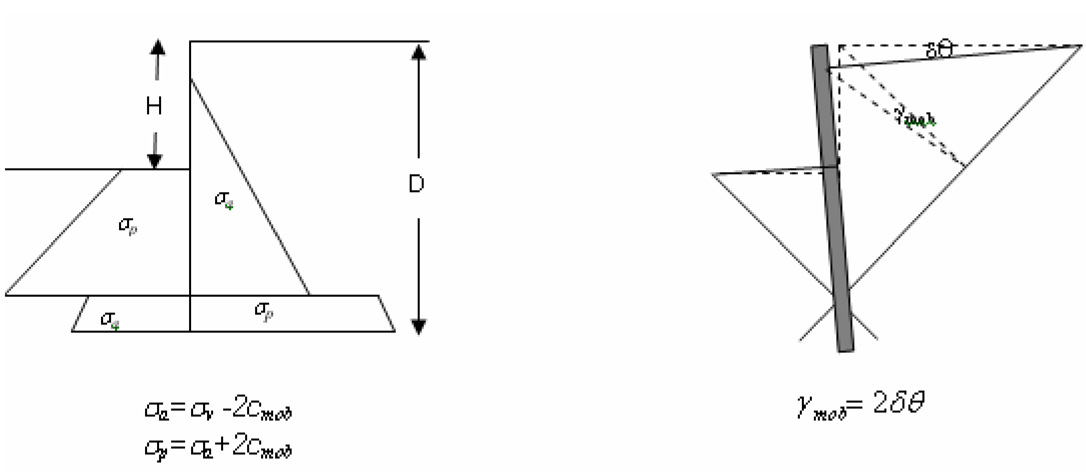
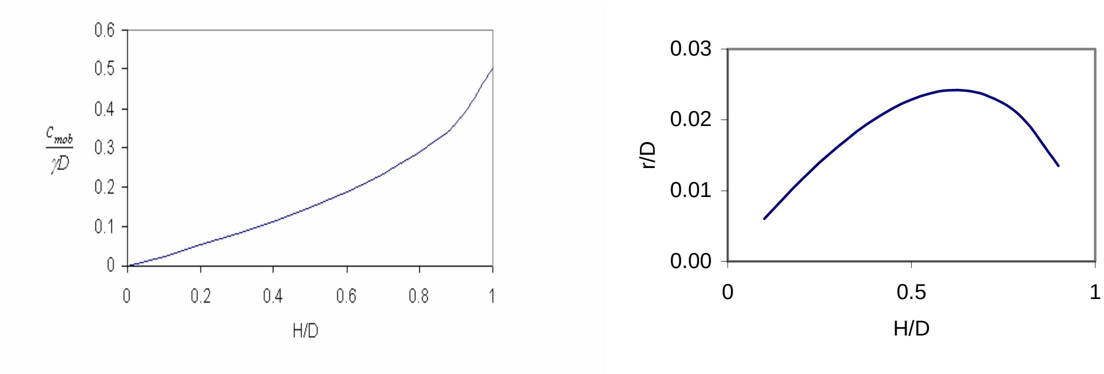


Figure1 Plastic deformation mechanism



(a) (b)

Figure 2 Equilibrium of forces on MSD method: (a) mobilised strength versus excavation height (b) height of rotation point.

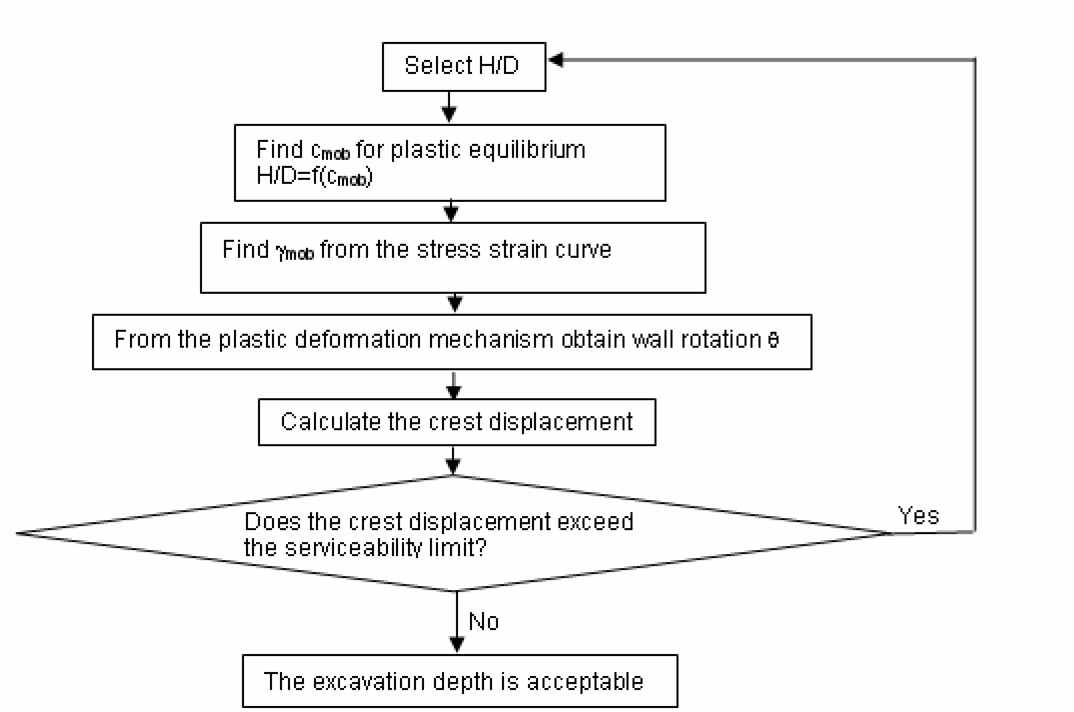


Figure 3 Design procedure

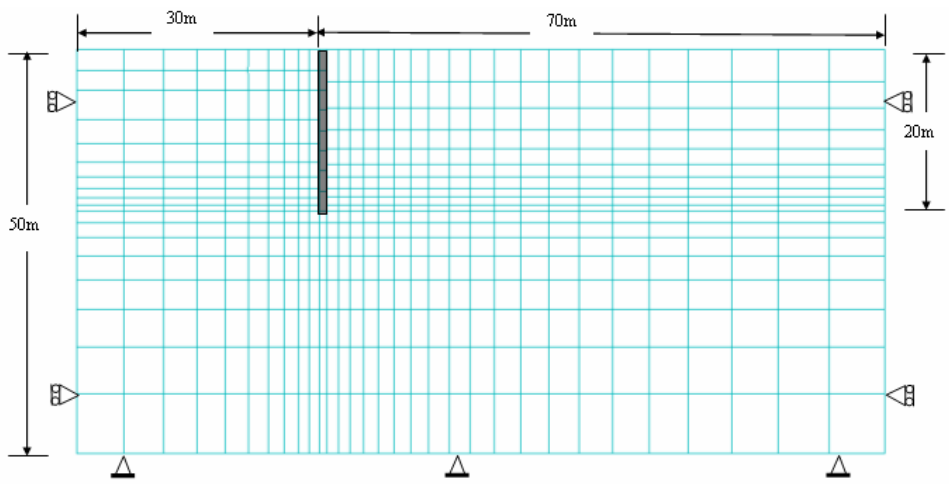


Figure 4 FE mesh

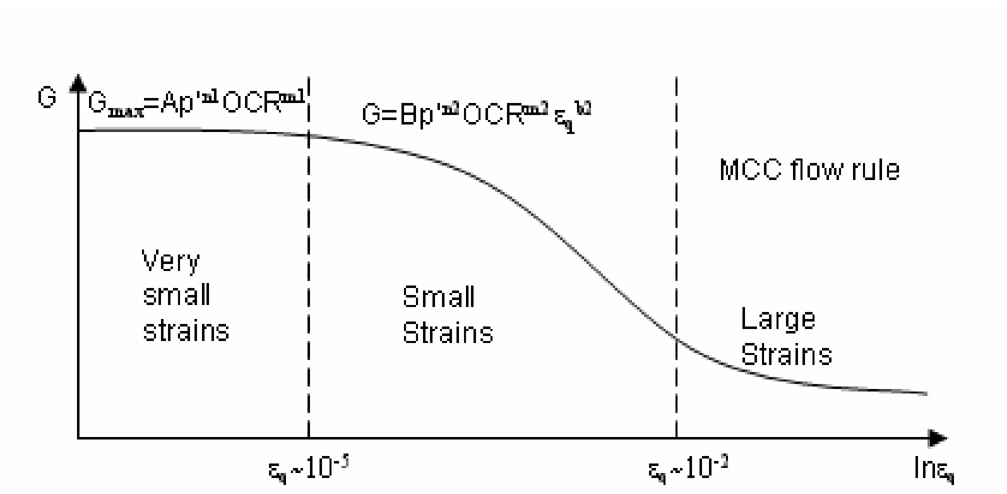


Figure 5 Typical stiffness-strain relationship in the SDMCC model

Comparison of the results

The impact of the various parameters that influence wall movements in the short-term was studied. The displacements of the crest of the wall calculated by the MSD method (Δ_{MSD}) are normalised by the FE displacements (Δ_{FE}) and are related to wall flexibility for different in-situ lateral earth pressure coefficient (K_o), different shapes of soil stress-strain curve, and different excavation ratios defined as the excavated depth divided by the overall height of the wall.

The wall flexibility can be characterized by the non-dimensional group (R), which was introduced by Rowe (1955). R is defined as relative soil/structure stiffness and is given by:

$$R = \frac{mH^4}{EI} \tag{1}$$

where *H* is height of the wall, and *EI* is the bending stiffness per unit width. The parameter *m* can be defined as the rate of change of the shear modulus with depth (Powrie and Li 1991). Fig. 6 summarises the relation between MSD predictions and FE calculations of the displacements for various excavation depths. Fig. 6 shows also different representative shapes of stress-strain curve for samples extracted from the same in-situ conditions with K_o=1.0. Curve A exhibits smaller strain to failure, while C and D show larger strain to failure, but all share the same maximum shear modulus (G_{max}).

Evidently, MSD predictions are most accurate when the soil stiffness deteriorates most markedly. Fig. 6 shows that for the whole range of wall flexibilities, initial earth pressure coefficients and shapes of stress-strain curves, studied here, the MSD predictions underestimate FE analyses, but generally by a factor of not more than 2. If Fig. 6 is used as a guide in decision-making, the designer should expect the MSD estimates to be much closer to the "correct" FE solution. The designer must, of course, decide in which situations MSD predictions alone can be accepted, and when FE solution must be obtained as well.

Two significant uncertainties will hamper the decision that the designer must make regarding the limit to be placed on wall movement or soil strain. Designers should first realise that criteria for limiting strains to prevent damage in different classes of structure are rather approximate (Burland and Wroth 1975). The actual condition of the existing building or services which are partly located in the zone of influence of a new excavation will also be open to doubt. These inevitable uncertainties may lead the designer to the conclusion that even a factor 2 error in the MSD calculations can be tolerated.

Fig. 7 shows lateral stress distribution and bending moment distribution for different excavation height. This figure shows that the calculated values using the MSD method are in good agreements with FE. Curve B of Fig. 6 represents the stress-strain curve that was used in the comparison shown in Fig. 7. The following example illustrates the MSD method calculations. Suppose a rigid wall of height (D) 20 m supports a retained height (H) of 5m. Then:

H/D=5/20=0.25

The bulk unit weight of the soil γ is 20 kN/m^3. The mobilised shear strength from Fig. 5, for H/D=0.25 and bulk unit weight of soil (γ) of 20 kN/m^3 is:

$c_{mob} / \gamma D=0.0675$

giving mobilised shear stress (c_{mob}):

$c_{mob}=0.0675x20x20=27$ kPa

Since the deviatoric stress mobilised (q_{mob}) is twice the shear strength, then;

$q_{mob}=2c_{mob}=2x27=54$ kPa.

From the stress-strain curve plotted in Fig. 11, the corresponding triaxial shear strain (ε_q) is:

$\varepsilon_q=0.00125$

The engineering shear strain $\varepsilon_{smob,}$ which has to be mobilised, is equal to 1.5 triaxial shear strain ε_q thus:

$\varepsilon_{smob}=1.5* \varepsilon_q=1.5*0.00125=0.00188$

From the plastic deformation mechanism (Equation 5), Therefore:

$2\delta\theta=0.00188$

$\delta\theta=0.00188/2=0.00094$

The height of the pivot point above the toe (r), normalised by the overall height (D), for various excavation ratios is plotted in Fig. 6. For H/D=0.25

$r/D =0.0115$

Thus:

$r=0.0115*20=0.23$m

The height of the wall above the rotation point (L) (Fig. 9) is given by:

$L=D-r=20-0.23=19.77$ m

The displacement at the top of the wall (Δ) is therefore given by

$\Delta=\delta\theta*L=0.00094*19.77=0.019m=19mm$

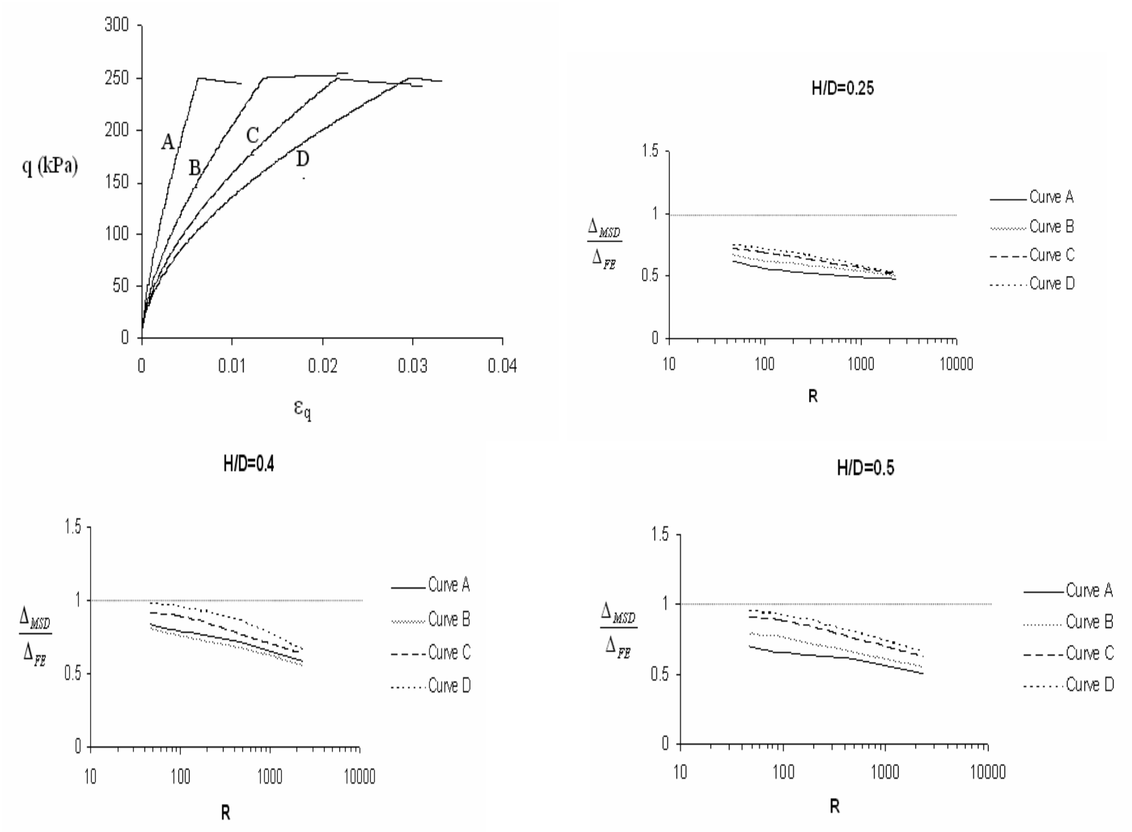
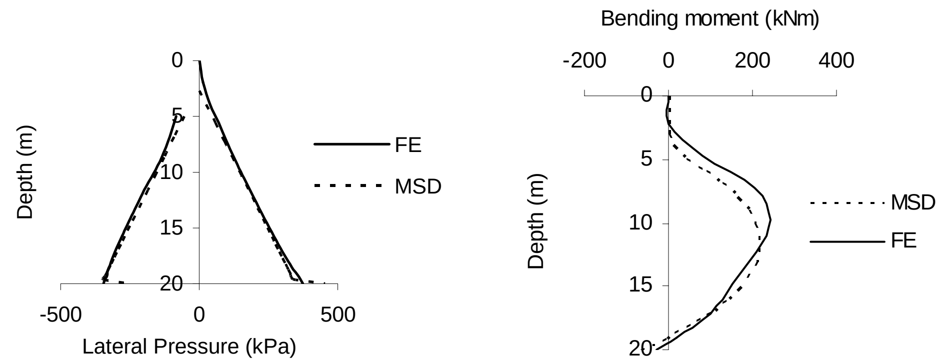


Figure 6 Comparison of crest displacements between FE and MSD for different stress-strain behaviour

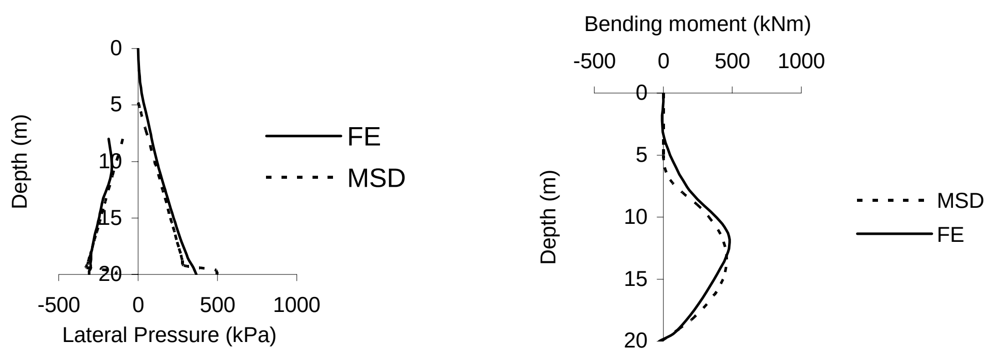
Conclusion

Displacements in the MSD method are controlled by the average soil stiffness in the zone of deformation. Stress-strain data from an undisturbed soil sample taken at the mid-height of the retaining wall prior to excavation can be used to deduce the average shear strength which can be mobilised at the required shear strain in MSD calculations.

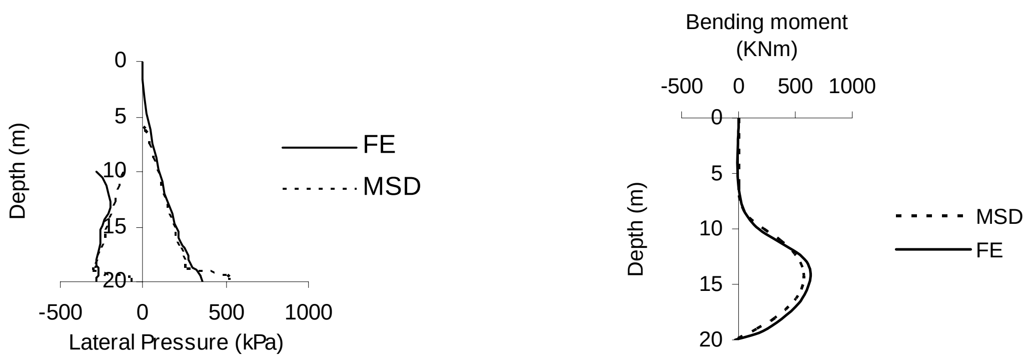
The key advantage of the MSD method is that it gives the designers the opportunity to consider the sensitivity of a design proposal to the non-linear behaviour of a representative soil element. It accentuates the importance of acquiring reasonably undisturbed samples, and of testing them with an appropriate degree of accuracy with the local measurement of strains (e.g. 0.01%). The extra step of actually performing FE analyses remains open, with the advantage that the engineer would then have an independent check on the answer to be expected, within a factor of 2 on displacement.



(a)



(b)



(c)

Figure 7 Lateral stress distribution and bending moments (a) 5m excavation (b) 8m excavation (c) 10m excavation

Acknowledgement

The authors are grateful to Cambridge Commonwealth Trust and to the Committee of Vice-Chancellors and Principals of UK universities (Overseas Research Scheme) for their provision of financial support to the first author.

References

Bolton, M.D., Powrie ,W. and Symons ,I. F. (1990a). "The design of stiff in-situ walls retaining overconsolidated clay, part 1 short term behaviour", *Ground Engineering*, **23**(1), 34-39.

Bolton, M.D., Powrie, W. and Symons, I. F. (1990b). "The design of stiff in-situ walls retaining overconsolidated clay, part II, long term behaviour", *Ground Engineering*, **23**(2), 22-28

Bolton, M. D. and Sun, H. W. (1991). "Modelling of bridge abutments on stiff clay", *Proceedings of 10th European conference on soil mechanics*, Florence, Italy, Vol. 1, 51-55.

Burland, J.B. and Wroth, C.P. (1975). "Settlement of buildings and associated damage", *Building Research Establishment*, Paper no. CP 33/75.

Dasari, G. R. (1996). "Modelling the variation of soil stiffness during sequential construction", *PhD dissertation*, University of Cambridge.

Hibbit, Karlsson & Sorensen Inc. (2001), *ABAQUS/Standard User's manual, version 6.2*

Osman, A.S. and Bolton, M.D. (2004). "A New Design method for retaining walls in clay", accepted for publication in *Canadian Geotechnical Journal*.

Powrie, W. and Li, E. S. F. (1991). "Finite element analyses of an in situ wall propped at formation level", *Geotechnique*, **41**(4), 499-514.

Rowe, P.W., (1955) "Sheet pile walls encastre at the anchorage", *Institution of Civil Engineers, Proceedings*, part 1, vol 4.

Simpson, B. and Driscoll, R. (1998). "Eurocode 7: A commentary, Construction Research Communications", Watford.