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A new approach to the estimation of undrained settlement of shallow foundations on soft clay

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ABSTRACT: The Mobilisable Strength Design (MSD) method is a new design approach based on the theory of plasticity and the concept of "mobilisable soil strength". The objective behind the introduction of the MSD method is to achieve a simple unified design methodology, which could satisfy both safety and serviceability in a single step of calculation. In conventional terms, this offers a rational procedure for selecting safety factors according to the stress-strain behaviour of soil. The possible use of MSD in the design of shallow foundations on soft clay is examined. The MSD method is used to back analyze tests on instrumented rigid square pads performed at the soft clay test site at Bothkennar in Scotland (UK). The MSD predictions for pad settlements conform well to the measured load-displacement behaviour.

1. INTRODUCTION

Designers have to check that shallow foundations will neither penetrate the soil subgrade in a bearing capacity failure, nor settle excessively. Bearing failure is checked using plasticity theory, whereas settlement is usually checked using elasticity. Conventionally, the calculations for settlement in saturated clay are divided into two components: immediate settlements due to deformation taking place at constant volume and the consolidation settlement accompanying the dissipation of pore water pressure (Skempton and Bjerrum, 1957). Excessive total or differential settlements are a main cause of unsatisfactory building performance. Although this is sometimes due to unexpected consolidation, the inadequacy of linear elasticity to describe the earlier phase of undrained settlement leads to significant uncertainties. This paper proposes a resolution of the latter problem.

The stress-strain behaviour of soil is highly non-linear from very small strains. Non-linear stress-strain characteristics can have a dominant influence on the form and scale of the displacement distribution of structures on soft clay. Therefore, there is a need for a simple design approach, which can relate successfully serviceability and collapse limits to the real nature of the soil.

A new design approach has been developed. The proposed design method treats a stress path in a representative soil zone as a curve of plastic soil strength mobilised as strains develop. Conventional bearing capacity factors are used to derive mobilised shear stresses from working loads. The working strain is then deduced from the mobilised shear stress using raw test data. Strains are entered into a simple plastic deformation mechanism to predict boundary displacements. Hence, the proposed Mobilisable Strength Design (MSD) method might satisfy both safety and serviceability in a single step of calculation.

2. PLASTIC DEFORMATION MECHANISM

2.1 Theoretical formulation

This solution uses the geometry of the well-known Prandtl mechanism (Figure 1) for plane strain indentation to propose a plastic region of continuous deformation beneath a rigid circular punch. Outside this region, it is assumed that strain is negligible (Osman and Bolton, 2004). The solution includes three zones of distributed shear. These zones are assumed to shear and deform compatibly and continuously with no relative sliding at their boundaries. Soil strains and compatible deformations are developed according to the shear stress that keeps the foundation in equilibrium.

The shear stresses in the soil are related to the external loading of the footing by the usual bearing capacity coefficient (N_c):

$$\sigma_{mob} = N_c c_{mob} \qquad (1)$$

where σ_{mob} is the applied bearing pressure, and c_{mob} is shear stress mobilized in the soil.

Compatibility conditions are satisfied through the specification of a kinematically admissible mechanism. Figure 1 shows the selected deformation pattern in which there are no displacement discontinuities. Soil displacements vary quadratically with the position inside the plastic mechanism.

Since there is no volume change in undrained conditions; the following condition should be satisfied:

$$\frac{\partial u}{\partial r} + \frac{u}{r} + \frac{\partial v}{\partial z} = 0 \qquad (2)$$

where u and v are the radial and the vertical displacement respectively, r is the radial distance from the centreline of the footing, and z is the depth below the ground surface.

The imposition of axial symmetry, the requirement for zero displacement at the outer boundary, together with equation (2), allow the parameters of the quadratic displacement field to be written down (Osman and Bolton 2004). Each displacement component is proportional to the footing displacement δ. Strains can then be found from the first derivative of the displacements. Since the spatial scale is fixed by the footing diameter D, all strains components are proportional to δ/D.

The engineering strain γ, which is equal to 1.5 times the axial strain ε_a in an undrained triaxial test, can be defined as the difference between the maximum and the minimum principal strain. The average shear strain γ_{mob} mobilized in the deforming soil can be calculated from the spatial average of the shear strain in the whole volume of the deformation zone (Figure 1):

$$\gamma_{mob} = \frac{\int_{vol} \gamma dvol}{\int_{vol} dvol} = 1.33\frac{\delta}{D} \tag{3}$$

A full mathematical derivation is given in Osman and Bolton (2004).

A relation between applied bearing pressure and the displacement of footing can be established if the relation between shear stresses and shear strains can be obtained, such as from a carefully chosen undrained triaxial test. The calculation procedure of the MSD method is summarized in Figure 2.

The compromise of the new approach is therefore couple together an equilibrium solution based on the mobilisation of a constant shear strain c_{mob}, with a kinematic solution based on the creation of an average mobilised shear strain γ_{mob}.

2.2 Finite Element validation

Figure 3a shows the results of finite element (FE) calculations for a triaxial sample and for a surface plate test (Hillier & Woods, 2001) plotted on a logarithmic scale. The analyses were carried out using the non-linear numerical model of Gunn (1993) with the same set of material parameters which are shown in Figure 3a. The stress-strain curve for the triaxial sample is plotted with axes q and ε_a (deviatoric stress and deviatoric strain respectively). The behaviour of the foundation is plotted as loading pressure σ and settlement to diameter ratio δ/D.

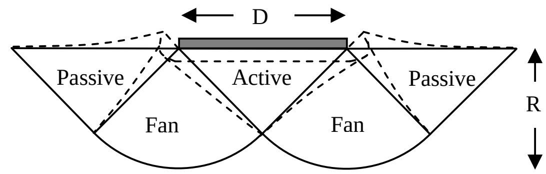


Figure 1 Plastic deformation mechanism for shallow foundation on clay

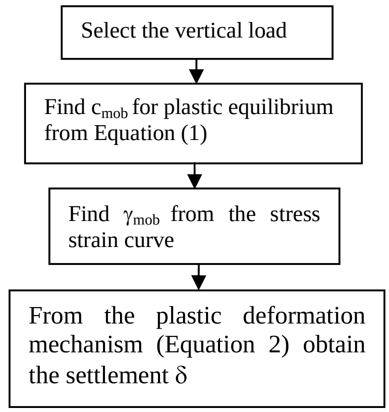


Figure 2 Calculation procedure in the MSD method

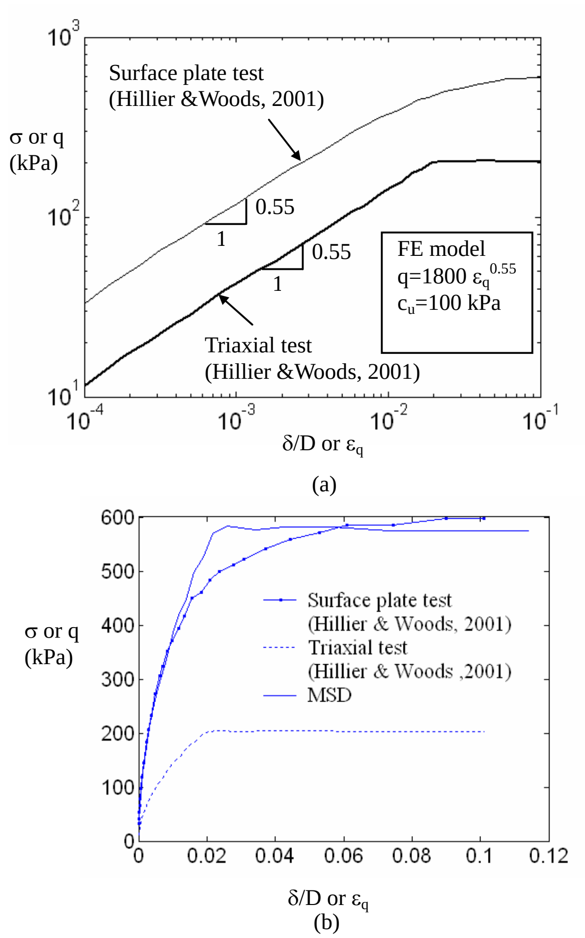


Figure 3 Load-settlement response and triaxial stress-strain relation

Figure 3b shows a comparison between MSD predictions for the load-displacement curve and the FE calculations. From the figure the MSD predictions conform well to FE results. These results support the hypothesis that stress-strain data from an undisturbed soil sample can be used to predict the settlements of foundations.

3. BACK ANALYSIS OF LOADING TESTS ON A STIFF FOOTING AT BOTHKENNAR

3.1 Site

Bothkennar soft clay test site was a facility for large or full scale experimental research. It was owned and managed by the UK government through the Engineering and Physical Science Research Council (EPSRC). It lies approximately midway between Edinburgh and Glasgow, and borders on to the River Forth in Scotland (UK). The site has an area of 11 ha and 20m depth of soft saturated soils. The site has an uncomplicated soil profile which facilitates back-analysis and the interpretation of field experiments. An extensive site investigation was performed and documented by various authors (Institution of Civil Engineering, 1992).

Pad loading tests were carried out by Jardine *et al.* (1995) to investigate bearing capacity and load displacement behaviour under short-term and long term conditions. The soil profile under the pad footings is summarized in Figure 4. Figure 5 shows the soil properties.

3.2 Field tests

Two reinforced concrete pads were cast in 0.8m deep excavations (Figure 6). Pad A was 2.2m square and pad B was 2.4m square. As it is common in bearing capacity calculations to treat circles and squares of equal areas as being equivalent (Skempton, 1951), the equivalent diameters of pad A and pad B are 2.48m and 2.71 respectively. However, there is no theoretical justification for this assumption. In this study the settlement of pad A only is compared with MSD preditions.

3.3 Pore pressure response during loading

Jardine et al. (1995) compared the pore pressure measured under the centre line of pads A and B with theoretical predictions and found that the upper silty strata (z/D<0.5) showed pronounced pore water pressure dissipation during loading pause period. However, the conditions were practically undrained on the centre line provided z exceeds 0.5D.

Partial drainage during loading has been noted in several field studies of structures on soft clays (Tavenas & Leroueil, 1980; Wood, 1980; Nicholson & Jardine, 1981; Watson *et al.* 1984, Jardine *et al.*, 1995). However, the field dissipation rate invariably slows down dramatically once large strain yield stresses are exceeded (Jardine *et al.*, 1995).

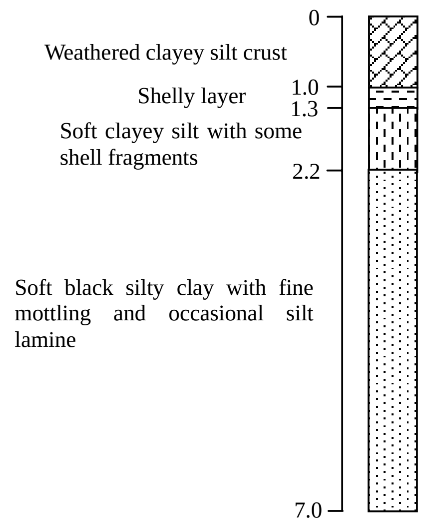
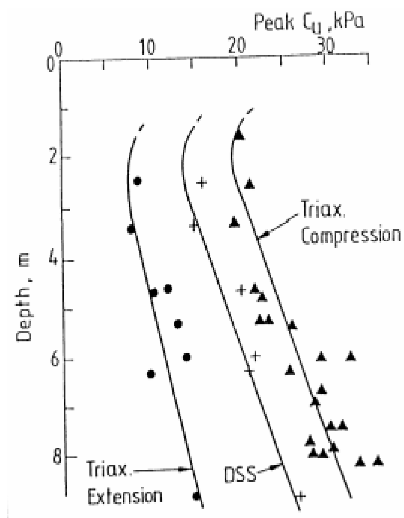
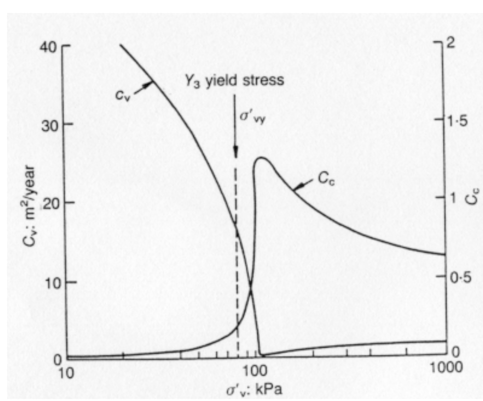


Figure 4 Soil profile (after Jardine *et al.*, 1995)



(a)



(b)

Figure 5 Soil properties (Gildea, 1993 & Jardine *et al.*, 1995)

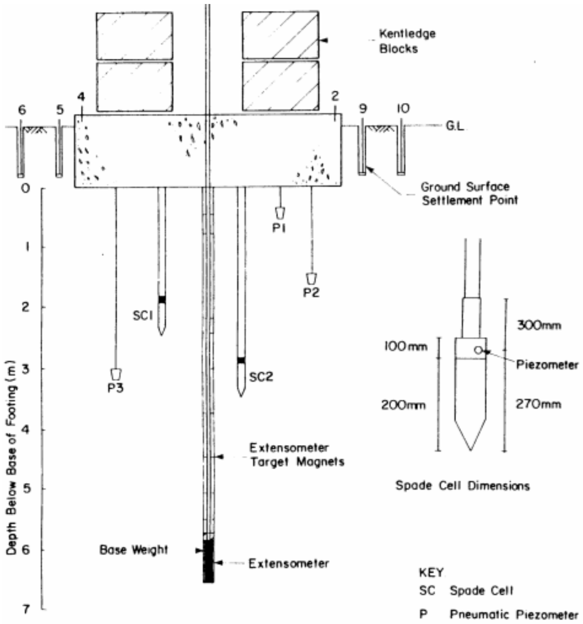


Figure 6 Instrumentations layout (Gildea, 1993)

3.4 Surface settlement during loading

Figure 7 shows the ground surface settlement during loading to failure. The bearing pressure is represented in terms of the mobilized load factor L_f, which is defined as the ratio of current bearing pressure to the ultimate capacity provided in test A. These load factors were plotted with axes r/D and δ_r/δ_c (radial distance from the centre of the footing to diameter ratio, and settlement to settlement of the pad's centre point respectively). It is clear that the ground surface settlement diminishes rapidly with radial distance. This result is in agreement with the proposed plastic deformation mechanism shown in Figure 1. However, no ground heave has been observed in the pad test. But, this may be due to the partial drainage in the upper soil layers.

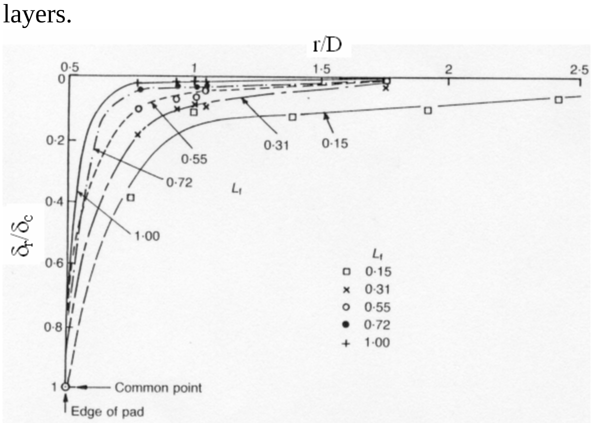


Figure 7 Surface settlement profile at a range of load factors (Jardine *et al.*, 1995)

4. MSD CALCULATIONS

4.1 Assumptions

The behaviour of Bothkennar clay may be described at various levels of sophistication. However, the MSD method provides a simplified model of the complex reality for use in design and decision-making. The approximation is good if the mechanism is appropriate. Overall function is more important than local details. In structural engineering, beam theory axiomatically fails to deal with stress concentrations at joints and ignores shear deformations; however, it offers a valuable procedure in the design of buildings.

The following assumptions have been made in the back analyses of the pad tests using 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 standard bearing capacity coefficients 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.

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 deformation mechanism in the MSD method is derived for a surface footing. However the pad footing at Bothkennar is embedded by a depth of 0.8m (z/B=0.32). Brinch Hansen's (1970) depth correction factor (f_d) was adopted to account for embedded depth (z) in the calculation of the mobilized strength in MSD of a foundation of width (D). The bearing capacity factor N_c should be increased by factor f_d.

$$f_d = 1 + 0.4z/D \qquad (4)$$

However, no comparable adjustment was made to the plastic deformation mechanism. Although this approach is clearly approximate, it will be shown that the use of correction factor f_d can lead to acceptable predictions. For back-analysis of pad A the correction factor (f_d) of 1.13 is adopted. The bearing capacity factor for a rough surface circular is 6.05 (Cox *et al.*, 1961); applying the depth correction factor gives an overall bearing capacity factor (N_c) of 6.83.

4.2 Stress-strain behaviour

The representative sample in the MSD calculations should be taken at a depth of 0.3 D, which, in this case, is about 1.5 m below the ground surface. Although it is routine in design practice to monitor the footing settlement and ground deformations around structures, it much less common to take representative samples for testing and extremely rare to take these samples at shallow depths underneath footings. Figure 8a shows triaxial compression data for different depths of Bothkennar soft clay, and Figure 8b shows triaxial extension stress-strain data. No triaxial data at shallower depth (1.5m) is reported at the literature. Engineering judgment is needed to predict stress-strain behaviour at the required characteristic depth. It should be borne in mind that: at the characteristic depth, the peak undrained shear strength in compression and extension are 20 kPa and 10 kPa respectively (Figure 5a), the soil is less stiff at shallower depth, and the Sherbrooke sampler produced higher quality samples than other samplers in Bothkennar soft clay (Hight *et al.* 1992). These three considerations were used to select the representative stress-strain curves adopted in the MSD calculation (Figure 8).

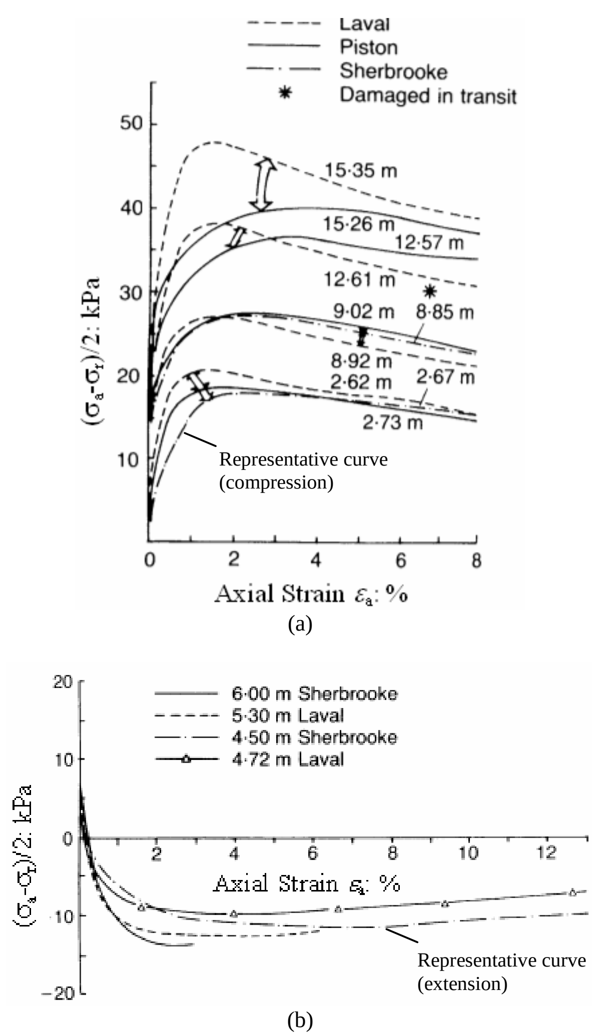


Figure 8 Soil stress-strain behaviour (after Hight *et al.*, 1992) (a) CK_oU triaxial compression (b) CK_oU triaxial extension

4.3 Comparison of the results

Figure 9a shows the MSD calculations compared with the field measurements. Although there is a discrepancy at high bearing pressures, these results show a good agreement in the prediction of the settlement pattern. In the MSD method, the deformation is assumed to be controlled by the average soil stiffness. Therefore, the average value of settlements predicted from triaxial extension data and compression data should be taken. However, the designer should consider whether to use the average value or the worst estimate of the settlement (Figure 9b), depending on the degree of uncertainty associated with the design parameters.

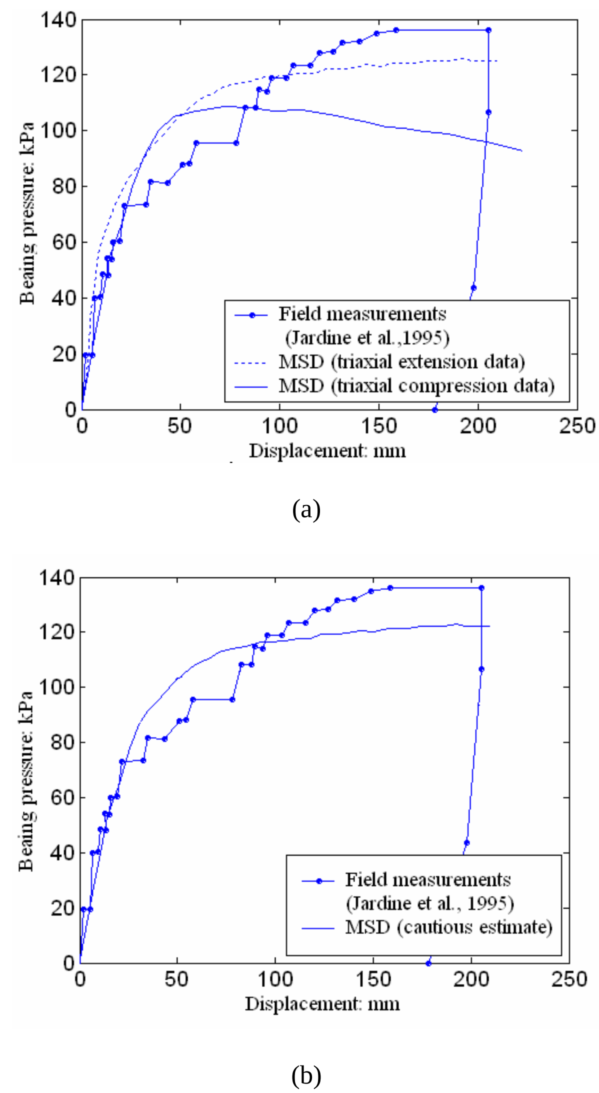


Figure 9 Comparison between the MSD prediction for load-displacement curve and field measurements

The allowable settlement of a shallow foundation as proposed by Décourt (1992) is 0.75% of the diameter of the loading surface, which gives a settlement of 18.6 mm in the present case. This is corresponding to an applied load of 60 kPa. The mobilisable undrained shear strength was about 10 kPa, which represents about 50% of the undrained strength (c_u). This example shows that serviceability checks can be more critical than collapse checks. Satisfaction of serviceability limits can lead consequently to the satisfaction of safety requirements.

This method offers a rational procedure for selecting safety factors according to the stress-strain behaviour of soil. These results show that in some cases, serviceability can be the most important design criterion.

5. CONCLUSIONS

Modelling soil stiffness properly in the analysis and design of shallow foundations is very important. For design purposes of circular footings in homogenous soils, displacements can be assumed to be controlled by the average soil stiffness in a typical zone of plastic deformation. Stress-strain data from an undisturbed soil sample taken at the mid-depth of the deformation mechanism can be used to deduce the average shear strength, which needs to be mobilised at the required shear strain in MSD calculations.

An extension of bearing capacity theory to include plastic deformation mechanisms with distributed plastic strains can provide a unified solution for design problems. This application can satisfy approximately both safety and serviceability requirements and can predict stresses and displacements under working conditions.

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 appropriate degree of accuracy in the local measurement of strains (e.g. 0.01%).

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REFERENCES

Brinch Hansen, J. (1970), A revised and extended formula for bearing capacity, *Danish Geotechnical Institute*, Bulletin 28,5-11.

Décourt, L. (1992), SPT in non classical material. *Proceedings of US-Brazil Geotechnical Workshop on Applicability of Classical Soil Mechanics Principles in Structured Soil*,Belo Horizonte, pp. 67-100.

Gildea, P. (1993), Instrumented footing load tests on soil sensitive Bothkennar clay, *Ground Engineering*, vol. 26, No. 6, pp. 30-38.

Gunn, M. J. (1993), The prediction of surface settlement profiles due to tunneling, *Predictive soil mechanics*, Houlsby and Schofield (eds.), Proceedings of Wroth Memorial Symposium, Thomas Telford, London, pp. 304-314.

Hight, D.W., Bond, A.J., and Legge, J.D. (1992), Characterization of the Bothkennar clay: an overview, *Geotechnique*, 42, No. 2, pp. 303-347.

Hillier, P. R., and Woods, R. I. (2001), Characterisation of non-linear elastic soil behaviour from field load test, *Proceedings of the Fifteenth International Conference on Soil Mechanics and Geotechnical Engineering*, A.A. Balkema, Rotterdam, Vol. 1, pp. 421-424.

Institution of Civil Engineers (1992), Bothkennar soft clay test site : Characterization and lessons learned, *Geotechnique*, Vol. 42, No. 3, pp. 161-378.

Jardine, R.J., Lehane, B.M., Smith, P.R. and Gileda, P.A. (1995), Vertical loading experiments on rigid pad foundations at Bothkennar, *Geotechnique*, Vol.45, No 4,pp. 573-597.

Nicholson, D. P., and Jardine, R. J.(1981), Performance of vertical drainage at Queenborough Bypass, *Geotechnique*, Vol. 31, No. 1 ,pp. 67-90.

Osman, A. S., and Bolton, M. D. (2004), Plasticity based method for predicting settlement of shallow foundations, Submitted to ASCE *Journal of Geotechnical and Geoenvironmental Engineering*, for review.

Skempton, A.W (1951), The bearing capacity of the clay, *Proceedings of Building Research Congress*, vol. 1, 180-189.

Skempton, A.W, Bjerrum, L (1957), Contribution to the settlement analysis of foundations on clay, *Geotechnique*, Vol.7, No. 4,pp. 168-178.

Tavenas, F, and Leroueil, S. (1980), The behaviour of embankments on clay foundations, *Canadian Geotechnical Journal*, Vol.17, No. 2 ,pp. 236-260.

Watson, G. H, Crooks, J. H. A., Williams, R.S. and Yam S.S. (1984), Performance on preloaded and stage loaded structures on soft soils in Trinidad, *Geotechnique* , Vol.17, No. 2 ,pp. 236-260.

Wood, D.M.(1992), Yielding of soft clay at Backebol, Sweden, *Geotechnique* , Vol. 38, No. 1 ,pp. 49-65.