Teaching geotechnical engineers to avoid excessive deformations

Ingénieurs géotechniques d'enseignement pour éviter des déformations excessives

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ABSTRACT

This paper introduces a design approach based directly on the data of carefully chosen soil tests, conceived within the framework of plasticity theory, but allowing for strain-hardening. Mobilized shear stresses beneath foundations, for example, are found by using conventional bearing capacity factors. Strains required to mobilize these stresses are deduced from a triaxial test on a representative sample taken from a specified location in the plastic zone of influence. These strains are entered into a simple plastic deformation mechanism to predict boundary displacements.

RÉSUMÉ

Cet article présente une approche de dimensionnement basée directement sur les données d'essais de sol soigneusement choisis. La méthode est conçue dans le cadre de la théorie de la plasticité et tient compte de l'écrouissage. Les efforts de cisaillement mobilisées en dessous des fondations, par exemple, sont trouvées en utilisant les facteurs de capacité portante conventionnels. Les déformations nécessaires pour mobiliser ces efforts sont déduites d'essais triaxiaux sur un échantillon représentatif pris d'un emplacement choisi dans la zone d'influence plastique. Ces déformations sont entrées dans un simple mécanisme de déformation plastique pour prédire les déplacements de bord.

1 INTRODUCTION

The conceptual understanding involved in the estimation of ground displacements is generally poor. Engineers use factors of safety against the peak soil strength in the hope that strains will broadly be acceptable. However, the strain needed to mobilise peak strength varies from soil to soil. Also, there are different definitions and rules for selecting safety factors in design codes. Most of these definitions fail to address the real nature of soil, which always shows a non-linear and sometimes brittle response. Elasticity theory may be used to predict displacements, but the applications are often algebraically complex even though they are based on an arbitrary equivalent modulus. Although many aspects of non-linear soil behaviour are incorporated into specialist constitutive models and included in finite element packages, practising engineers complain that the parameters lack clear physical meanings, and analyses require disproportionate effort. The purpose of this paper is to show geotechnical engineers how to use a stress-strain curve from a single soil test, together with a simple hand calculation, to calculate both stability and soil deformation without the need for complex computer calculations. Currently, the method is restricted to construction effects in clays that are presumed to remain undrained.

2 MOBILISABLE STRENGTH DESIGN (MSD) METHOD

The two basic elements of this approach are:

- Simple plastic mechanisms are used, which represent the working state of geotechnical facilities. These mechanisms represent both the equilibrium and displacements of the various soil bodies, especially at their junction with the superstructure. Since solutions are intended for working states, displacement discontinuities must generally be avoided.
- Raw stress-strain data from soil tests (e.g. triaxial or direct shear tests) on undisturbed samples taken from representative locations are used directly to predict displacements under working conditions. The use of constitutive laws and soil parameters is avoided.

3 APPLICATION EXAMPLE: SHALLOW FOUNDATIONS

3.1 *Theoretical formulation*

Conventional bearing capacity theory has been extended by including plastic deformation mechanisms with distributed plastic strains. The proposed plastic deformation mechanism uses the well-known Prandtl solution for indentation to set the boundaries of a plastic zone of deformation beneath a circular punch. Within this zone, a continuous displacement field has been imposed to avoid shear discontinuities and cracks and to satisfy incompressibility (Figure 1). Soil displacements vary quadratically with the position inside the plastic mechanism.

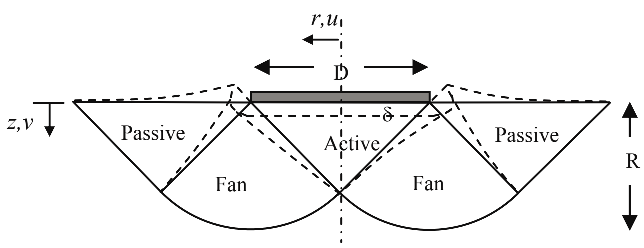


Figure 1 Plastic deformation mechanism for circular pad foundations

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

$$\frac{\partial u}{\partial r}+\frac{u}{r}+\frac{\partial v}{\partial z}=0 \qquad (1)$$

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, and the satisfaction of undrained conditions, allows the parameters of the quadratic displacement field to be written down (Osman and Bolton 2004a). 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 strain components are proportional to δ/D. The engineering shear 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 strains. 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 assumed deformation zone (Figure 1):

$$\gamma_{mob}=\frac{\int_{vol}\dot{\gamma}\,dvol}{\int_{vol}dvol}=M_c\frac{\delta}{D} \qquad (2)$$

in which M_c can be shown to take the value of 1.33. A full mathematical derivation is given in Osman and Bolton (2004a).

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

$$\sigma_{mob}=N_c c_{mob} \qquad (3)$$

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

A relation between applied bearing pressure and the displacement of the 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 required location can be shown to be at a depth of about 0.3D below the footing, considering the spatial distribution of plastic work within the assumed mechanism, and assuming a linear profile of shear strength. The essential compromise of the new approach is therefore to 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}. Figure 2 illustrates the method of estimating the load settlement curve directly from the stress-strain curve. This makes clear that the non-linearity of the representative stress-strain curve is taken as identical to that of the normalised load-displacement curve of the foundation. Plasticity theory is used to obtain the linear transformations of the *axes* through the normalisation factors M_c and N_c.

3.2 *Validation: Back analysis of a stiff pad test at Bothkennar*

The MSD method is used to back analyze a test on an instrumented rigid square pad, performed at the soft clay test site at Bothkennar in Scotland, UK (Jardine et al. 1995). The aim of the test (test A) was to study the short-term behaviour and the ultimate bearing capacity of rigid foundations under vertical loads. Pad A was 2.2 m and 0.8 m embedded depth. As it is common in bearing capacity calculations to treat circles and squares of equal areas as being equivalent (Skempton, 1951), the equivalent diameter of pad A is taken as 2.48m.

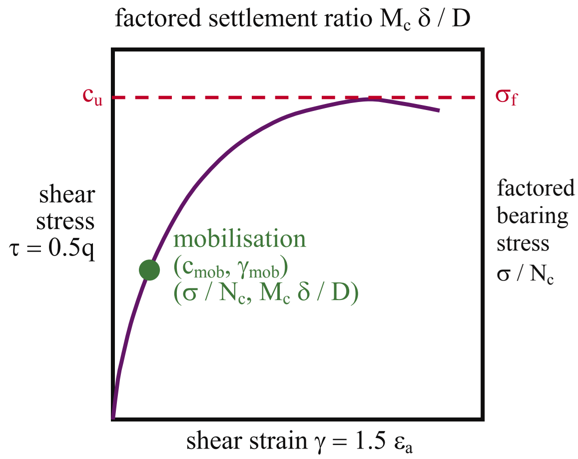
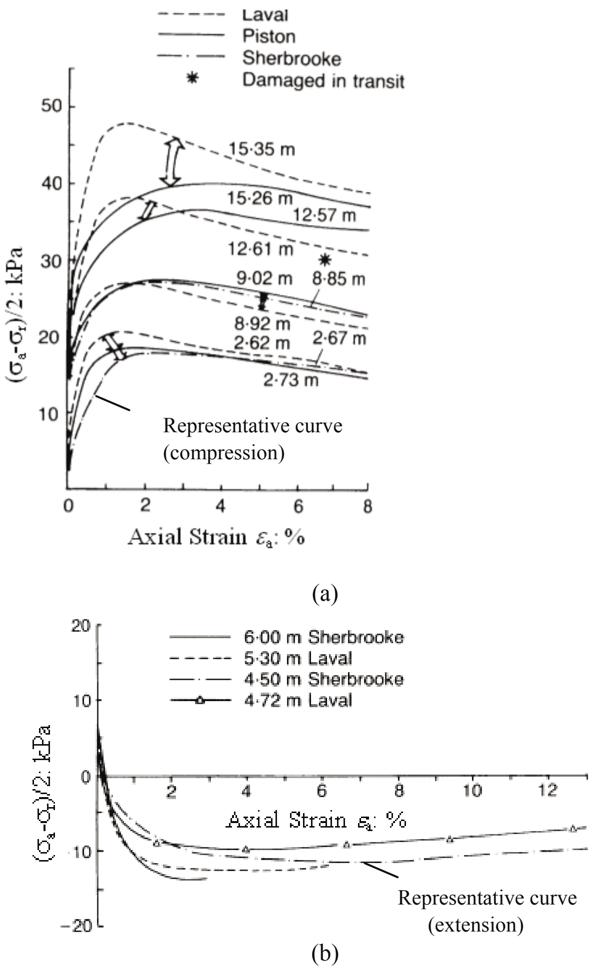


Figure 2 Bearing stress versus settlement for pad foundations

Figure 3 shows triaxial data for different depths of Bothkennar soft clay. Engineering judgment is needed to predict stress-strain behaviour at the required characteristic depth of 0.3D below the pad base. There are three considerations: the peak undrained shear strengths in compression and extension are 20 kPa and 10 kPa respectively (Jardine et al 1995), the soil is less stiff at shallower depth, and the Sherbrooke sampler gave higher quality samples than other samplers in Bothkennar soft clay (Hight *et al.* 1992). The representative stress-strain curves adopted in the MSD calculation are indicated in Figure 3.



(b)

Figure 3 Soil stress-strain behaviour (after Hight *et al.*, 1992) (a) CK_oU triaxial compression (b) CK_oU triaxial extension

Figure 4 shows the MSD calculations compared with the field measurements. 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. Although there is some discrepancy at high bearing pressures, these results show a good agreement in the prediction of the bearing pressures associated with settlements up to 25mm.

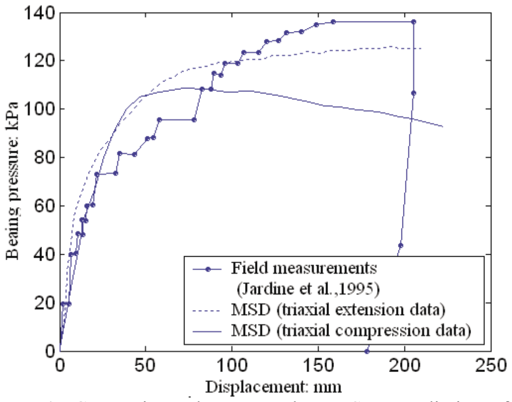


Figure 4 Comparison between the MSD prediction for load-displacement curve and field measurements

4 APPLICATION EXAMPLE: RETAINING WALLS

4.1 *Braced excavation*

Following O'Rourke (1993), the incremental lateral displacement profile of a multi-propped wall retaining an excavation in soft clay, and subject to excavation of the soil beneath the lowest level of support, can be assumed to conform to a cosine function as follows (also see Figure 5).

$$\delta w = \frac{\delta w_m}{2}\left[1-\cos\left(\frac{2\pi y}{\ell}\right)\right] \qquad (4)$$

Figure 6 proposes a new plastic deformation mechanism for such an incremental lateral displacement, (Osman and Bolton 2004a). In these mechanisms, the wall is assumed to be fixed incrementally in position and direction at the lowest level of props, which implies that the wall has sufficient strength to avoid the formation of a plastic hinge. The wall and soil are deforming compatibly and the soil deformation profile follows the cosine function of Equation 4. From the assumed displacement field the average shear strain mobilized in the soil can be linked to the maximum incremental displacement:

$$\gamma_{mob} = \frac{\int_{vol} \dot{\gamma} dvol}{\int_{vol} dvol} \approx 2\frac{\delta w_m}{\ell} \qquad (5)$$

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

At each stage of the excavation, the strength c_{mob} mobilised due to the excavation of soil beneath the lowest support is obtained from the virtual work equation by equating the energy dissipated in the plastic deformation mechanism with the work done by the gravity force. The corresponding mobilised shear strain γ_{mob} is found from the stress-strain curve obtained from a soil test (e.g. direct simple shear tests) on a representative undisturbed sample. The maximum incremental wall movement is then calculated from the corresponding increment in shear strain (Equation 5). The incremental wall displacement profile is then plotted using the cosine function of Equation 4. The total bulging displacement profile at the end of each stage of the excavation is obtained by accumulating the incremental movement profile at the current excavation stage with the incremental profiles from previous stages.

In an ideal excavation process, the first supports are installed at an early stage in order to minimize cantilever movements in the wall. However, this may not be possible in practice due to the variety of site conditions and construction sequences. Therefore, the wall often deforms in a cantilever mode before the installation of first support level. Clough et al. (1989) suggest that the movements due to the cantilever mechanism and bulging mechanism can be added together to obtain the final movement (Figure 7). Osman and Bolton (2004c) explained the procedure for estimating deformations of cantilever retaining walls using MSD.

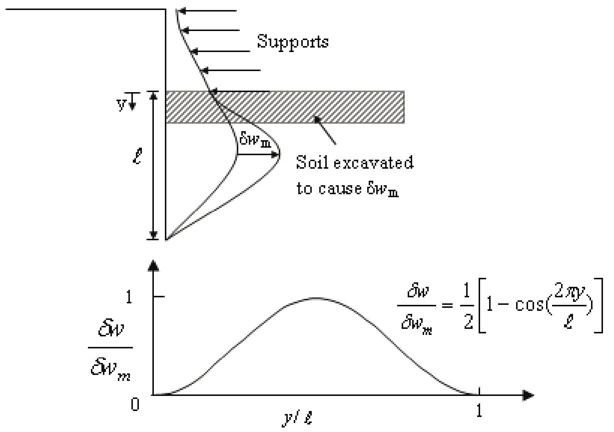


Figure 5 Incremental displacements in braced excavation (after O'Rourke 1993)

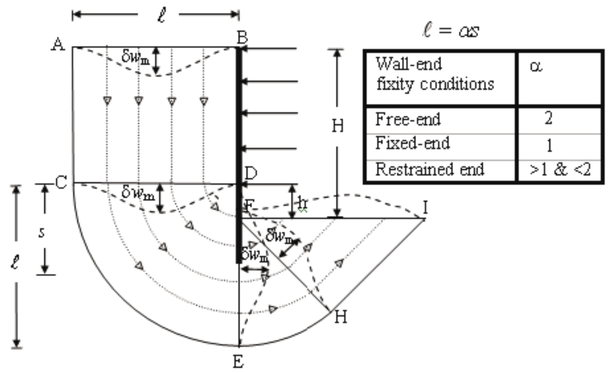


Figure 6 Plastic deformation mechanism for braced excavations in clay

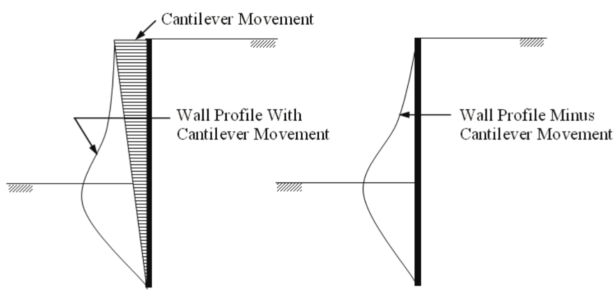


Figure 7 Effect of cantilever stage movement on system displacement (Clough et al. 1989).

4.2 *Validation: Back analysis of Boston Square Garage braced excavation*

Osman and Bolton (2004b) used the MSD method to back-analysis the performance of the deep excavation of Post Office Square Garage in Boston (Whittle et. al 1993). Figure 8 shows typical stress-strain behaviour of Boston Blue Clay. Figures 9 and 10 show that MSD predictions conform well to the measured displacement data.

5 CONCLUSIONS

Plasticity theory in engineering practice has almost exclusively been confined to the prediction of collapse loads. The work presented in this paper shows that non-linear materials exhibiting plastic hardening have been brought within the framework of simple plasticity theory by considering plastic deformation mechanisms that avoid slip discontinuities. "Limit equilibrium" concepts such as bearing capacity factor N_c can equally be applied at working load to obtain a mobilised soil shear stress. A new class of compatibility factor M_c has been introduced to scale proportional ground displacements into soil shear strains. In the case of construction-induced ground displacements in clays it has been shown that an undrained stress-strain test can equally be read as a scaled load-displacement plot for the actual construction in the field. Constitutive models are not necessary if appropriate stress paths have been followed.

This conclusion challenges the current consensus of researchers who demand complex numerical analyses in cases where ground movements during construction may be critical. The key to good prediction turns out to be good-quality soil tests in which strains are measured with sufficient accuracy for samples recovered from the appropriate location defined with respect to the MSD plastic deformation mechanism. This aids the development of a more coherent syllabus for ground and foundation engineering that focuses on the real uncertainties.

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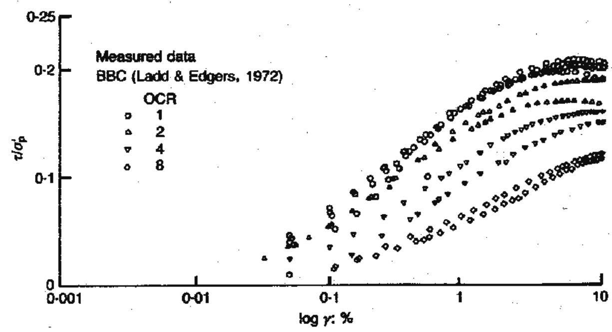

Figure 8 Stress-strain response for K_o consolidated undrained direct simple shear tests on Boston Blue Clay (Ladd and Edgers, 1972)

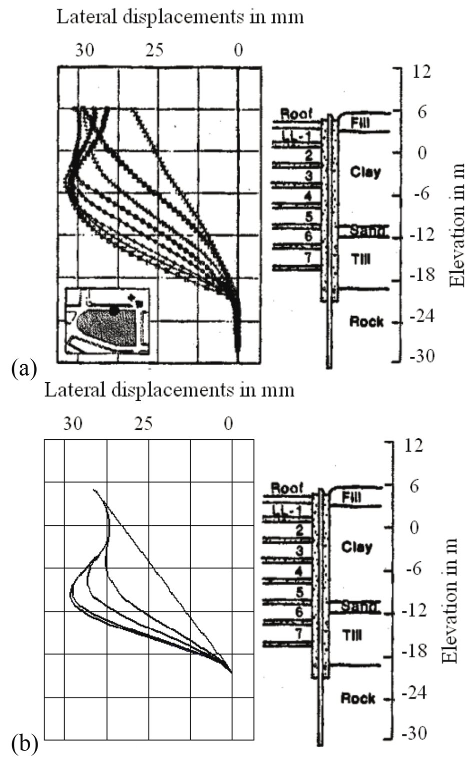

Figure 9 Wall movements during construction of Boston Square Garage (a) measurements (Whittle et al. 1993) (b) MSD predictions

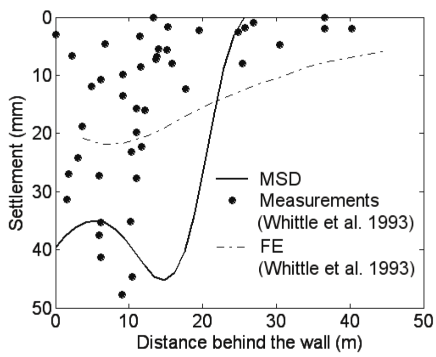

Figure 10 Predicted and measured surface settlements after the installation of the sixth floor at Boston Post Office Square Garage.