Soft Soil Engineering – Chan & Law (eds)

Back analysis of three case histories of braced excavations in Boston Blue Clay using MSD method

Ashraf Osman
Durham University, UK

Malcolm Bolton
Cambridge University, UK

ABSTRACT: This paper illustrates a new design approach conceived within the framework of plasticity theory, but allowing for strain hardening. Three examples demonstrating the applicability of modified plasticity theory to predict and control deformations around stiff-propped systems of braced excavations in soft clay soils are presented.

1 INTRODUCTION

The Finite Element (FE) method can provide a useful framework to evaluate the performance of geotechnical structures and to estimate the ground movements around structures. However, one of the most difficult factors to consider in the numerical analyses of geotechnical problems is the constitutive behavior of the soil. The soil is quite a complicated material that always shows a non-linear and sometimes brittle response. Furthermore, the soil has no unique relationship between strains and stresses. Many states of strain can correspond to a single state of stress or vice versa. The stress-strain relation in soil is complicated and subject to many factors such as stress history and anisotropy. Although, many aspects of non-linear soil stiffness are well understood and have been incorporated into numerical models, many of these models are relatively complex and the parameters lack clear physical meanings. Also, these analyses require special testing and lengthy computer calculations and therefore occupy a disproportionate time for practicing engineers. Thus, there are many practical cases for which these complex and expensive numerical analyses are not justified. 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 behavior.

The authors developed a new design approach for braced excavations retaining soft deposits. The proposed design method treats a stress path in a representative soil zone as a curve of plastic soil strength mobilised as strains develop. Limit analysis is used to derive mobilised shear stresses from working loads. 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 THEORETICAL BACKGROUND

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 (Figure 1).

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

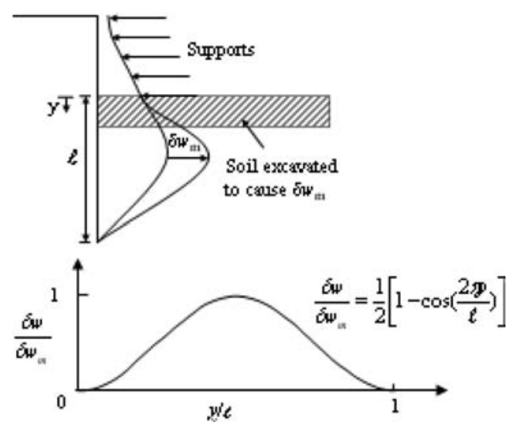


Figure 1. Incremental displacements in braced excavation (after O'Rourke 1993).

where δw is incremental wall displacement at any distance y from the lowest wall support; δw_m is maximum incremental displacement; and ℓ is the full wavelength of the deformation pattern.

Figure 2 shows a new plastic deformation mechanism for an incremental lateral displacement of a wall retaining soft clay. In these plane-strain 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 propping system is assumed to be stiff. The wall and soil are deforming compatibly and the soil deformation profile follows the cosine function of Equation 1. The average shear strain mobilized in the soil within the assumed displacement field can be linked to the maximum incremental displacement:

$$\delta\gamma_{mob} = \frac{\int_{vol} \delta\gamma\, dvol}{\int_{vol} dvol} \approx 2\frac{\delta w_m}{\ell} \tag{2}$$

where $\delta\gamma$ is the engineering shear strain defined as the difference between the major $\delta\varepsilon_1$ and minor $\delta\varepsilon_3$ principal strain increments $\delta\gamma = |\delta\varepsilon_1 - \delta\varepsilon_3|$, and vol is the volume of the plastic deformation mechanism shown in Figure 2.

At each stage of the excavation, the strength c_{mob} mobilised due to the excavation of soil beneath the lowest support can be found using the Principle of Virtual Work by balancing the virtual loss of potential energy to the virtual plastic work in distributed shearing.

$$\int_{volume} \gamma_t \delta v dVol = \int_{volume} c_{mob} \delta\gamma\, dVol \tag{3}$$

where γ_t is total unit weight of the soil, δv is vertical component of the displacement.
so that:

$$\beta = \frac{\int \gamma_t \delta v dvol}{\int c_u \delta\gamma\, dvol} \tag{4}$$

where $\beta = c_{mob}/c_u$.

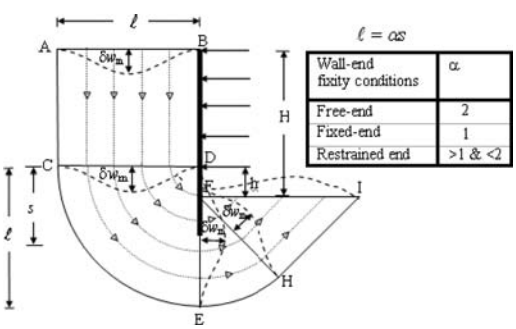


Figure 2. Plastic deformation mechanism for braced excavations in clay.

Detailed derivation and validation is given in Osman and Bolton (2006a, 2006b).

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 2). 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.

The wall often deforms in a cantilever mode before the installation of the 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 3). Following Bolton and Powrie (1988) and Osman and Bolton (2004), the deformation around a cantilever retaining wall can be idealized by means of triangles, one on each side of the wall, deforming in uniform shear (Figure 4) such that the mobilized shear strain γ_{mob} is twice the wall rotation $\delta\theta$. The proportional strength mobilised ($\beta = c_{mob}/c_u$) can then be obtained by Virtual Work, using appropriate integrals in equation 3. The corresponding mobilised shear strain γ_{mob} is found from the representative stress-strain curve. Then, the angle of wall rotation $\delta\theta$ is obtained by

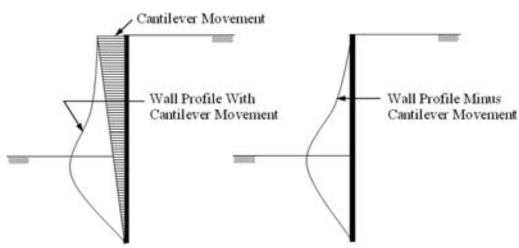


Figure 3. Effect of cantilever stage movement on system displacement (Clough et al 1989).

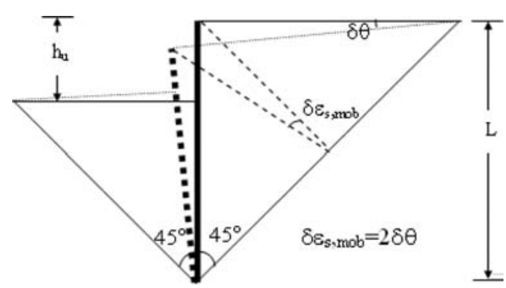


Figure 4. Plastic deformation mechanism for cantilever retaining walls in undrained conditions.

dividing the mobilised shear strain by 2 and the displacement at the top of the wall is calculated by multiplying the angle of wall rotation by the wall length. This cantilever movement then defines the initial ground displacement profile prior to propping, and the subsequent bulging displacements are added as explained above.

This cantilever movement then defines the initial ground displacement profile prior to propping, and the subsequent bulging displacements are added as illustrated in Figure 5.

3 CASE HISTORIES

The usefulness of the MSD method in practical application will be demonstrated for three case histories of excavations in the soft deposits of Boston Blue Clay: the 7-level underground Post Office Square Garage,

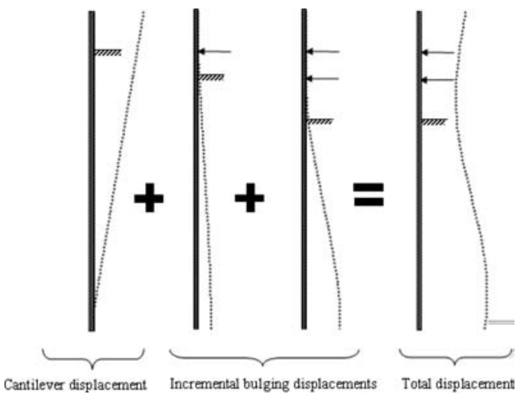


Figure 5. Calculation of wall total displacement in the MSD method.

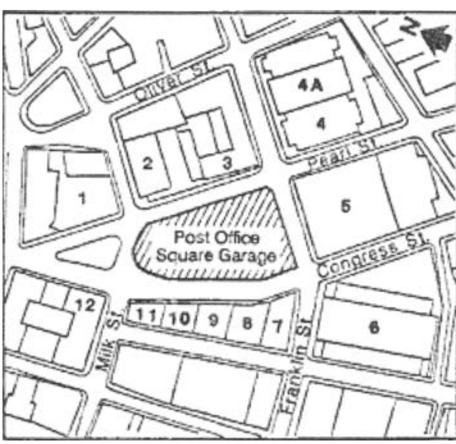
Boston, the Stata Centre at the Massachusetts Institute of Technology campus, and an excavation for 31-storey office tower development at 75 State Street, Boston.

3.1 *Post office square garage, Boston*

The first case history is the Post Office Square Garage braced excavation in Boston Blue Clay (Whittle *et al*., 1993, and Becker and Haley, 1990). The 1400 car parking underground garage was constructed with seven levels of below-grade structure in the heart of the downtown financial district of Boston in late 1980s. The garage occupies a plan area of 6880 m^2, and is bounded by the intersection of Pearl, Congress, Milk, and Franklin Streets (Figure 6). The excavation is surrounded by existing buildings up to 40 stories tall. The site was previously occupied by an old car park of three storeys (two storeys above the ground and one underground) which was demolished at the beginning of the construction of the new garage.

The top-down construction technique was used to provide a stiff, permanent bracing system as excavation proceeded. The initial construction of the roof of the garage was to create a staging platform throughout the construction process, thus minimizing traffic tie-ups on nearby streets. The top-down construction technique is also used to eliminate the use of tie-backs and temporary props (Becker and Haley, 1990).

The structure consists of a cast-in-place 0.9 m concrete diaphragm around the garage perimeter using the slurry trench method. The wall extended down into the bedrock and is braced internally by the floor slabs. The floor slabs are supported by interior steel columns (H-section) founded on sockets cast in the bedrock. Both the diaphragm wall and interior columns were installed prior to excavation. The slab roof and the seven lower floors were then cast in sequence from the top-down; the earth below each slab level was excavated and



Key	No. of stories	Foundation type	Bearing soil
1	14	Piers/caissons	Till
2	40	PIF's	Till
3	5	Footings	Clay
4	12	Footings/caissons	Clay
4A	34	Mat	Till
5	21	Footings	Till
6	38	Mat	Till
7	7	Mat/footings	Clay
8	11	Footings	Clay
9	7	Footings(?)	Clay(?)
10	12	Strip footings	Clay
11	11	Mat/footings	Clay
12	17	Mat	Clay

Figure 6. Post Office Square Garage site location and adjacent buildings (after Whitman et al. 1991).

the soil removed through temporary openings in the slabs. Movable concrete slabs were lowered into position as each floor was excavated. Since the 24 m deep excavation is very close to nearby streets, utilities and structures, extensive field monitoring was implemented to observe and document conditions around excavation. A detailed description of the method of construction and the performance of the wall is given in Becker and Haley, (1990) and Whittle et al (1993).

Subsurface soil and rock conditions at the site are summarized as follows:

1. A surface layer of fill consisting of a heterogeneous mixture of sand, gavel and construction debris varies in thickness from 0.6 to 4.0 m.
2. Below the fill, a clay deposit of low plasticity ($I_p = 20$–30%) ranges in thickness between 10 to 15 m. The overconsolidation ratio ranges from 6 to 2, decreasing with depth (Figure 7).
3. The clay is underlain by a layer of medium dense to very dense, fine to coarse sand with a varying amount of fine to coarse gravel. The thickness of the sand varies from 0.3 to 5.8 m. No sand layer is observed at locations north and east of Pearl Street.
4. Below the sand, there is a layer of till consisting of a hard heterogeneous mixture of particles of various sizes embedded in a compact silty-clay matrix. The till thickness ranges from 1.5 to 11.6 m.
5. The Bedrock is moderately to severely weathered argillite containing discontinuous layers of sandstone. The surface of the bedrock layer is located at elevations ranging from −16 to −20 m.

The soil profile found at Post Office Square is obviously complex and variable. However, the MSD method provides a simplified model of the complex reality for use in design and decision-making. The following assumptions have been made in the back analysis of this case history using the MSD method:

- The undrained strength of the soil is assumed to be isotropic and is determined from direct simple shear data.
- The overconsolidation profile is based on the average soil profile assumed by Whittle et al (1993) as shown in Figure 7. The direct simple shear data is generated from the overconsolidation ratio and the normalised value of c_u/σ_{vo}' obtained from undrained direct simple shear tests on Boston blue clay (Ladd and Edgers1972) as given in Table 1. The DSS undrained strength profile adopted in the MSD calculations is shown in Figure 7.
- The till and the sand will be stiffer than the clay; however, a conservative approach is used by assuming that the stiffness of the till and the sand is the same as the stiffness of the clay. The undrained strength of the clay is extrapolated to the underlying layers of sand and till.

Table 1. Ratio of undrained shear strength to the effective vertical stress (c_{uo}/σ'_v) in the direct simple shear DSS of shearing mode for different overconcolidations ratios (OCR).

OCR	c_u/σ'_{vo}
1	0.21
2	0.34
4	0.60
8	0.96

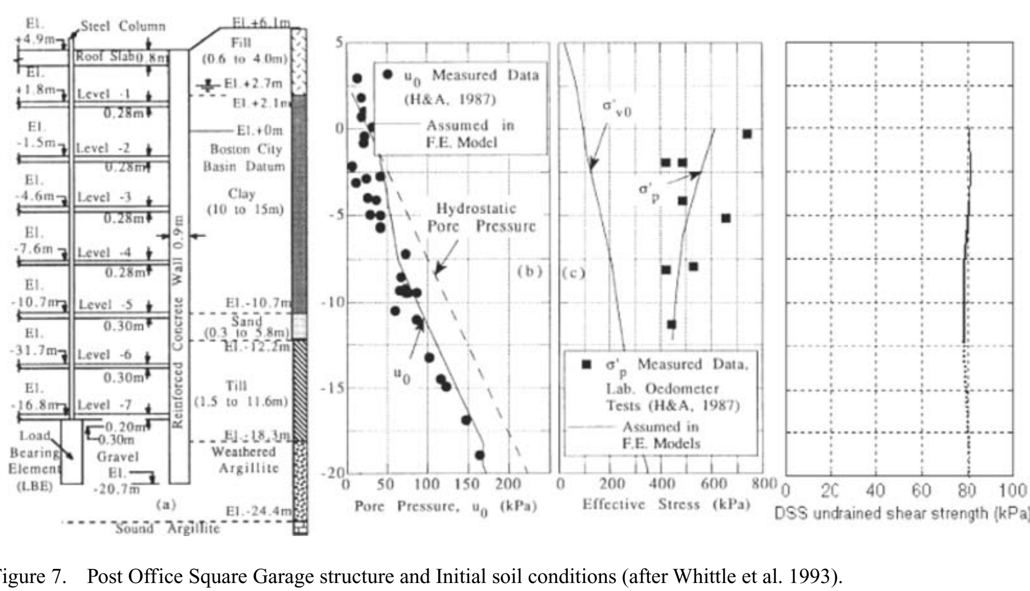


Figure 7. Post Office Square Garage structure and Initial soil conditions (after Whittle et al. 1993).

- The selection of a representative value for design is always a controversial issue in geotechnical engineering. The selection of a representative value for soil is complicated by the stress-strain behaviour. In soil, there is no unique relationship between strains and stresses. Many states of strain can correspond to a single state of stress or vice versa (Wood, 1990). The stress-strain relation in soil is complicated and subject to many factors such as stress history. Since the clay is assumed to govern the wall behaviour, and it is overconsolidated with OCR varying from 6 to 2, the stress-strain curve equivalent to OCR = 4 is taken as a representative curve (Figure 8).

Figure 9 presents measurements of lateral deflections of four inclinometers on the perimeter of the site compared with the MSD predictions. The inclinometers along Pearl Street show large displacements at the top of the wall. This is due to the fact that, prior to the roof construction, the excavation is unsupported with a maximum depth of 6 m and the diaphragm wall deforms in a cantilever mode. Measurements from the inclinometers along Milk Street show significantly less movement at the top of the wall. This is due to a difference in construction sequence. In these locations, the slab roof was constructed prior to the excavation. Two MSD calculations were carried out assuming 0 m

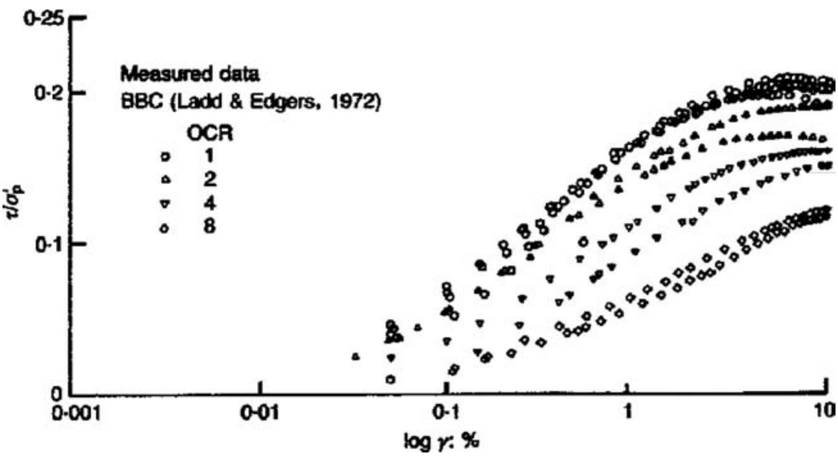


Figure 8. Stress-strain response for $K_o$consolidated undrained direct simple shear tests on Boston Blue Clay (Ladd and Edgers, 1972).

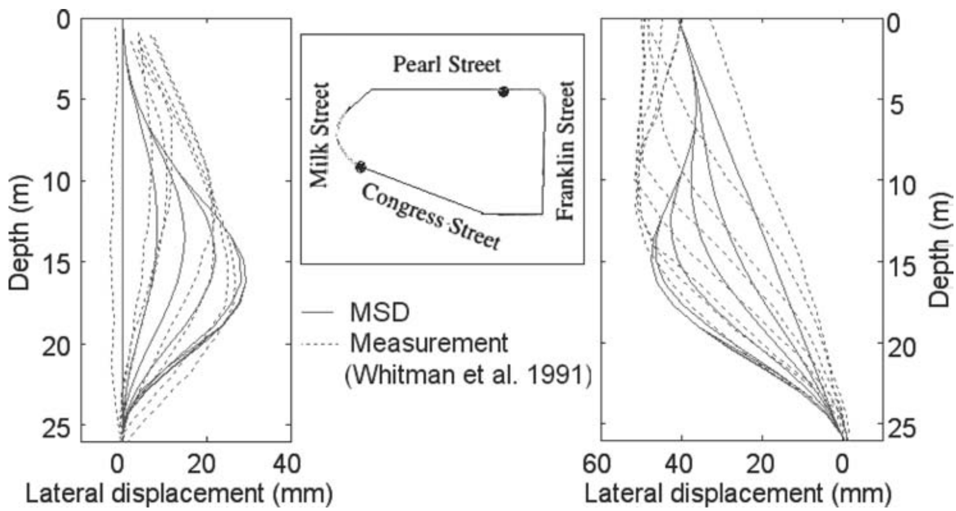


Figure 9. Wall movements during construction.

and 6 m of unsupported excavation. In the MSD the supports are assumed rigid; no shrinkage is allowed to occur at the roof slabs. This explains the small differences in the deformation profiles. However, MSD estimations for the maximum movements, generally, are in very good agreements with the measured data.

Figure 10 shows a wide scatter in the field settlement measurements of up to 50 mm at locations adjacent to the excavation. This is can be due to the variation of construction sequences and soil strata. Further away from the site, the observed settlement tapers off rapidly with maximum values ranging from 12 mm at a distance of 15 m to less than 3 mm at 40 m.

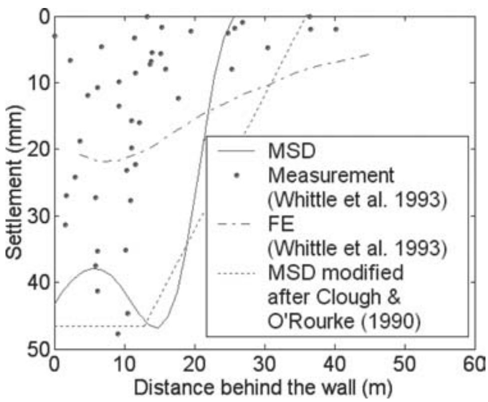


Figure 10. Comparison of predicted and measured surface settlement after the installation of the sixth floor at Boston Post Office Square Garage.

The localisation of settlement adjacent to the excavation is predicted successfully by the MSD method. The figure shows also that the maximum settlement predicted by the MSD method conforms well to the field measurements. However, the settlement trough predicted by the MSD method extends only to 25 m behind the wall.

Clough and O'Rourke (1990) suggested a more conservative trapezoidal settlement profile, based on observations from several case histories, for excavations in soft to medium clay. Their suggested profile assumed that the settlement has a maximum value at locations up to a distance away from the wall equal to 0.75 times the maximum excavation height; then the settlement decreases to zero at a distance equal to twice the maximum depth of the excavation. The method of Clough and O'Rourke (1990) is used to modify the MSD settlement profile as shown in Figure 10. The maximum settlement is assumed to be equal to the maximum lateral wall movement which is shown to be a conservative design expedient. Figure 10 shows a general agreement of the extent of the region of larger settlements between unmodified and modified MSD settlement profiles. Although the FE analysis carried out by Whittle et al. (1993) predicts the extension of the settlement trough away from the wall, it does not produce a conservative prediction for locations adjacent to the wall.

3.2 *The stata center, MIT campus, Cambridge*

The new Stata Centre at Massachusetts Institute of Technology campus was designed with a two storey basement for underground parking requiring a 13 m deep excavation.

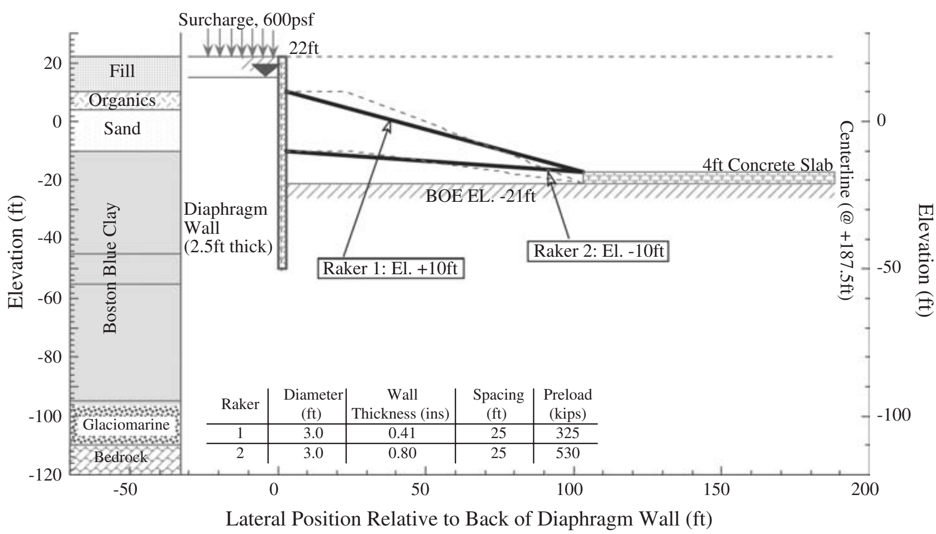
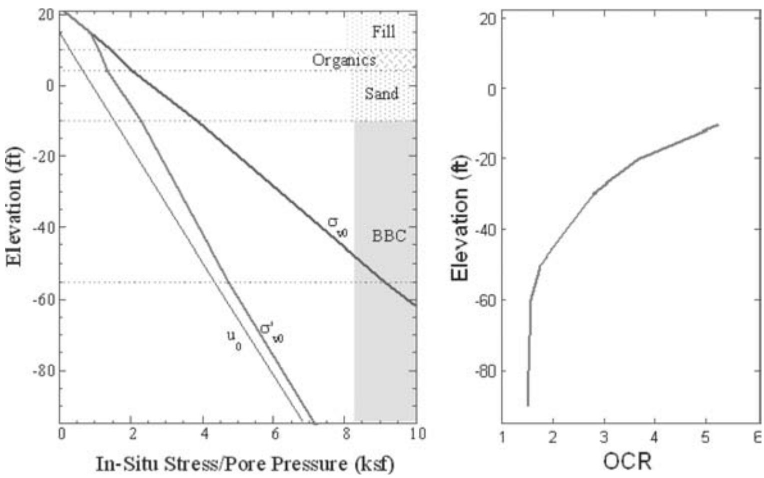
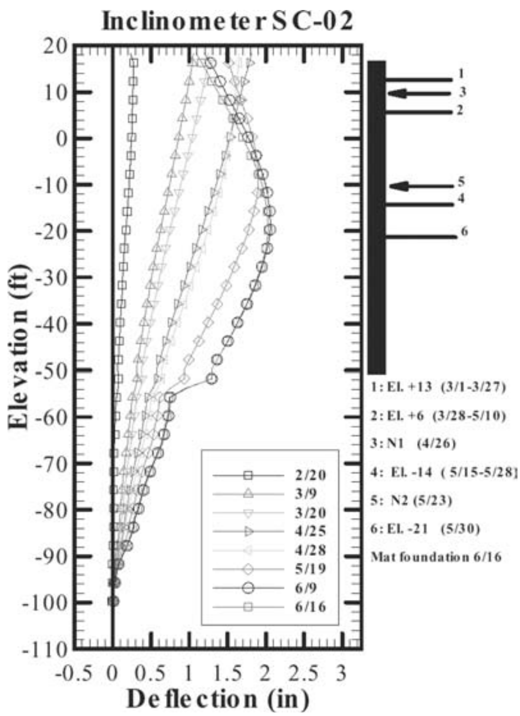


Figure 11. Cross section at the north wall (Olsen 2001).



(a)

Figure 12a. In-situ stress conditions (after Olsen 2001).



(b)

Figure 12b. Measured lateral displacements (after Olsen 2001).

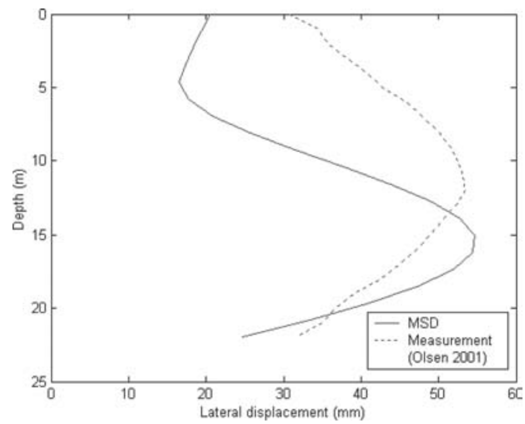


Figure 13. Comparison of measured and predicted lateral wall displacement at the end of the excavation at the north wall of Stata Center.

The site has a very large rectangular plan area (approx. 97 m × 118 m). The excavation is supported by a permanent perimeter diaphragm wall of a 9 m embedded depth. The wall is braced by a combination of pre-stressed tieback anchors, preloaded raker and corner bracing elements. The slurry wall was constructed during the period July – November 2000 and the excavation works were completed in June 2001. Construction details and performance of the structure is given in Olsen (2001).

The soil conditions consist of fill and organic soil followed by a layer of marine sand overlying a thick layer of medium stiff BBC that overlies a thin layer of glacially deposited soils and bedrock (Figure 11). Figure 12 shows the in-situ stresses and pore pressure distribution. The water table is located at a depth of about 1.2 m in the upper fill layer. The overconsolidation ratios of the BBC (Figure 12) were estimated from the measured undrained strength profile and the

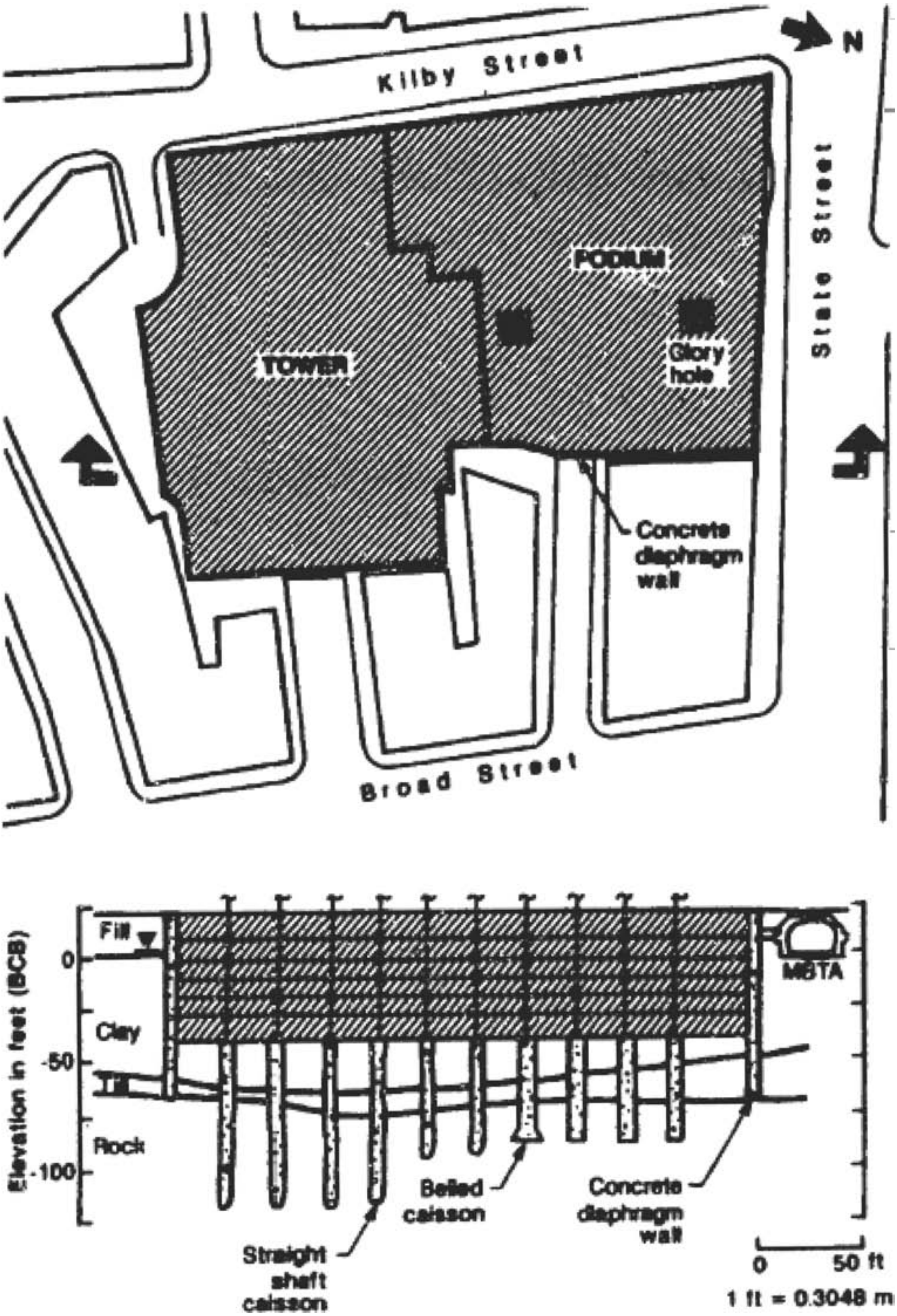


Figure 14. Plan and cross section at 75 state street (Becker and Haley 1990).

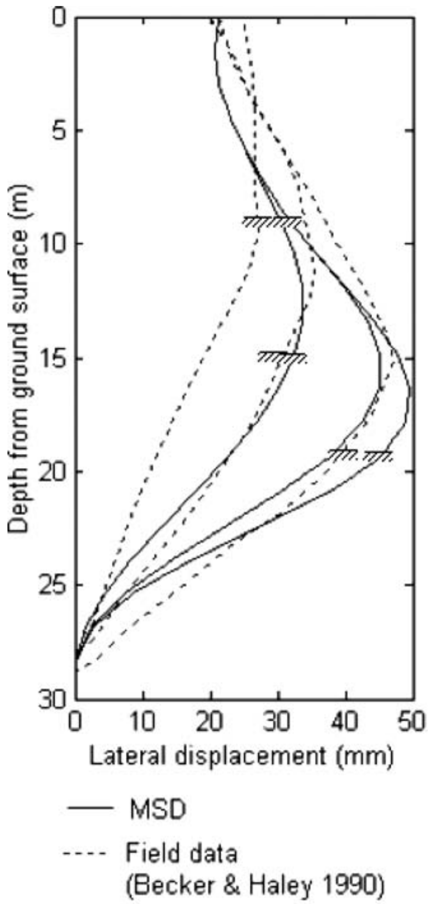


Figure 15. Measured and predicted wall deflection.

c_u/σ_{vo} – OCR relation obtained from DSS (Table 1). This figure shows that BBC has an overconsolisdation ratio (OCR) ranging from 5 to 1.5.

This case study uses the MSD method to evaluate the lateral earth support systems at the north wall supported by inclined rakers reacting against a 1.2 m thick central concrete base slab (Figure 11). The characteristic undrained stress-strain curve is selected according to the average soil stiffness of BBC was taken to be of OCR = 4.0 (Figure 8). since the wall is embedded into the medium to stiff Boston Blue clay layer, the end condition is assumed of the restrained type ($\alpha = 4/3$). Figure 13 shows the measured displacements, with the maximum measured deflection at the end of the excavation shown as 50 mm (2 in). This figure shows also that the soil beneath the base of the wall deform laterally following, approximately, the same deformation pattern of the retaining wall. This reinforces the assumption of having a single function to describe the deformation of both free-end walls and the soil beneath (Equation 1). In the MSD the supports are assumed rigid; no shrinkage is allowed to occur at the roof slabs. This explains the discrepancy in Figure 13. However, the MSD method predicts the maximum deflection with reasonable accuracy.

3.3 *75 state street, Boston*

The 31-storey office tower development at 75 State Street includes 6 levels of below-grade parking. The site has a plan area of 5500 m^2. The development includes 69500 m^2 of above-grade floor area and 35000 m^2 of below-grade space for a 700-car garage. The subsurface condition at 75 State Street consists of an overburden of fill on top of clay. The bedrock 21 to 31 m below ground surface is a highly altered argillite. A soil profile, along with a substructure section is shown in Figure 14.

The sub-grade structure was constructed using the Up/Down construction technique. The surrounding diaphragm wall, the permanent structural columns and foundations were all installed prior to the excavation using the slurry trench method. The wall extended down into the bedrock and is braced internally by the floor slabs, which are in turn supported by 84 interior columns composed of structural steel encased in concrete. The interior columns are connected to belled caissons and straight shaft caissons below the lowest floor level. Details of the construction method and the performance of the wall were documented by Becker and Haley (1990).

The MSD calculations were carried out assuming an average shear strength of 70 kPa which is a typical value in BBC. Figure 6 shows lateral displacement profiles at three different excavation stages. The DSS data of OCR = 4.0 (Figure 8) are used as representative stress-strain behavior in the MSD calculations. The MSD method overestimates lateral displacements in the early excavation stages (Figure 15). However, its predictions conform well to the measured data in the final excavation stages. The differences in the deformation profiles are due to rigid supports assumed in the MSD method. It can be argued that the overall functionality of the design method is more important than any local detailed deviations.

4 CONCLUSIONS

Plasticity theory in engineering practice has previously been confined to the prediction of collapse loads. However, the MSD method shows how non-linear materials exhibiting plastic hardening can be brought within the framework of simple plasticity theory through the assumption of a plastic deformation mechanism. The MSD method demonstrates the usefulness of Virtual Work not only in assessing the stability of retaining structures, but also in providing an estimate of working shear stresses that can lead directly to the prediction of compatible wall and ground movements.

Examples from the professional field of Geotechnical Engineering have been given to demonstrate the applicability of this modified plasticity theory to predict and control deformations around stiff-propped systems of braced excavations in soft clay soils

The non-linear displacements of braced excavations can be predicted successfully, accurately, and rather simply by respecting the equilibrium and compatibility of plastic deformations and by using the raw data of a well-chosen soil test. This challenges the current consensus of researchers and professionals, who invariably demand complex numerical analyses in critical cases.

REFERENCES

Becker, J. M., & Haley, M. X. (1990). Up/down construction—Decision making and performance. *Proceedings of the ASCE Conference on Design and Performance of Earth Retaining Structures, Geotechnical Special Publication No. 25*, ASCE, New York, 170–189.

Bolton, M. D. & Powrie. (1988). Behaviour of diaphram walls in clay prior to collapse. *Geotechnique* 38(2): 167–189.

Clough, G. W., Smith, E. M., & Sweeney, B. P. (1989). Movement control of excavation support systems by iterative design. *ASCE Foundation Engineering Congress,* ASCE, New York, N.Y., Vol.1, 869–884.

Clough, G. W., O'Rourke, T. D. (1990). Construction induced movements of insitu walls." *Design and performance of earth retaining structures,* Geotechnical Special Publication No. 25, ASCE, New York, N.Y., 439–470.

Ladd, C. C., and Edgers, L. (1972). Consolidated-undranined direct simple shear tests on Boston blue clay. *Res. Report R72–82,* MIT, Cambridge, Mass.

Olsen, M. B. (2001). Measured performance of a large excavation on the MIT campus. *SM thesis*, Dept. of Civil and Environmental Engineering, MIT, Cambridge, Mass., U.S.

O'Rourke, T. D. (1993). Base stability and ground movement prediction for excavations in soft clay. *Retaining structures*, Thomas Telford, London, 131–139.

Osman, A. S., & Bolton, M. D. (2004). A new design method for retaining walls in clay. *Canadian Geotechnical Journal*, 41(3): 451–466.

Osman, A. S., & Bolton, M. D. (2006a). Ground movement predictions for excavations in undrained clay", ASCE *Journal of Geotechnical and Geoenvironmental Engineering* 132(4):465–477.

Osman, A. S., & Bolton, M. D. (2006b). Design of Braced Excavations to Limit Ground Movements. *Proceedings of Institution of Civil Engineers: Geotechnical Engineering*, 159(3): 167–175.

Whittle, A. J. (1993). Evaluation of a constitutive model for overconsolidated clays. *Geotechnique,* 43(2): 289–313.

Whittle, A. J., Hashash, Y. M. A., and Whitman, R. V. (1993). Analysis of deep excavation in Boston. *ASCE Journal of Geotechnical Engineering*, 119(1): 69–90.