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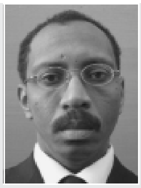

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Design of braced excavations to limit ground movements

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The authors have developed a new approach to the estimation of ground movements around braced excavations retaining thick deposits of soft clay, incorporating actual stress–strain data and the undrained shear strength profile of the soil on site. The method is based on the assumption of a plastic deformation mechanism local to a braced excavation, and which avoids any slippage on shear surfaces. Mobilised shear stresses beneath and around the face are found from an equilibrium calculation derived from the deformation mechanism by virtual work. The strains required to mobilise these stresses are finally entered into the deformation mechanism to predict boundary displacements. The outcome is a prediction based on simple calculations that otherwise would have called for elaborate constitutive modelling and finite element analysis. The capability of this mobilisable strength design (MSD) method in calculating the magnitude of wall displacements is demonstrated through the back-analysis of a previously published case history of a braced excavation at a soft clay site in Singapore. The relative effectiveness of heave-reducing piles in controlling ground movements is then demonstrated using the MSD method.

NOTATION

c_u undrained shear strength
D diameter of heave-resisting pile
l full wavelength of deformation pattern
l_p length of heave-resisting pile within plastic deformation mechanism for retaining wall
N_L ultimate lateral load factor
Vol volume of plastic deformation mechanism
y distance from lowest wall support
α wall-fixity end-condition parameter for braced excavation
α_p adhesion factor
β mobilisation strength ratio
γ total unit weight of soil
δu_p horizontal increments of relative displacement at heave-resisting pile centreline
δv vertical component of incremental displacement
δv_p vertical increments of relative displacement at the heave-resisting pile centreline
δw incremental wall displacement
δw_m maximum incremental wall displacement
ε_1, ε_3 major and minor principal strain respectively
ε_s engineering shear strain
$\varepsilon_{s,mob}$ average shear strain mobilised in soil

1. INTRODUCTION

Engineers do not currently have much confidence in predicting ground displacements, because the choice that has confronted them has been either to treat the soil as quasi-linear-elastic, or to face the task of validating and applying complex constitutive models in finite element analyses.

Practitioners are not generally aware that some simple but reasonably accurate calculations are available to predict ground displacements around retaining walls in clay, as first illustrated by Bolton and Powrie[1] for cantilever walls. This approach is called the mobilisable strength design (MSD) method. It is set out in more detail, together with validations for excavations against cantilever walls, by Osman and Bolton,[2] and has recently[3] been extended to the calculation of ground movements during the construction of braced excavations in clay.

The three basic elements of this approach are set out below.

(*a*) Simple plastic deformation mechanisms are used, which feature displacement fields and compatible distributed strains that have been derived from the examination of closely monitored field studies, from analogous centrifuge models, or from finite element simulations. These mechanisms avoid slip displacements on any surfaces and may, in principle, be applicable either to small deformations or to collapse.
(*b*) The virtual work principle is used to calculate the shear strength that must be mobilised in the soil at each stage of excavation, by relating plastic work inside the mechanism to loss of potential energy associated with base heave. It is assumed that the profile of mobilised strength with depth will follow the predetermined profile of peak strength, with a simple reduction factor β. The approach resembles upper-bound calculations of collapse, but is also applicable to working conditions.
(*c*) Raw stress–strain data from direct simple shear tests on undisturbed samples, taken from locations representative of the assumed zone of deformation, are used directly to relate the proportion β of strength mobilisation to the magnitude of observed shear strain. The average shear

strain inferred from the mobilisation curve is finally inserted back into the assumed deformation mechanism in order to calculate ground displacements and wall deformations at each stage of excavation.

The key advantage of this approach for practising engineers is that they can use a stress–strain curve from an appropriate soil test, together with a simple calculation, to calculate both stability and wall displacements without the need for constitutive laws and complex computer calculations.

2. PLASTIC DEFORMATION MECHANISM FOR BRACED EXCAVATIONS

Following O'Rourke,[4] 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 (Fig. 1).

$$\delta w = \frac{\delta w_{\mathrm{m}}}{2}\left[1 - \cos\left(\frac{2\pi y}{l}\right)\right] \quad (1)$$

where δw is the incremental wall displacement at any distance y from the lowest wall support, δw_{m} is the maximum incremental displacement, and l is the full wavelength of the deformation pattern.

Figure 2 shows a new plastic deformation mechanism proposed by Osman and Bolton[3] for such an incremental lateral displacement. 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 deform compatibly, and the soil deformation profile follows the cosine function of equation (1). The dimensions of the proposed mechanism depend on the wavelength l, which is related to the length s of the wall beneath the lowest support by

$$l = \alpha s \quad (2)$$

The value of α depends on the wall-toe fixity condition,[4] as shown in Fig. 2.

The average shear strain increment $\delta\varepsilon_{\mathrm{s,mob}}$ mobilised in the soil within the assumed displacement field can be linked to the maximum incremental displacement by

$$\delta\varepsilon_{\mathrm{s,mob}} = \frac{\int_{Vol} \delta\varepsilon_{\mathrm{s}}\,\mathrm{d}Vol}{\int_{Vol} \mathrm{d}Vol} \approx 2\frac{\delta w_{\mathrm{m}}}{l} \quad (3)$$

At each stage of the excavation the strength c_{mob} mobilised

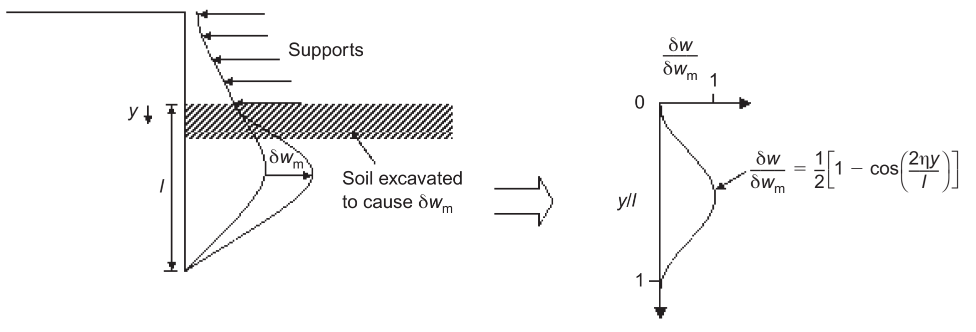

Fig. 1. Incremental displacement in braced excavation (after O'Rourke[4])

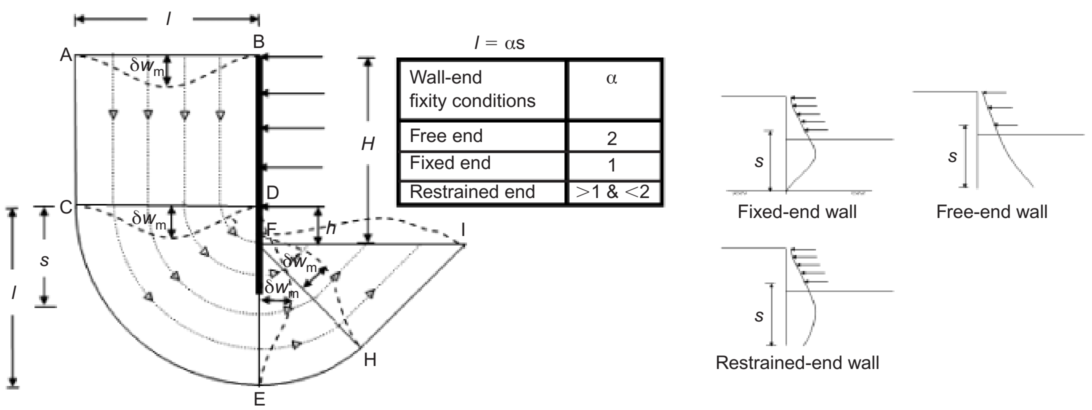

Fig. 2. Plastic deformation mechanism for braced excavation in clay

owing to the excavation of soil beneath the lowest support can be found using the principle of virtual work[5] by balancing the virtual loss of potential energy to the virtual plastic work in distributed shearing.

$$\int_{Vol} \gamma \delta v \mathrm{d} Vol = \int_{Vol} \beta c_u \delta \varepsilon_s \mathrm{d} Vol \quad (4)$$

where γ is the total unit weight of the soil; δv is the vertical component of the incremental displacement; $\delta \varepsilon_s$ is the engineering shear strain, defined as the difference between the major $\delta \varepsilon_1$ and minor $\delta \varepsilon_3$ principal strain increments, $\delta \varepsilon_s = |\delta \varepsilon_1 - \delta \varepsilon_3|$, and β is the proportional strength mobilised ($\beta = c_{mob}/c_u$).

If is assumed that the profile of mobilised strength with depth will follow the predetermined profile of peak strength, with a simple constant reduction factor β, then

$$\beta = \frac{\int \gamma \delta v \mathrm{d} Vol}{\int c_u \delta \varepsilon_s \mathrm{d} Vol} \quad (5)$$

A detailed derivation and validation are given in reference 3.

The corresponding mobilised shear strain $\varepsilon_{s,mob}$ is found from direct simple shear tests on undisturbed samples, taken from locations representative of the assumed zone of deformation. The maximum incremental wall movement is then calculated from the corresponding increment in shear strain (equation (3)). The incremental wall displacement profile is then plotted using the cosine function of equation (1). 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.

A wall often deforms in a cantilever mode before the installation of the first support level. Clough *et al.*[6] suggest that the movements due to the cantilever mechanism and bulging mechanism can be added together to obtain the final movement. Bolton and Powrie[1] and Osman and Bolton[2] showed that displacements around a cantilever retaining wall can be idealised by the deformation mechanism shown in Figure 3. In this mechanism, the mobilised shear strain $\varepsilon_{s,mob}$ 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 5. The corresponding mobilised shear strain $\varepsilon_{s,mob}$ is found from the representative stress–strain curve. This cantilever movement then defines the initial ground displacement profile prior to propping, and the subsequent bulging displacements are added as illustrated in Figure 4.

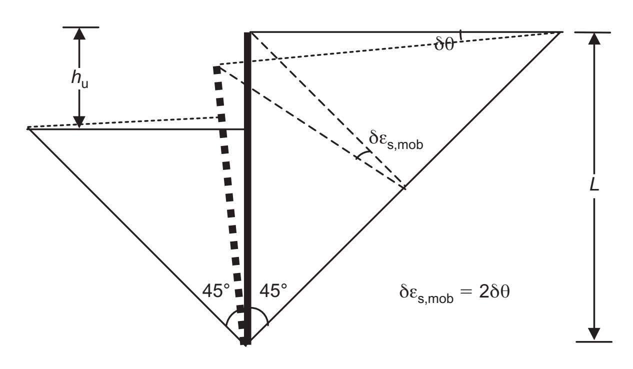

Fig. 3. Plastic deformation mechanism for cantilever retaining walls in undrained conditions

The maximum settlement behind the wall calculated by the MSD method is always equal to the maximum lateral wall movement, and this might be a conservative prediction.[7] Bending moments and shear forces can be calculated from the deflected shape and the wall stiffness; however, the assumed cosine shape of the plastic deformation mechanism (Fig. 5) might lead to overprediction.

The MSD method requires engineers to obtain stress–strain data from direct simple shear tests on high-quality samples, taken from locations representative of the assumed zone of deformation. Lafebvre *et al.*[8] and Lafebvre and Pfender[9] proved that the Sherbrooke block sampler[10] preserves the natural

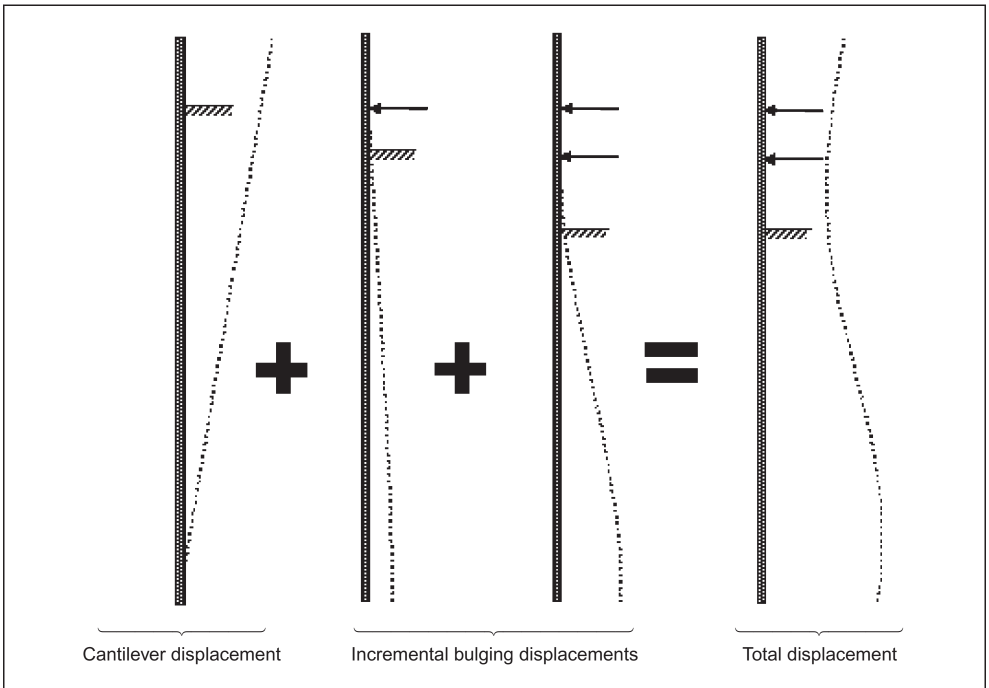

Fig. 4. Calculation of wall total displacement in the MSD method

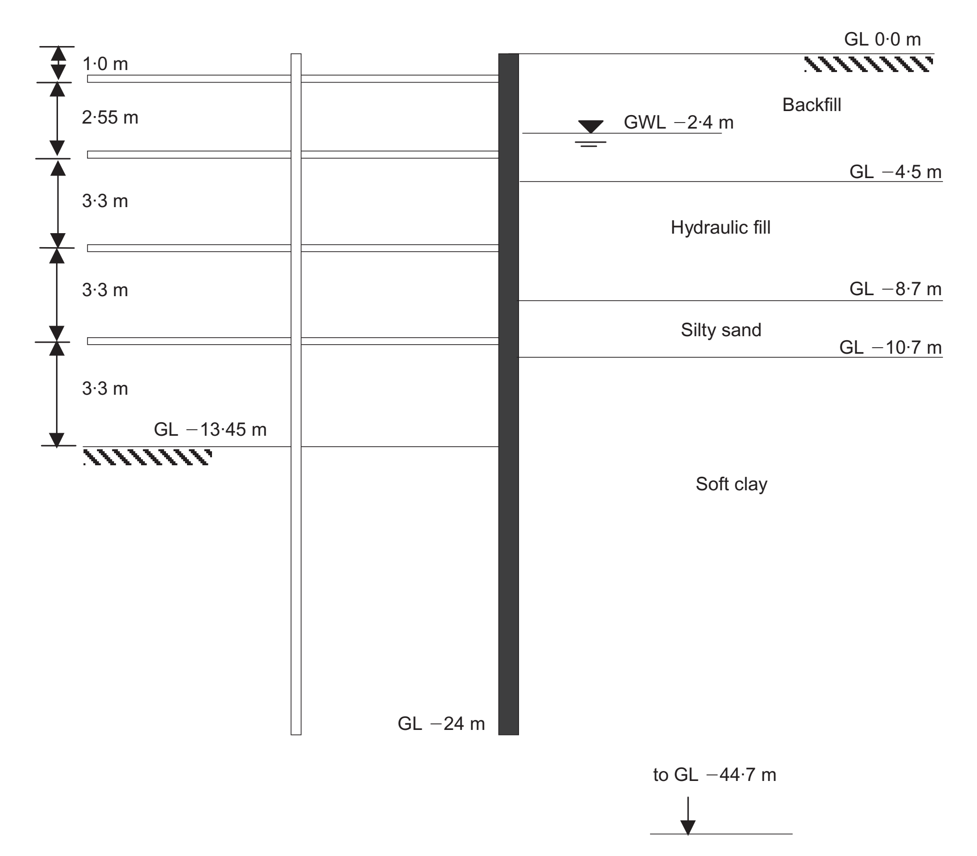

Fig. 5. Cross-section at Taipei case (after Su *et al.*[12])

extrapolated upwards as shown in Fig. 6. The embedded depth was assumed to be sufficient to restrain the movement at the wall toe. The parameter α of equation (2) is taken to be equal to 4/3, which gives a point of inflection at the bottom of the wall. The average unit weight γ of the soil is taken to be 20 kN/m^3, which is consistent with another excavation site at Taipei.[13] The factor of safety (FS) against basal instability in the conventional calculation is often defined as the undrained shear strength divided by the mobilised strength (c_u/c_{mob}). Fig. 7 shows FS calculated from the inverse of the strength ratio β in equation (5) plotted against excavation depth. At an excavation depth of 13·45 m, corresponding to observed failure on site, MSD calculated an FS of 0·98, which is in close correspondence with the required value of 1·00.

structure and the intact characteristics of the soil. This sampler was used in the sensitive clay in eastern Canda[10] and in the soft clay of Bothkennar in Scotland,[11] and proved capable of producing the highest obtainable quality of samples.

3. STABILITY ASSESSMENT

Stability requirements of deep braced excavations in soft clay often control the design. Evaluation of base stability can play a key role in assessing displacements. The MSD method calculates the mobilised undrained strength at each stage of the excavation using virtual work by balancing the loss of potential energy against the work dissipated in distributed shearing. If the mobilised strength is equal to the undrained strength ($\beta = 1$), then MSD can be considered to give an upper bound to the collapse load in conventional terms.

To demonstrate the plausibility of MSD in assessing the stability of a full-scale excavation, an actual failure case that occurred in Taipei[12] is back-analysed. The site is about 100 m long, 17·5 m to 25·8 m wide and 13·45 m deep. The excavation was supported by a 0·7 m thick diaphragm wall 24 m deep. The vertical spacing between the struts varied between 2·55 m and 3·5 m (Fig. 5). The excavation site was located on reclaimed land in the Taipei Basin. The soil profile comprises 8·7 m backfill and hydraulic fill materials, 2·0 m silty sand layer, 30 m soft clay, 11·3 m silty sand, and bedrock. An in situ undrained strength profile is shown in Fig. 6.

In MSD calculations the undrained strength profile of the clay was based on field vane data. In the absence of any additional information regarding the strength of the upper layers of fill and sand, the strength profile of the deeper clay was simply

4. DESIGN EXAMPLE: UNITED OVERSEAS BANK IN SINGAPORE

The applicability of MSD in design and decision-making is demonstrated through the back-analysis of a case history of an excavation in Singapore marine clay.

4.1. The site

The new headquarters of the United Overseas Bank in Singapore, UOB Plaza, is located between Chulia Street and Boat Quay in

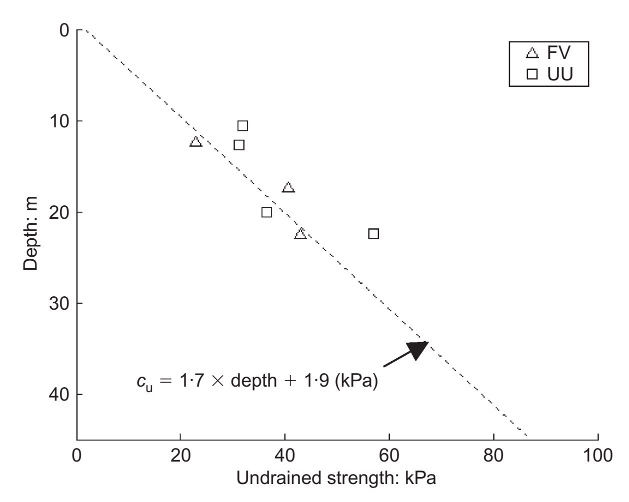

Fig. 6. Comparison of undrained shear strength distribution obtained from different test methods (data taken from Su *et al.*[12])

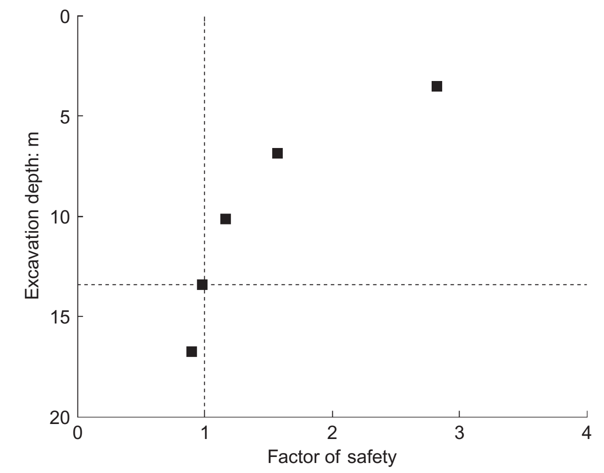

Fig. 7. Change of factor of safety with excavation depth in Taipei case history

the central business district adjacent to Singapore River. The structure comprises a 66-storey tower, podium and basement car park, which were constructed in the 1990s adjacent to the old 30-storey UOB tower (Fig. 8). The basement excavation was 10 m to 16 m deep. The excavation was supported by a permanent perimeter diaphragm wall of a maximum depth of 40 m. The thickness of the wall varied from 0·8 to 1·2 m, and it was supported by three levels of propping. The performance of the wall during construction is described by Wallace *et al.*[14]

The site is underlain by up to 30 m of soft marine clay existing as distinct upper and lower layers. The lower marine clay is underlain by a hard bouldery clay. Fig. 9 shows the undrained strength profile. The solid line shows the average undrained strength. The upper layer is high-plasticity clay with a plasticity index of 70%; the undrained strength obtained from field vane tests ranges from 20 to 40 kPa. The lower layer has a plasticity index of 45%; the undrained strength ranges between 40 and 60 kPa. The properties of Singapore marine clay are documented by many authors.[15,16]

4.2. Back-analysis

The MSD method uses a single stress–strain curve to predict displacements around braced excavations. Fig. 10 shows normalised stress–strain curves (β against ε_s) obtained from Norwegian Geotechnical Institute (NGI) direct simple shear tests on Singapore soft marine clay samples.[16] Each sample was tested from an initial normal effective stress of 250 kPa. Tanaka *et al.*[15] showed that the overconsolidation ratio (OCR) of lower marine clay is within the range 1·30–1·45. Therefore the normalised stress–strain curve for OCR = 1 is taken as the representative curve in the MSD calculations.

Because the wall is embedded in the stiff boundary clay, the parameter α of equation (2) is taken to be equal to 1·0, which gives zero wall movement at the toe. Fig. 11 shows that MSD overestimates lateral displacements in the early excavation stages, possibly because of inaccurate small-strain measurements in Fig. 10. However, it predicts a maximum cumulative wall movement of 60 mm at an excavation depth of 13 m compared with 54 mm from field measurements. The non-linear FE analyses carried out by Simpson,[17] in which the BRICK model is used, reproduces the general shape of wall lateral deformations; however, it appears to overestimate the maximum displacements at the end of the excavation process. In MSD no mathematical expressions are needed to model the constitutive behaviour of soil. The predictions were based directly on a single stress–strain curve obtained from a laboratory test on a representative sample. Note that in the MSD method the supports are assumed rigid, no cracks are permitted between the soil and the wall, and the lack of downward movement of the soil in contact with the wall implies sufficient wall friction to prevent slippage. These simplifying assumptions partly explain the discrepancy between the observed and the predicted wall deformation profiles shown in Fig. 11.

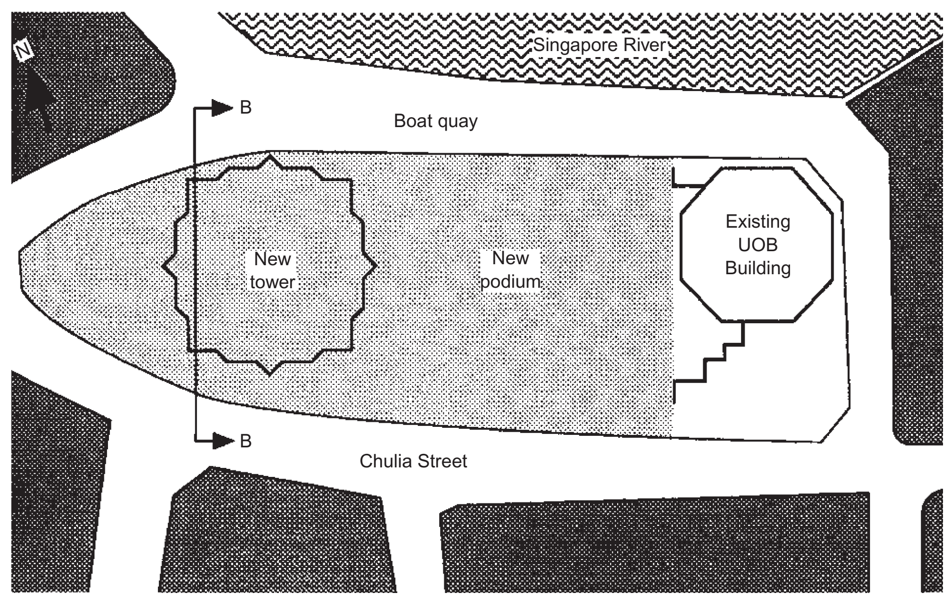

Fig. 8. United Overseas Bank site location[17]

4.3. Alternative design options: different support spacing

In the existing design three levels of support were used. The MSD estimates a maximum displacement of 60 mm at the end of the excavation, achieved with a factor of safety of 1·75 (Fig. 12). The effect on displacement and stability of reducing the number of supports is easily studied using MSD. Two design alternatives are examined: a single level of props at 2 m depth (i.e. at the location of the existing first level of props), and two levels of props at depths of 2 m and 7·5 m. Table 1 summarises the MSD predictions. The

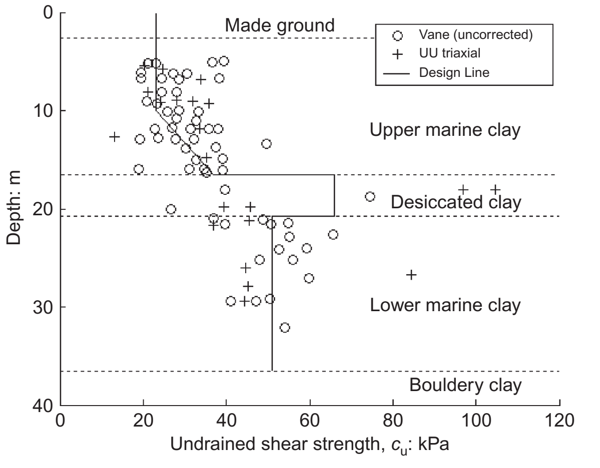

Fig. 9. Undrained strength profile at UOB (after Wallace *et al.*[14])

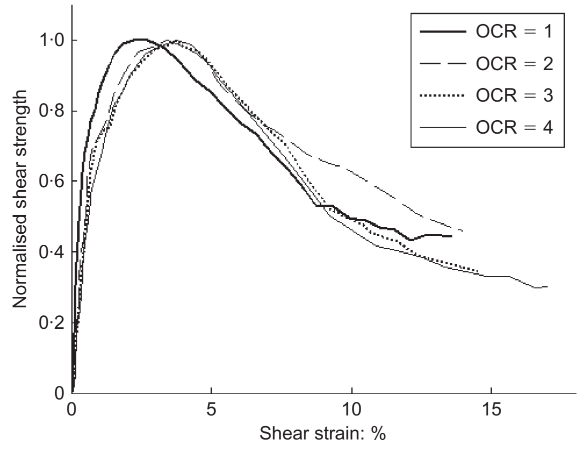

Fig. 10. Stress–strain curves obtained from NGI direct simple shear tests on samples of Singapore soft marine clay (data adapted after Chu and Choa[16])

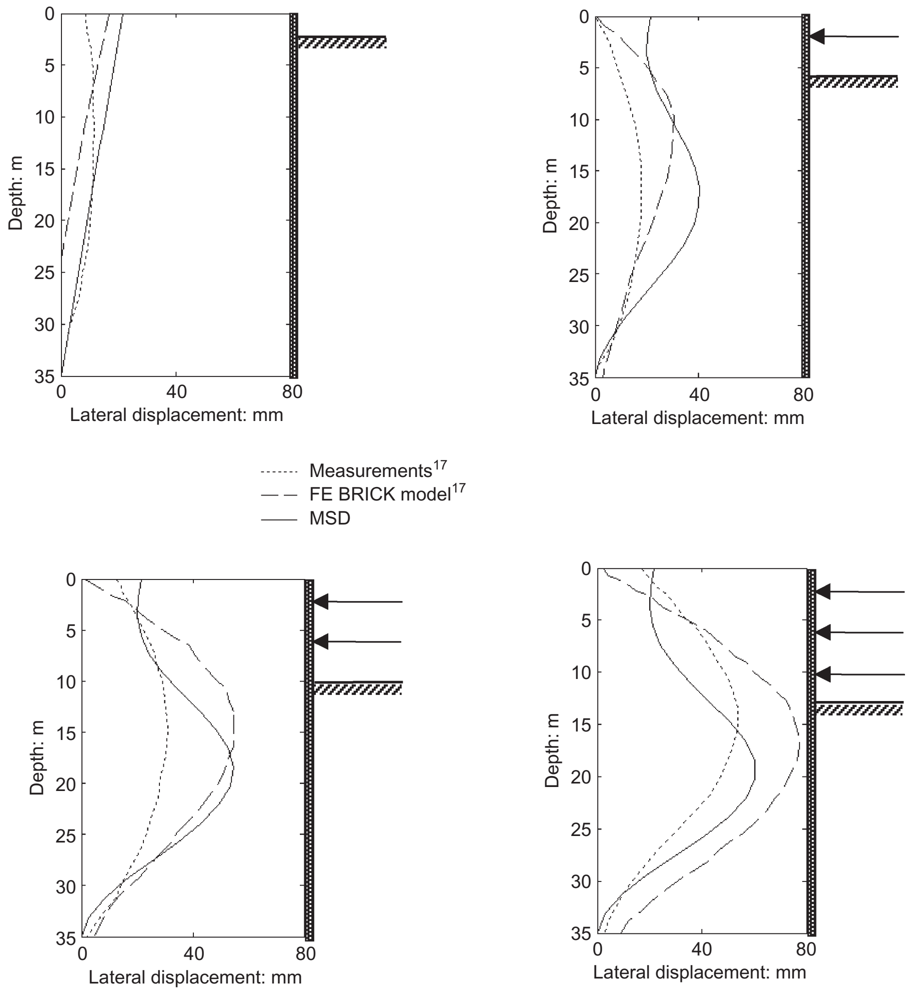

Fig. 11. Comparison between observed and predicted lateral wall displacements at UOB, Singapore

MSD method indicates that the displacement would have more than doubled had a single prop level been used. The factor of safety calculated by MSD could be checked against the safety requirements suggested by the code of practice. Using a single level of props, for example, leads to a safety factor of 1·30, which falls below the partial factor of 1·4 required by Eurocode 7[18] and the mobilisation factor of 1·5 required by BS 8002[19] for the undrained strength of soil.

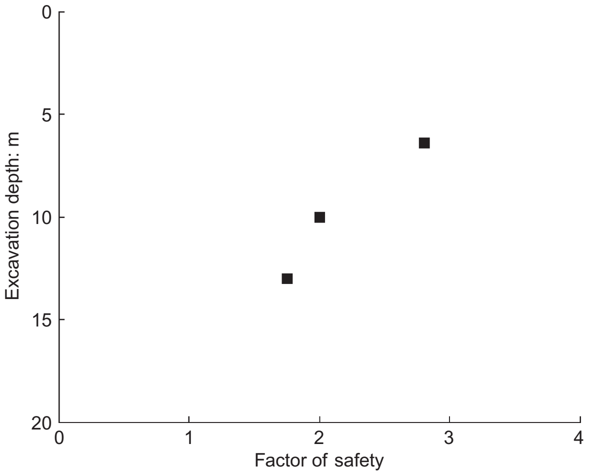

Fig. 12. Change of factor of safety with excavation depth in UOB case history

Number of levels of props	Maximum lateral displacement: mm	Factor of safety
Single level	125	1·30
Two levels	73	1·54
Three levels	60	1·75

Table 1. Influence of number of prop levels on displacement and on stability of UOB case history predicted by MSD

For two levels of props, the displacements predicted by MSD would increase by about 22% to 73 mm with a factor of safety of 1·54.

4.4. Alternative design: using heave-resisting piles in front of the wall

The construction of excavations and foundations in urban areas requires control of the surrounding ground surface, because excessive ground movements damage adjacent facilities. In recent years there has been an increasing use of heave-resisting piles as a mean of reducing ground deformations.

Randolph and Houlsby[20] developed an exact solution for the problem of the ultimate lateral resistance of circular piles in undrained clay due to purely horizontal movement. If the lateral load P is non-dimensionalised with respect to the soil strength c_u and the diameter of the pile D, it is found that the ultimate lateral load factor N_L $(= P/c_uD)$ per unit length of a pile varies between 9·14 for a perfectly smooth pile and 11·94 for a perfectly rough pile. Table 2 shows the variation of N_L with the adhesion factor α_p, which is defined as the ratio of the pile interface shear strength to the undrained soil strength c_u.

Adhesion factor, α_p	Ultimate lateral load factor, N_L
0·0	9·142
0·1	9·527
0·2	9·886
0·3	10·220
0·4	10·531
0·5	10·820
0·6	11·088
0·7	11·336
0·8	11·563
0·9	11·767
1·0	11·940

Table 2. Values of normalised limiting pressure for a pile due to purely horizontal movements[20]

In order to obtain simplified predictions of displacements around propped retaining walls with heave-resisting piles

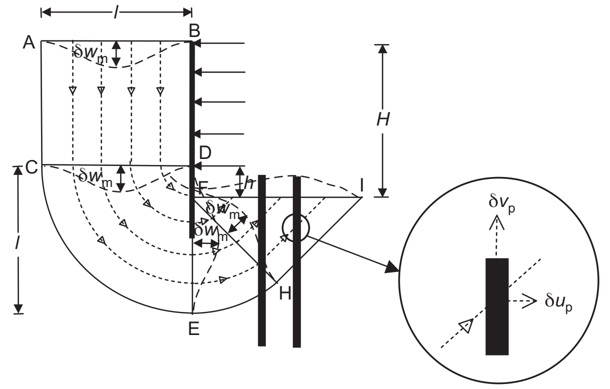

Fig. 13. Plastic deformation mechanism for braced excavations with heave-resisting piles

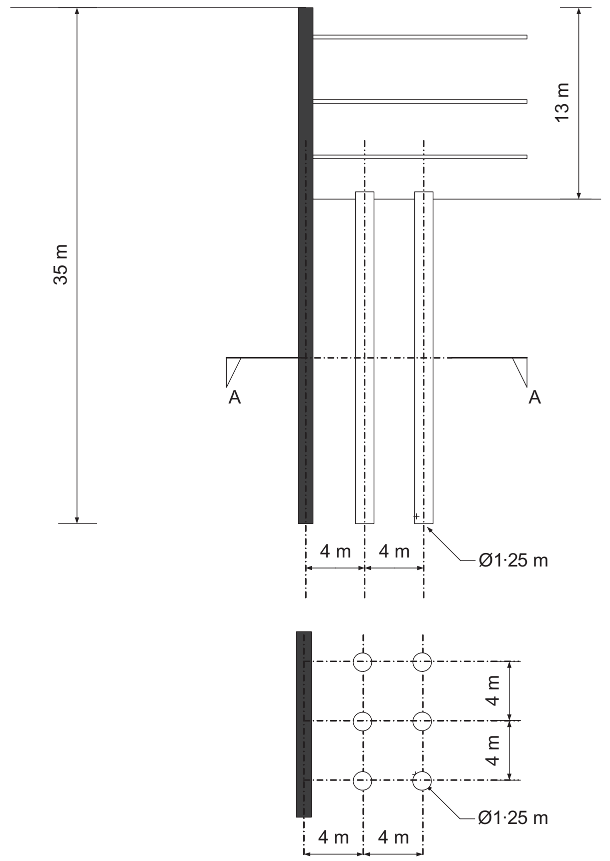

Fig. 14. Layout of piles in a possible design alternative to UOB case history

installed in front of the walls, it is assumed that the heave-reducing piles do not alter the global plastic displacement mechanism around a braced excavation (Fig. 2), but that they increase stability owing to the work dissipated at a displacement discontinuity representing the circumferences of the piles (see Fig. 13). Therefore equation (5) can be modified as

$$\beta = \frac{\int \gamma \delta v \mathrm{d} Vol}{\int c_\mathrm{u} \delta \varepsilon_\mathrm{s} \mathrm{d} Vol + \int_0^{l_\mathrm{p}} N_\mathrm{L} c_\mathrm{u} D |\delta u_\mathrm{p}| \mathrm{d} l + \int_0^{l_\mathrm{p}} \alpha_\mathrm{p} c_\mathrm{u} |\delta v_\mathrm{p}| \pi D \mathrm{d} l} \tag{6}$$

where l_p is the length of the pile in the deformation zone, and δu_p and δv_p are, respectively, the horizontal and vertical increments of relative displacement at the pile centrelines.

Here, the *local* strain around the piles is taken to be sufficient to cause full plastic yielding even when the *global* strains in the overall mechanism are small. This is felt to be reasonable, because the ratio of the strains will be proportional to l/D (wavelength to pile diameter). The calculations will therefore tend to overestimate the influence of piles.

In order to study the possible effects of heave-resisting piles on displacements, the case history of UOB was back-analysed with rough heave piles of 1·25 m diameter assumed to be installed in two rows at 4 m and 8 m in front of the wall with 4 m spacing between the piles in each row, as shown in Fig. 14. Fig. 15 compares the lateral displacement of this wall–piles configuration with the previous results obtained without heave piles (Fig. 11). The heave piles appear to reduce the maximum lateral displacements from 60 mm to 29 mm (by about 52%). A similar magnitude of reduction was reported by McNamara[21] for centrifuge tests in which a similar arrangement of piles was used. However, it should be borne in mind that the plastic deformation mechanism shown in Fig. 13 was developed for excavations in soft clay supported by rigid props. The compressibility of the props in McNamara's centrifuge tests hinders the direct comparison between MSD and the centrifuge tests.

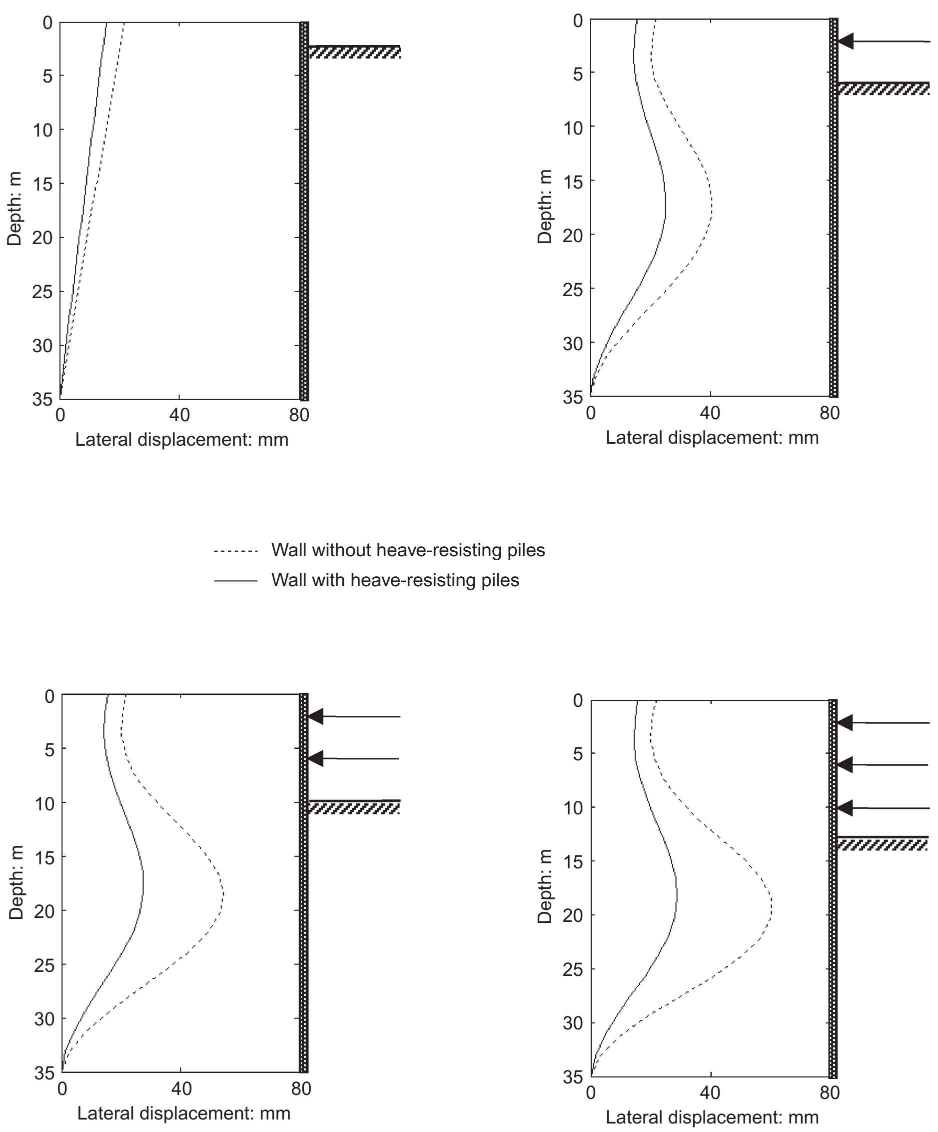

Fig. 15. Influence of heave-resisting piles on lateral displacements of propped walls

5. 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.

A new method of analysis of heave-reducing piles installed in front of well-propped excavations in soft clay is developed. Heave-reducing piles can be an efficient means of reducing displacements around braced excavation by a factor of 2, as shown in the design example given in this paper.

6. ACKNOWLEDGEMENTS

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