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MSD applied to the construction of the British Library basement: a multi-stage excavation in London Clay

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Abstract

This note presents the application of the mobilisable strength design (MSD) method to the monitoring results of the multi-propped excavation in the south area of the British Library Euston, constructed in a highly overconsolidated stiff clay deposit. The MSD method is an energy-based approach (a nonlinear finite-element method for a single-degree-of-freedom soil-wall system) introduced to develop a simplified design methodology that satisfies both ultimate and serviceability limit states. Wall displacement predictions based on the MSD method are compared with considerable field monitoring data. The sensitivity of the method to reasonable variations in input parameters is considered. A spreadsheet and python code demonstrating the MSD analysis from this paper are provided in the online supplement alongside details of the mathematical formulation.

Key words: mobilisable strength design (MSD), braced excavation, British Library Euston

Introduction

The mobilisable strength design (MSD) method for analysis of geotechnical structures offers a dependable yet quick and simple method to quantify ground movements caused by geotechnical construction. This method provides a robust and relatively simple analysis (suitable for spreadsheet calculations) that allows the incorporation of soil material nonlinearity and is based on routinely determined parameters and clear assumptions. The MSD method was originally developed at the University of Cambridge from displacement mechanisms observed during centrifuge testing (Powrie 1986; Bolton and Powrie 1987, 1988) and is underpinned by principles of similarity introduced by Skempton (1951). This design process was developed for various geotechnical constructions including bridge abutments (Bolton et al. 1990; Bolton and Sun 1991*a*), shallow and deep foundations (Bolton and Sun 1991*b*; Osman and Bolton 2004*a*, 2005; Klar and Osman 2008; Bouzid et al. 2013), and tunnelling (Osman et al. 2006).

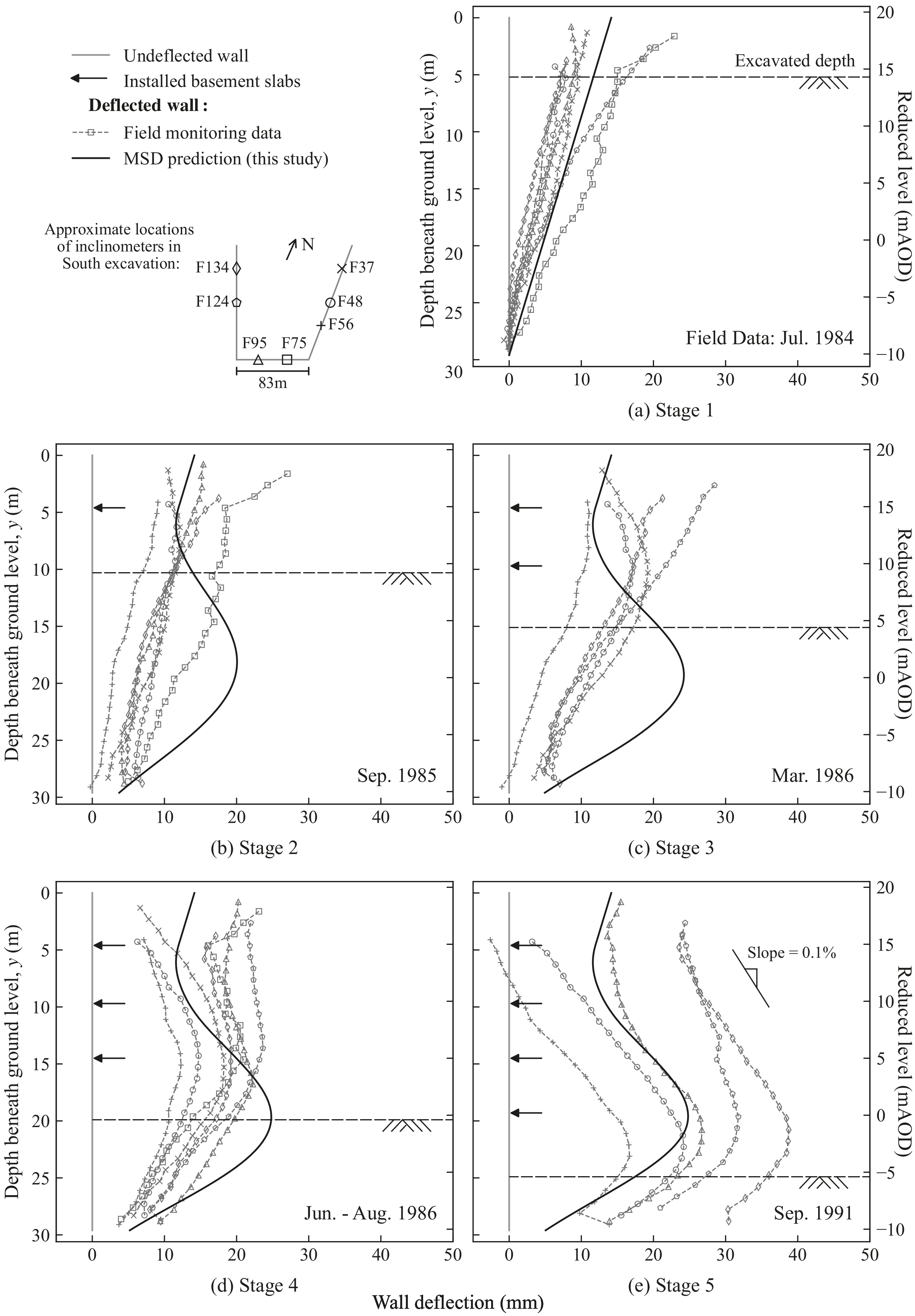
Additionally, MSD principles have been used to analyse deep excavations, inspired by earlier work developing approximate analytical methods (e.g., Clough et al. 1989; O'Rourke 1993). These include cantilever retaining walls (Osman and Bolton 2004*b*, 2004*c*; Li and Bolton 2014; Wang et al. 2018; Deng and Haigh 2022), braced excavations (Osman and Bolton 2006*a*, 2006*b*), flexible walls (Diakoumi and Powrie 2013; Deng et al. 2021; Madabhushi and Haigh 2022), and narrow excavations (Lam and Bolton 2011).

In comparison to finite element analysis (FEA), this approach relies on an assumed displacement mechanism, is limited to simpler constitutive behaviour (e.g., cannot include stress dependent stiffness directly), and would require reformulating to incorporate additional problem elements (such as flexible props and wall roughness). However, it offers a quick and simple alternative to FEA, particularly during early stages of design, to predict displacements from simplified nonlinear soil constitutive models (or even directly from site-specific element test data) with a much simpler mesh, boundary conditions, and calculations that can be carried out using a spreadsheet.

The validity of the MSD method has previously been investigated by comparison with FEA (e.g., Lam and Bolton 2011) and also in the context of deep excavation design practice in Sweden by Bjureland (2013). Bolton et al. (2014) used the database of excavation records from the thesis of Xu (2007) along with the principles presented in Lam and Bolton (2011) to develop pertinent dimensionless groups and hence produce new design charts for predicting excavation performance in soft Shanghai soils. Similarly, Deng et al. (2021) modified the MSD approach and successfully applied it to case studies in Dublin Boulder Clay and Oslo Clay.

In this note, predictions of wall displacements using the MSD method are compared with field monitoring data from

Fig. 1. Predicted wall displacements (black; this study) against field monitoring data (grey; data and location sketch from Simpson and Vardanega 2014, including Appendix W3) for each excavation stage. Soil parameters are given as mean values in Table 1 ($EI = 2192$ MNm^2/m, $\alpha_\lambda = 1.2$).



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the staged construction of the British Library Euston excavation. A preliminary analysis of this case study using the MSD method is presented in Campbell (2017). The MSD method employed here is based on that presented in Lam and Bolton (2011) (extending the work of Osman and Bolton 2006*b*) with some minor simplifications (primarily no discretisation in horizontal cells of the mechanism with depth).

Case study: British Library Euston excavation

This high-profile project was completed many years ago and is described in detail by Ryalls and Stevens (1990), Stevens and Ryalls (1990), and St John Wilson (1998). Further details related to the instrumentation and anchorage design for the project are provided by Loxham et al. (1990) and Raison (1987*a*, 1987*b*, 1988). The wall/excavation details and monitoring data are detailed by Simpson and Vardanega (2014) including detailed site progress drawings in Appendix W5 of this publication. This note examines the excavation and staged construction of the South area (see plan view in Fig. 1), detailed as follows:

1) A wall length of 29.6 m is supporting a 25 m excavation below ground level at 19.5 metres above ordinance datum (mAOD). The wall construction is detailed in Ground Engineering (1984).
2) The excavation was carried out top-down in five stages to the following reduced levels [14.3, 9.2, 4.4, −0.4, and −5.4] (mAOD).
3) Slabs of 0.4 m thickness were constructed 0.6 m above each excavation level, which act as props supporting the wall.

Soil properties

The wall was constructed in ~2 m of fill, 18 m of London Clay, and 10 m of Lambeth Group deposits (described as clay) which continue below the base of the wall (Simpson and Vardanega 2014). Analysis of some of the soil properties for the site in question is presented in Simpson et al. (1981) with further analysis conducted in Vardanega et al. (2012a, 2012*b*) as explained in Simpson and Vardanega (2014). London Clay has been the subject of many high-quality testing studies (e.g., Gasparre 2005; Gasparre et al. 2007*a*, 2007*b*; Hight et al. 2007; Kamal et al. 2014). Therefore, as the London Clay and Lambeth Group are similar materials and the former makes up most of the excavated depth and likely dominates the response, London Clay properties are used as representative values throughout the excavation. For this analysis, a power-law soil constitutive relationship was selected to describe the soil behaviour (Vardanega and Bolton 2011*a*, 2011*b*):

$$\frac{\tau}{s_u} = \frac{1}{2}\left(\frac{\gamma}{\gamma_{50}}\right)^b \qquad (1)$$

where τ and γ are the mobilised soil engineering shear stress and strain, respectively, s_u is the soil undrained shear strength, γ_{50} is the soil shear strain when 50% of the shear strength is mobilised, and b is a soil nonlinearity exponent. Vardanega and Bolton (2011*a*) calibrated this model against high-quality test data in London Clay (parameters given in Table 1). Different soil parameters could be selected for different zones within the assumed deformation mechanism dependent on their expected shear failure mode (discussed further in Osman and Bolton 2006*b* and Lam and Bolton 2011, see also Beesley and Vardanega 2020). However, for simplicity, in this analysis, single b and γ_{50} values were selected.

Table 1. London Clay soil properties (depth y is measured in metres).

Parameter	Mean (μ)	Variation considered
Saturated unit weight, γ_{sat} (kN/m^3)*	20	19 (Low value) 21 (High value)
Undrained shear strength, s_u (kPa)†	$40.0 + 11.0\,y$ (eye-fit)	39.0 + 9.9 y (25th percentile) 24.8 + 13.9 y (Triaxial data only)
Nonlinearity exponent, b‡	0.58	0.47 ($\mu - 1\sigma$) 0.69 ($\mu + 1\sigma$)
Shear strain at 50% of s_u, γ_{50}‡	7.0×10^{-3}	5.1×10^{-3} ($\mu - 1\sigma$) 8.9×10^{-3} ($\mu + 1\sigma$)

*Based on a review of the datasets presented in Hight et al. (2003).
†Vardanega et al. (2012a).
‡Vardanega and Bolton (2011a).

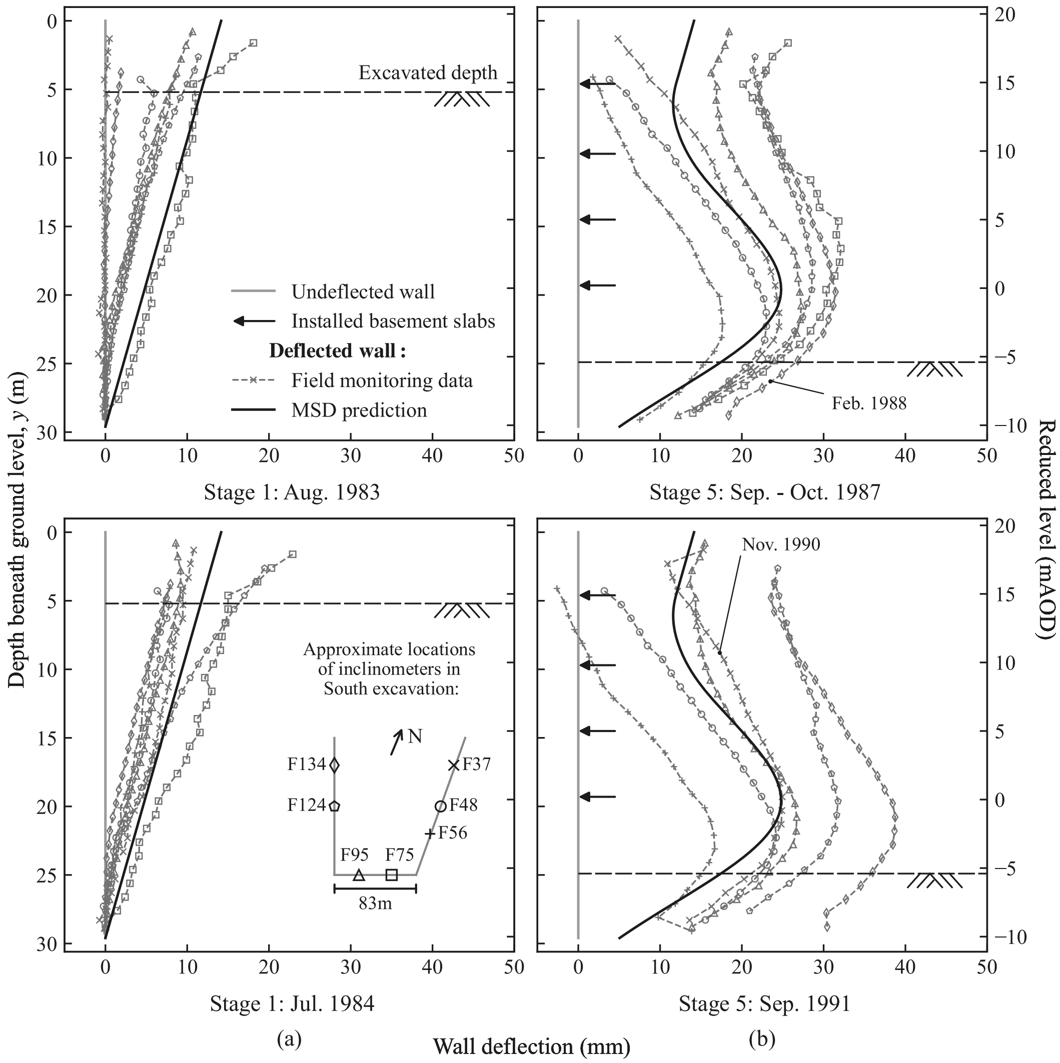
Application of MSD

Using the mean (μ) soil parameters from Table 1, the MSD approach (detailed in the online supplement) was applied to predict the wall displacements in the South excavation of the British Library Euston. The first stage of excavation (Stage 1) is assumed to result in the rotation of the wall only. In subsequent stages, a slab is first installed, then further excavation is carried out, which is assumed to result in cumulative bulging of the wall below the slab level. The predicted wall displacements are plotted in Fig. 1 against field monitoring data (where available) from inclinometers installed in the wall (Simpson and Vardanega 2014, Appendix W3). Anomalous data from the top of some inclinometers have been excluded in accordance with Simpson and Vardanega (2014).

Figure 1 shows reasonable predictions of the wall displacements, particularly for Stages 1 and 5. The variation in the intermediate stages is likely due to flexibility in the constructed slabs which are modelled in the MSD analysis as perfectly rigid lateral supports. This results in further wall rotation during these stages that is not accounted for in the MSD solution. In addition to this, piles installed within the excavation likely reduced the heave observed, which is not considered in this MSD analysis. The incremental maximum wall displacement calculated for the final stage (Stage 5) is negative (−0.6 mm). This is due to the method being inherently approximate and is considered to be negligible.

The field monitoring measurements for Stages 1 and 5 plotted in Fig. 1 were taken ~1 and 4 years, respectively, after the corresponding stage of excavation was completed. The effect of time on the wall displacements is shown in Fig. 2 for Stages 1 and 5 (where suitable data are available). Generally, over time, the maximum wall displacements increase, and the

Fig. 2. Effect of time after excavation on the field monitoring data (grey; data and location sketch from Simpson and Vardanega 2014, including Appendix W3) for (*a*) Stage 1 (excavation completed May 1983) and (*b*) Stage 5 (excavation completed August 1987). Soil parameters are given as mean values in Table 1 ($EI = 2192$ MNm^2/m, $\alpha_\lambda = 1.2$).



location of maximum bulging moves down the wall. This may also explain some of the variations between predicted and measured results in Stages 2, 3, and 4 observed in Fig. 1. For these stages, field monitoring data are only available ∼1–3 months after each excavation was completed.

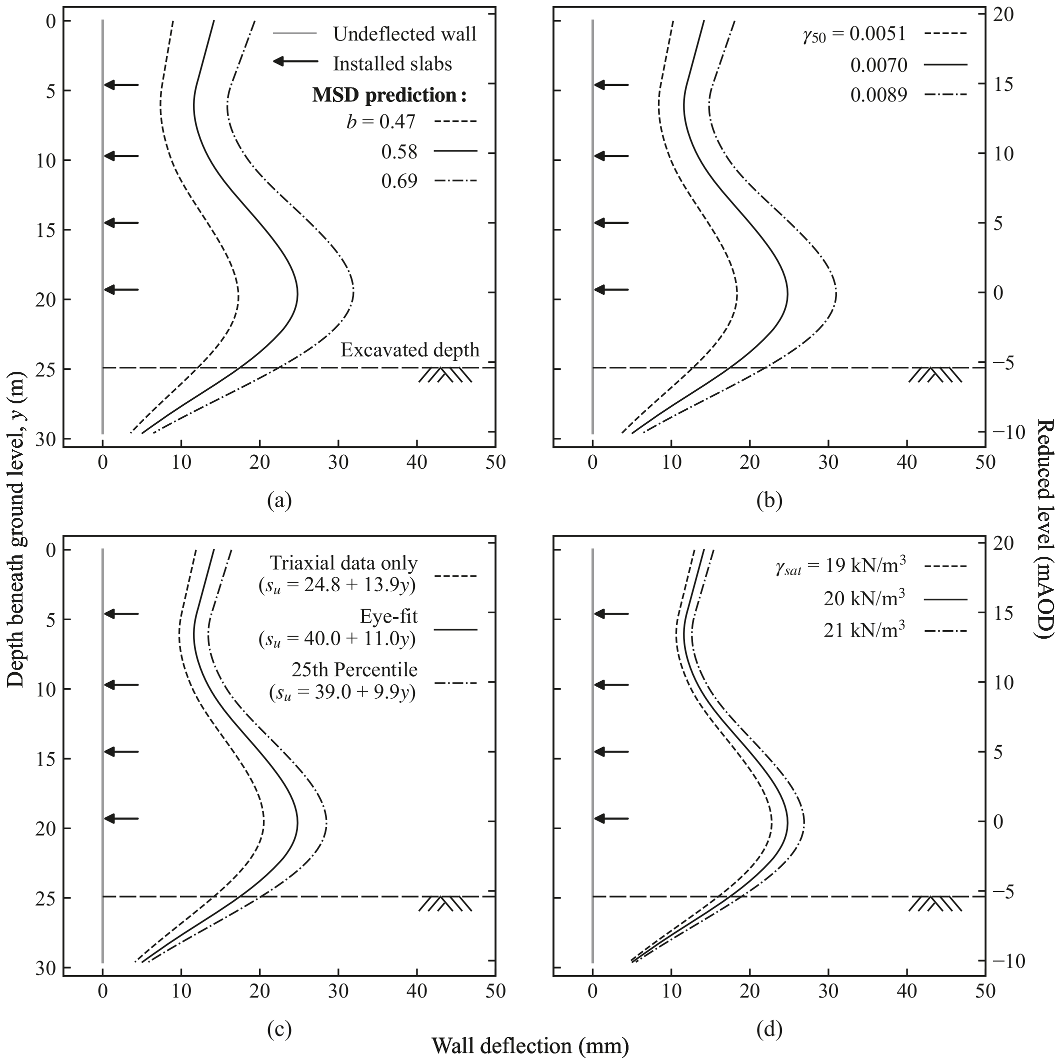
Sensitivity to parameter variability

The impact of varying b and γ_{50} by one standard deviation, σ, from the mean, μ, (obtained from the results presented in Vardanega and Bolton 2011*a*) on the predicted wall displacements is shown in Figs. 3*a* and 3*b*, respectively. The maximum wall displacements at Stage 5 can vary by up to 30% due to changing the b value $\pm 1\sigma$ and 26% due to changing the γ_{50} value $\pm 1\sigma$. This highlights the importance of having either site-specific triaxial test deformation data or a suitable database of similar test results in the same material (as used here) with which to fit the constitutive model (eq. 1). The mobilisation factor [$\beta_m = \tau / s_u$], calculated using eq. 1 with the average shear strain within the mechanism, is around 0.2. This means that the power-law soil model is near the limit of the recommended fitting range and, for smaller excavations or similar excavations in stiffer soil, an alternative simplified soil model (e.g., a modified hyperbolic model) may be more suitable.

The different s_u lines provided in Table 1 were fitted by Vardanega et al. (2012*a*) from site-specific test data in London Clay. The impact of the s_u lines employed on the predicted wall displacement in Stage 5 is shown in Fig. 3*c*. Similarly, the impact of the variations γ_{sat} for London Clay on the predicted wall displacements is shown in Fig. 3*d*. The impact of each of these two parameter choices is notably lower than that observed for the constitutive model parameters.

The secant pile wall construction is detailed in Ground Engineering (1984). The bending stiffness *EI* of this section can be calculated with various assumptions; the three considered in this work are illustrated in Fig. 4*a*. A steel Young's modulus of 210 GPa was used, alongside 31 GPa for the concrete (based on a target 90-day strength of 30 MPa, Ground Engi-

Fig. 3. The effect of the variation of soil parameters on the wall displacements predicted for Stage 5 from the mobilisable strength design (MSD) method. Soil parameters (unless otherwise noted) are given as mean values in Table 1 ($EI = 2192$ MNm^2/m, $\alpha_\lambda = 1.2$).



neering 1984). The different assumptions are shown in Fig. 4*b* to have minimal effect on the predicted wall displacements, likely due to this excavation being in a stiff clay.

The parameter α_λ, introduced by Osman and Bolton (2006*b*), is used in this method to describe the fixity at the base of the wall ($1 \leq \alpha_\lambda \leq 2$). This factor is applied to the size of the assumed deformation mechanism (illustrated in Fig. 5*a*, further details are provided in the online supplement). Selecting this parameter requires the designer to predict the location of maximum wall bending. If $\alpha_\lambda = 1$, the base of the wall is assumed to be fixed in a stiff stratum with zero lateral displacement. Alternatively, if $\alpha_\lambda = 2$, the base of the wall is assumed to be the location of maximum displacement, a reasonable assumption for walls embedded in soft soils (Clough and Reed 1984). Most walls installed in practice will have α_λ values between 1 and 2, but no guidance exists on how to select this value. Prior to construction, similar case studies can be used to estimate α_λ. Then, once the first bulging stage of excavation is completed, a preliminary α_λ value can be estimated from measured wall displacements. For this case study, a value of $\alpha_\lambda = 1.2$ (used in the above analysis) was empirically determined from the field test results in Stage 2. This value can be updated throughout construction; for this case study, $\alpha_\lambda = 1.4$ may be a better fit based on the observed behaviour after Stage 5. The authors acknowledge that more research into the value of α_λ is needed for the application of this method prior to construction. The effects of α_λ on the predicted wall displacement (Stage 5) are shown in Fig. 5*b*.

Conclusions

The results of the MSD analysis to predict wall displacements of the British library were compared with previously reported field monitoring data. To this end:

- The mean maximum inclinometer displacement 1 year after the completion of Stage 1 (where available) was 12.3 mm (with a range of 7.8–22.9 mm), compared to a predicted value of 14.2 mm (shown in Fig. 1). The mean maximum inclinometer displacement 4 years after the completion of

Fig. 4. (*a*) Calculation of the wall stiffness, *EI*, options (based on information from Ground Engineering 1984) and (*b*) the effect of the variation of *EI* on the wall displacements predicted for Stage 5 from the mobilisable strength design (MSD) method. Soil parameters are given as mean values in Table 1 ($\alpha_\lambda = 1.2$).

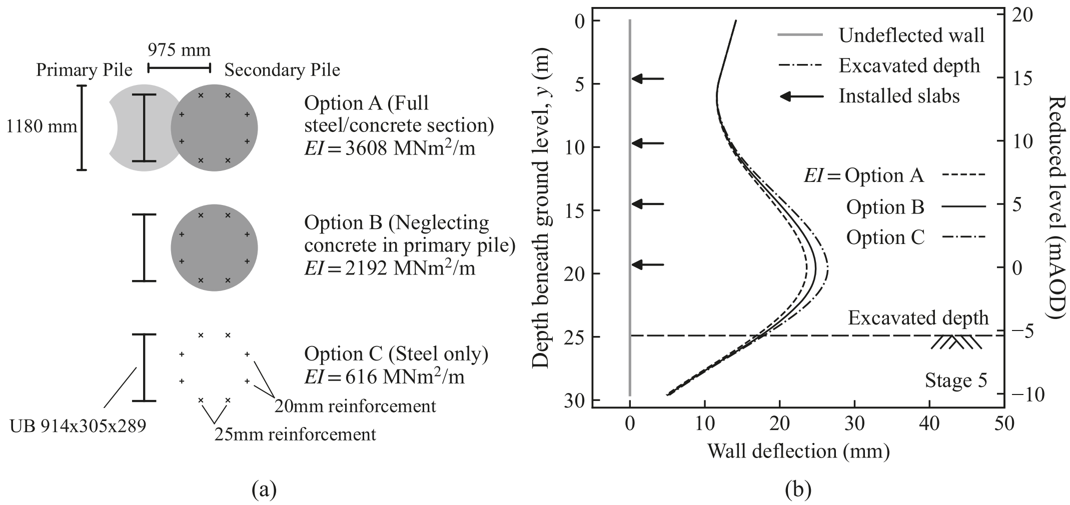
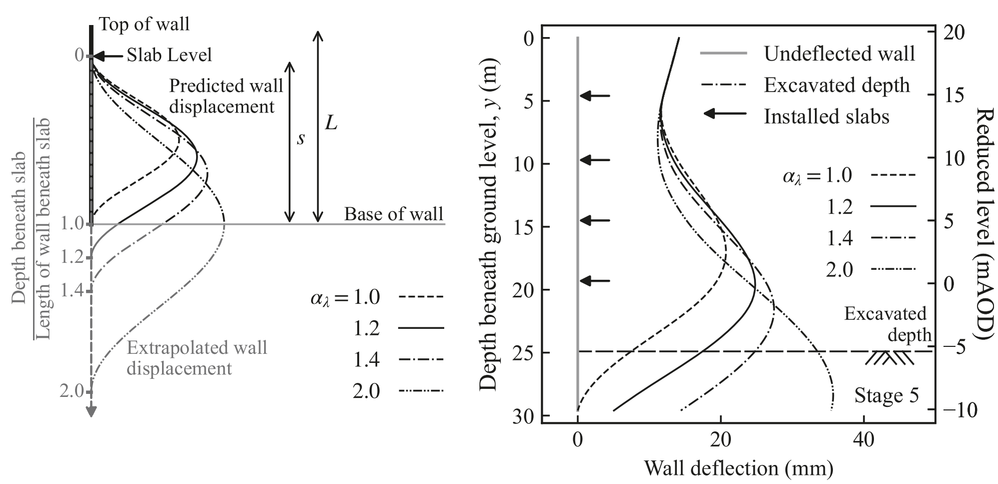


Fig. 5. (*a*) Illustration on the effect of α_λ on the predicted incremental wall displacements (during bulging) and (*b*) the effect of α_λ on the wall displacements predicted from the mobilisable strength design (MSD) method. Soil parameters are given as mean values in Table 1 ($EI = 2192$ MNm^2/m).



Stage 5 (where available) was 27.6 mm (with a range of 16.7–38.8 mm), compared to a predicted value of 24.8 mm. The mean depth to the maximum displacement measured at 21.3 m, compared to a predicted value of 19.7 m.

- The predictions for Stages 2–4 show larger maximum bulging (lower down the wall) than the field monitoring data, possibly due to the modelling assumption of perfectly rigid props and/or the short time after the excavation that field measurements were taken.
- The predicted results better match field monitoring results after they have been given time to deflect after excavation (shown in Fig. 2). Investigating how time effects could be incorporated into the MSD method (either directly or indirectly) could be an interesting avenue for future work.
- Realistic variation in the soil undrained shear strength profile, soil saturated unit weight, and the wall bending stiffness had minor effects in the predicted wall displacements. The soil constitutive model properties (b and γ_{50}) had a greater effect, highlighting the importance of site-specific soil deformation parameters.
- A preliminary α_λ value can be estimated from measured wall displacements in similar excavations or early stages of construction. However, more research into selecting this parameter is required to use this MSD method a priori.

List of symbols

FEA	finite element analysis
mAOD	metres above ordinance datum
MSD	mobilisable strength design
b	soil nonlinearity exponent
s_u	soil undrained shear strength
EI	plane-strain bending stiffness per metre length of the wall
y	depth below the top of the wall

α_λ	modification factor to scale the displacement mechanism for different wall base conditions
β_m	soil shear stress mobilisation factor (τ/s_u)
γ	soil engineering shear strain
γ_{50}	soil shear strain when 50% of the soil shear strength is mobilised
γ_{sat}	soil saturated unit weight
μ	mean value
σ	standard deviation
τ	soil engineering shear stress

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Data availability statement

As supplementary material to the main paper the authors also provide:

- a summary of the modified MSD method (and required equations) employed in this analysis
- a spreadsheet formulation of the analysis
- a python code of the analysis

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Visualisation: JJC, AHB
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Competing interests

The authors declare there are no competing interests.

Supplementary material

Supplementary data are available with the article at https://doi.org/10.1139/cgj-2023-0238.

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