Predicting tunnelling-induced ground movements and interpreting field measurements using numerical analysis: Crossrail case study at Hyde Park

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Abstract

Ground response to the construction of the Crossrail tunnels in London Clay beneath Hyde Park has been modelled numerically using advanced finite element analyses. The soil model used for modelling the London Clay was a kinematic hardening soil model (named M2-SKH). This model, when used for the St. James's Park greenfield site, provided excellent predictions of tunnelling-induced ground movements. Comparison of the results from the analysis of the Hyde Park greenfield site with associated field monitoring data, also suggests excellent predictions even though in this case the tunnels were: of larger diameter; deeper in the London Clay; and constructed with earth-pressure-balance machines. The influence of lining permeability was found to influence significantly short- and longer-term predictions. Interpretation of the predicted surface and subsurface vertical and horizontal displacements due to the construction of the Crossrail tunnels, exemplifies how numerical analysis can assist in explaining and identifying potential ambiguities in field measurements.

Keywords

Field instrumentation, ground movements, monitoring, numerical modelling, tunnels and tunnelling

1 Introduction

Predicting tunnelling-induced ground response using numerical analysis has been the focus of numerous publications in the past two decades and there have been clear progressive improvements in the quality of the predictions made. Avgerinos et al. (2016) discuss the salient points of a number of these papers before presenting results from their own recent numerical analyses, where a kinematic hardening model was used to model the St James's Park greenfield site ground response to open-face shield tunnelling. By comparing the results with field monitoring data from Nyren (1998) and Nyren et al. (2001), the analyses were shown to have been very successful in reproducing the ground response in both the short and long term by using different calibration parameters. A major difference with previous studies was that the full stress history of the ground was modelled, using input parameters based on advanced laboratory testing of appropriate London Clay samples. In view of a recent publication by Gasparre et al. (2014) that suggests that stiffness values reported historically for London Clay over-estimate true values, a focus of the study was to assess the effect of using a high and low range of stiffness values. In addition, this study revealed that the model cannot predict accurately results from both oedometer and undrained triaxial tests using the same parameter values. This translates in the tunnelling boundary value problem as:

- a. short-term soil displacements are more accurately predicted when the model is calibrated to predict the shearing behaviour of the soil and how the stiffness changes with strain
- b. long-term soil displacements are more accurately predicted when the model is calibrated to capture the consolidation (oedometric) behaviour of the soil.

In this paper predictions from numerical Finite Element analyses are presented in relation to the ground response at Hyde Park in London to the construction of twin Crossrail running tunnels. Comprehensive field monitoring was undertaken at the greenfield site as part of an extensive research project investigating the effect of tunnelling on existing tunnels (Standing et al., 2015). At the time the analyses were made, the volume loss (of the surface settlement trough) was known and used in the analyses and hence this aspect constitutes a class-C prediction (according to Lambe, 1973). However, at the same time, no information was available concerning the detailed ground displacements and hence the predictions in this respect were class-A.

Major differences between the two case studies are that the tunnels at Hyde Park are of greater diameter and depth to those at St James's Park and earth-pressure-balance tunnel boring machines (EPBM) were used rather than an open-face shield. As with the St James's Park study, the results from the numerical analyses are compared and validated with the field monitoring data.

In making the comparisons, ambiguities in some of the field monitoring data relating to both vertical and horizontal displacements were identified and investigated. It was found that the results from the numerical analyses were able to help unravel and explain these ambiguities.

2 Description of the numerical model

2.1 General site information and monitoring scheme details

The adopted soil profile for the finite element analysis is presented schematically in Figure 1 and is based on the deepest of the boreholes sunk to install instrumentation at the greenfield site, HP6 (Wan and Standing, 2014a). A 6m thick layer of superficial deposits (comprising made ground, alluvium and terrace gravels) overlies the B2 and A3 units of the London Clay (King, 1981) which have thicknesses of 30 m and 12.5 m, respectively. Unit A2 of the London Clay extends below A3 for another 11.4 m and is underlain by the Lambeth Group beds. The Lambeth Group has been divided into two layers representing the upper more clayey and less permeable units and the lower more granular and more permeable units.

A general layout of the area where the new Crossrail tunnels pass beneath the existing Central Line tunnels below Bayswater Road on the northern edge of Hyde Park is shown in Figure 2. In the layout the exact position of the installed instrumentation scheme is also presented. Three lines of surface monitoring point were installed and are denoted X-, Y- and Z-lines (results from the Z-line are not presented herein). A total of 38 boreholes were sunk to accommodate one or more instruments for subsurface monitoring. The positions of the installed multi-level rod-extensometers and the in-place electrolevel inclinometers, are shown in Figures 3 and 4 respectively. Extensometer and inclinometer boreholes were positioned in tandem so that vertical and horizontal displacements could be obtained at the same depths and locations (Wan and Standing, 2014b).

Vertical and horizontal surface displacements on the X- and Y-lines were determined by means of precise levelling and micrometer stick measurements, with estimated accuracies of ± 0.3 mm and ± 0.5 mm respectively. Concerning the subsurface measurements: each rod-extensometer borehole contained a maximum of eight anchors installed at specific depths (Wan and Standing, 2014b). The absolute accuracy achieved is dictated by that of the precise levelling (i.e. ± 0.3 mm) while the accuracy of measurements made with a dial gauge relative to the reference head is ± 0.05 mm. In each inclinometer borehole a series of micro-electro-mechanical system accelerometer sensors, which measured tilt, were positioned at selected intervals to complement the extensometer anchor depths. A maximum of 16 sensors were installed in any one inclinometer borehole with the independent error at each location being equivalent to 0.08mm for a sensor spacing of 4 m (Wan, 2014). Horizontal displacements are determined by integrating the rotations over the gauge length between successive sensors. The horizontal displacement profiles presented in this paper for the various inclinometer boreholes are related to the absolute surface horizontal movements determined from the micrometer stick measurements.

Multi-level vibrating wire piezometers with six sensors at different depths, a conventional standpipe piezometer and spade cells were also installed to measure pore water pressures and earth pressure changes in the ground.

The Crossrail tunnels were constructed using EPBMs from within which conventional bolted precast concrete segmental lining rings were erected of internal and external diameter $D_{in}=6.2$ m

and $D_{ex}=6.8$ m respectively (i.e. ring thickness of 30 cm). The annulus void between the erected tunnel lining extrados and the excavated ground was grouted via injection ports as the ring exited the shield.

2.2 Analyses performed and analysis sequence

Modelling the previous stress history of a site has been shown to be necessary for realistic predictions of tunnelling-induced ground displacements when using kinematic hardening models to simulate ground response (Avgerinos et al., 2016). As such, for the analyses presented here, the previous stress history of the Hyde Park site was simulated prior to modelling excavation of the Crossrail tunnels and included erosion of 180 m of overburden material, deposition of the superficial deposits and underdrainage of the site. The excavation of each of the two Crossrail tunnels followed with a 73-day period between them.

The construction sequence and the increments involved in each stage are summarized in Table 1. A parametric study was undertaken to establish whether and how the lining permeability of the Crossrail tunnels affect the predicted soil displacements. Three different cases were examined with the lining assumed to be either fully permeable or impermeable or with a finite permeability equal to 1% of the permeability of the soil adjacent to the tunnel axes.

2.3 Analyses details

2.3.1 General

The finite element mesh used in the analyses is presented in Figure 5. All the analyses were performed with plane-strain conditions. As the analyses involved coupled consolidation, the solid elements representing the London Clay and Upper Lambeth Group have four additional pore water pressure degrees of freedom at their corner nodes in addition to the 16 displacement degrees of freedom at each of their eight nodes. The beam elements (simulating the tunnel linings) have three degrees of freedom (two displacements and one rotation) at each of their three nodes. In all the analyses two-by-two integration was used and a modified Newton-Raphson technique was implemented as the non-linear solver (Potts & Zdravković, 1999).

2.3.2 Modelling of soil and tunnel linings

The superficial deposits deposit in the first 6m of the stratigraphy comprise a thin layer of Made Ground and a more substantial layer of River Terrace Deposits. These two layers were for simplicity modelled as a linear elastic perfectly plastic material having a Young's modulus $E=10000$ kPa and Poisson's ratio $\mu=0.3$ and incorporating a Mohr-Coulomb yield surface with $c'=0$ kPa, $\phi'=25^\circ$ and $v=12.5^\circ$. As the superficial deposits are far from the tunnels, v was in fact not mobilised in any of the runs. The behaviour of the Upper and Lower Lambeth Group deposits were simulated as non-linear elastic (model J4, based on Jardine et al., 1986) perfectly plastic (Mohr-Coulomb yield surface) with $c'=0$ kPa, $\phi'=28^\circ$ and 36° and $v=14^\circ$ and 18° , respectively. During the periods representing underdrainage, as part of the previous stress history, and

consolidation between the two tunnelling events, the angle of dilation ν of the Upper Lambeth Group was set to zero to prevent the Mohr-Coulomb model from predicting excessive dilation. The pre-yield model (J4) has been described by various researchers (e.g. Addenbroke et al., 1997). The parameters assumed for the Lambeth Group deposits are tabulated in Table 2. The small-strain model used for the Lambeth Group is a continuous degradation model. At times when an increase in the soil stiffness was anticipated due to a change in the stress path direction (e.g. before the deposition of the superficial deposits and prior to the WB tunnel excavation) this was accounted for by artificially zeroing the hardening parameters.

The London Clay units (B2, A3 and A2) were modelled with the modified kinematic surface model, M2-SKH implemented in ICFEP by Grammatikopoulou (2004) based on the “bubble” model for clay proposed by Al-Tabbaa & Wood (1989). Avgerinos et al. (2016) calibrated this model against the latest available experimental data for London Clay (Gasparre, 2005 and Hight et al., 2007) and proposed a set of parameters named ‘low triaxial’ that gave reasonable predictions for the short-term response of the ground due to tunnelling. The parameters are tabulated in Table 3 and reproduce satisfactorily undrained triaxial tests from Gasparre (2005) and Hight et al. (2007) in terms of stress paths, elastic stiffness and stiffness degradation while a softer response was observed for the unloading stages of oedometer tests. This model automatically accounts for changes in soil stiffness due to change in stress path direction. It should be noted that appropriate values were assigned for respective London Clay units, although the samples tested came from Heathrow, west of London.

Anisotropic permeability profiles, with values reducing with depth, were assumed as shown in Figure 6a. It was ensured that the permeability profile was consistent with the under-drained pore water pressure profile measured on site (see Figure 6b).

The Crossrail tunnel linings (conventional bolted precast concrete segments grade of C50/60 concrete with steel fibre dosage of 30-kg/m³) were modelled with linear elastic beam elements of unit weight $\gamma=30$ kN/m³, Young’s modulus $E=40000$ MPa, Poisson’s ratio $\nu=0.15$, cross-sectional area per unit width (i.e. lining thickness) $A'=0.3$ m²/m, second moment of area per unit width $I=2.25 \times 10^{-3}$ m⁴/m and a shear correction factor $k=0.8$.

2.3.3 Boundary conditions

During each analysis the lower boundary of the finite element mesh was fixed in both the vertical and horizontal directions while the vertical boundaries were fixed only in the horizontal direction. Excavation of the tunnels was simulated using the volume loss control method (Potts and Zdravković, 2001). The linings were installed as soon as the desired volume losses (at the boundary of the tunnels) were achieved: $V_L = 0.78\%$ and 1.39% for the Westbound (WB) and Eastbound (EB) tunnels respectively. These volume losses were determined from field measurements along the southern limb of the X-line (see Figure 2) and so represent the volume of the surface settlement trough.

The hydraulic boundary conditions were set so that at the top of the London Clay and at the side boundaries of the mesh the pore water pressures remained unchanged. At the interface between the Lower and the Upper Lambeth Group a 19.62 kPa pore water pressure was prescribed at the end of the previous stress history in order to achieve the desired under-drained pore water pressure profile (see Figure 6b), and was maintained at this value for the remainder of the analysis.

Three hydraulic boundary conditions were used for the tunnels.

- (i) When they were assumed to be fully permeable, a precipitation boundary condition was applied (enabling a zero pore water pressure or a zero flow boundary condition to be applied at individual nodes around the tunnel periphery depending on whether the analysis indicated flow out of or into the surrounding soil at these nodes, respectively; the software automatically accounted for any change in this boundary condition as the analysis progressed).
- (ii) When they were assumed to be completely impermeable, a no-flow boundary condition was applied.
- (iii) When tunnel lining was modelled as having a finite permeability equal to 1% of that of the adjacent soil, the precipitation boundary was applied (as above) and the desired tunnel lining permeability was assigned to the first ring of soil elements around the tunnel perimeters (a technique originally proposed by Shin et al., 2002).

In tunnelling practice, Haack classes are often used to quantify the lining permeability (Haack, 1991). It is judged that the lining permeability in condition (iii) above roughly relates to Haack class 1 and condition (ii) to Haack class 2.

3 Ground conditions prior to WB tunnel excavation

Grammatikopoulou et al. (2008) presented and discussed the influence of previous stress history on the ground surface settlement trough induced by tunnelling. Based on this work, and the results of simulating the St. James's Park case study with the M2-SKH model, Avgerinos et al. (2016) concluded that the K_0 and the yield stress ratio (YSR) profiles prior to analysing the effects of tunnel construction have to be consistent with the values of other parameters used in the model. The YSR and K_0 profiles resulting from the modelling of the previous stress history of the Hyde Park site are shown in Figure 7. The YSR and K_0 profiles prior to underdrainage differ from those presented by Avgerinos et al. (2016) for the St. James's Park case study despite the fact that the London Clay was modelled with exactly the same material model and properties. The differences at the Hyde Park site are attributed to (i) there being less of the superficial deposits laid down on top of the London Clay after the initial erosion of the overburden and (ii) to the greater thickness of the B2 unit of London Clay. This result further emphasises the fact that the distribution of K_0 with depth is not unique and is very sensitive to stress history (Burland et al., 1979).

It is well known that at many sites across London an under-drained pore water pressure profile is encountered due to historic and current pumping from the deep aquifer. The simulation of the underdrainage for this site was modelled over 70 years and the pore water pressure profile prior to any tunnel excavation is in agreement with the available field measurements (Figure 6b). The underdrainage effect on the YSR and K_0 profiles is also presented in Figure 7. The YSR was mainly affected towards the base of the London Clay where the pore water pressure profile has the greatest deviation from hydrostatic. The K_0 profile can be seen to be affected throughout most of the soil depth. The decrease in the pore water pressures resulted in a proportionate increase in vertical effective stress (σ_v') with depth. The horizontal effective stresses (σ_h') remained practically unchanged and this led to the K_0 values decreasing during underdrainage. The final K_0 values predicted for the Hyde Park site, prior to tunnel construction, vary from 1 to 0.8 generally decreasing with depth. Over the depth of the tunnels K_0 varies from 0.9 to 0.95. The kink in the K_0 profile at the Upper Lambeth Group horizon (which is a result of the constitutive models used for Lambeth Group) is not believed to play an important role in the predicted tunnelling-induced soil displacements as it is well below the invert of the Crossrail tunnels (see Figure 1).

4 Predictions after WB Crossrail tunnel excavation

4.1 Surface displacements

Table 4 summarises the volume loss (of the surface settlement trough) and the trough width parameter (K_y) values obtained from the field measurements (on the Y- and X-line surface monitoring points, denoted as YSMPs, XSPMs respectively in the table). The K_y values were determined by plotting the logarithm of normalised vertical displacements ($\ln(s/s_{max})$) against the square of the transverse offset distances (y^2) and calculating the slope of the line (Wan et al., 2017). Two points to note are that (i) the northern limb of the surface troughs for the Y-line is slightly wider than its southern limb and (ii) the volume losses determined for the X- and Y-lines differ quite substantially. The first point might result from the presence of the Central Line tunnels: Wan et al. (2017) suggest that they inhibited the development of Crossrail tunnelling-induced ground movements in their close vicinity. The second point is also thought to be related to the presence of the Central line tunnels but also to the fact that the bases of boreholes HP24 and HP21 were assumed to be fixed when calculating the vertical displacements along the Y- and X-lines respectively.

In performing the numerical analysis of the WB Crossrail tunnel using the volume loss control method, the target volume loss was set as that relating to the southern limb of the X-line (see Figure 2) as this is judged to be more representative of the 'greenfield' situation, being located further away from the Central Line tunnels. Even though the excavation of the WB tunnel was simulated using 100 increments, the final volume loss produced from the numerical analysis was slightly higher than the desired value (0.8%). Volume loss values relating to the numerical analyses were determined using the two approaches given below.

- a. Integration of the normal displacements of the soil around the perimeter of the excavated tunnel ($V_L = 0.93\%$). The desired value was 0.8% and this could only be achieved if many increments were used to model tunnel excavation. As the volume loss at the tunnel periphery and the one obtained from ground surface trough are different (see below) any refinement of that value was considered unnecessary.
- b. Integration of the best fit Gaussian curve to the part of the surface settlement trough that corresponds to the width of the X-line (-40 m to +5.8 m from the tunnel axis). Depending on whether the surface settlements were calculated in relation to the displacement of the node of the finite element mesh coinciding with the base of HP24 at 40 m depth or to a horizontal line at 50 m depth, values of $V_L = 0.55\%$ and 0.86% were obtained.

The surface settlement troughs and the normalised surface settlement troughs from the field measurements are compared with those from the numerical analysis in Figure 8. The surface settlement troughs obtained from the numerical analysis are based on the following datum positions (as discussed above with reference to volume loss values).

- a. A datum corresponding to the node of the finite element mesh coinciding with the base of HP24 (as was done for the field measurements for the Y-line).
- b. Datum positions along a horizontal line at 50 m depth: for each point on the surface trough the vertical displacement was corrected by the vertical displacement of the corresponding point located 50m below ground level at the same transverse offset distance from the WB tunnel axis. This correction is intended to replicate the correction applied to the X-line (base of HP21 which in the field did not move at all since it was on a section far behind the X-line). The vertical displacements predicted by the numerical analysis along the line 50 m below ground level varied, but by less than 1 mm (heave). Since some movements were predicted along this line in the numerical analysis, it was considered appropriate to apply a correction to take this into account so as to improve the compatibility between the predictions and the measurements.

In comparing the results from the numerical analysis with the field measurements, an overall satisfactory agreement is achieved when applying datum 'a' to the Y-line data and datum 'b' to the X-line data in terms of both the surface settlement magnitudes and the width of the surface trough. When the numerical results are expressed relative to datum 'a', heave is predicted at a distance of around 40 m from the tunnel axis and towards the vertical edges of the mesh (always less than 1mm). Heave has not been plotted in Figure 8 as it is not considered to be realistic. Thus the finite element predictions indicate that the base of HP24 is influenced by the WB tunnel. This implies that if this analysis was used for making Class- A predictions, the bottom of HP24 should not have been proposed as a suitable datum.

The marked difference in volume loss values (between the two lines) might be partly attributed to this but is much more likely to result from the presence of the Central Line tunnels shielding the ground in the vicinity of the Y-line during the WB tunnel construction, as discussed by Wan et al. (2017). However, the presence of the Central Line tunnels have not been simulated in this model and so their influence cannot be assessed in the numerical analysis described in this paper. Despite this, since the results for the surface trough when applying correction 'a' agree well with

field measurements, confidence is gained in using the same correction for subsurface vertical displacements which were measured along the Y-line.

In practice, when making class-A predictions using a numerical approach similar to that described in this paper, estimates of volume loss would have to be made. These would be based on past experience and well documented case studies: e.g. values between 0.5% and 1.0% might be appropriate for tunnelling with an EPBM.

A comparison of horizontal surface displacements from the WB tunnel excavation is shown in Figure 9. In addition to the field measurements (using a micrometer stick and assuming zero horizontal displacement above the tunnel axis) and the numerical analysis predictions, indirectly calculated horizontal displacement profiles are shown. These latter ground curves (from Wan, 2014) are based on fitting a Gaussian curve to the field settlement data and using this in conjunction with a point-sink assumption where the resultant vectors of displacement at any point along the surface X- and Y-lines are taken to be directed towards the tunnel axis (O'Reilly & New, 1982). They are therefore referred to as 'indirect field measurements' in Figure 9. The direct field measurements (with the micrometer stick) reveal that the surface horizontal displacements are not symmetrical but have slightly greater displacements towards the northern side. This can be attributed to the influence of the presence of the Central Line tunnels on the Crossrail tunnels. This asymmetry is not predicted by the numerical analysis since the Central Line tunnels were not modelled in the analyses. However, for the southern limb (which should be closer to a greenfield condition) the predicted horizontal displacements compare well with the field measurements in terms of both magnitude and shape. Both the field measurements and the numerical analysis predict wider surface horizontal displacement profiles than that determined using the indirect approach involving the point-sink assumption.

4.2 Subsurface displacements

Subsurface vertical displacements measured at the extensometer borehole locations along the Y-line are shown in Figure 10 and are compared with the numerical results expressed relative to the base of HP24. The predicted movements for borehole locations further than 10 m from the WB tunnel axis (i.e. about two tunnel diameters away) coincide with the field measurements, predicting the magnitude and the shape of the vertical displacement profile accurately. The comparison deteriorates for borehole locations closer to the tunnel axis with the numerical analysis under-predicting the magnitude of vertical displacements from about 10m depth below ground level to tunnel crown level. However, at shallower depths and below the invert of the tunnel (borehole location HP18, HP19, HP21 and HP22), the predictions agree well with the field measurements. In all cases the shape is also predicted well, with vertical displacements increasing with depth above the tunnel crown level and then decreasing for greater depths.

Similar conclusions can be drawn for the subsurface settlement troughs at different depths as shown in Figure 11. The shapes of the predicted troughs are in reasonably good agreement with the field measurements but the magnitudes are under-estimated in the close vicinity of the tunnel

(again within a region of about two tunnel diameters). The subsurface trough width parameter, K_{yz} , is plotted against depth both for the field measurements and the numerical analysis in Figure 12 (again K_{yz} was calculated by plotting the logarithm of normalised vertical displacements ($\ln(s/s_{max})$) against square of the transverse offset distances (y^2) and calculating the slope of the line for $\ln(s/s_{max}) > -1.5$). The numerical analysis satisfactorily predicts the trough width parameters, which increase with depth, although the magnitudes are marginally larger than those determined from the field measurements and empirical correlations.

Subsurface horizontal displacements measured in the field and predicted from the numerical analysis are compared in Figure 13 for the electrolevel inclinometer boreholes along the Y-line and in the close vicinity of the WB tunnel axis (see Figure 4 – note that results for HP5 are not shown as displacements directly above the axis of the tunnel are zero because of symmetry). The numerical predictions are in good agreement with the field measurements for all the inclinometer borehole locations, irrespective of their distance from the tunnel axis in terms of both the shape and the actual magnitude of the horizontal displacements. Individual profiles and any associated discrepancies are discussed below. Unless stated otherwise, the field monitoring data relate to when the EPBM face was 40 to 50 m beyond the Y-line.

The horizontal displacements from the field measurements for HP3 have been processed using two approaches: either by integrating the rotations over gauge lengths spanning successive sensors or by integrating a polynomial fit to the measured values of rotation. The two resulting profiles of horizontal displacements are shown in Figure 13b. It can be observed that the two interpretations provide broadly similar results although as the magnitudes of displacement are very small (they are within a range of 4 mm) the percentage error between the two profiles is potentially large. The deviation between the profiles generated using the two approaches is dependent on the gauge lengths between sensors and the nature of the displacement profile. In both approaches, the closer the spacing between successive sensors, the more representative should be the resulting profiles which should converge towards each other with reducing spacing. For this reason the spacing between successive sensors is recommended to be less than 2 m in the latest field monitoring guide (BSI, 2016). At this particular HP3 borehole location, the horizontal displacement profile from the polynomial approach is closer to that from the numerical prediction which is also based on closely spaced nodal points. Another potential cause for differences between the field measurements and the numerical predictions is that the former are made within inclinometer tubes grouted into the borehole. If the stiffness of the grouted inclinometer tube is greater than that of the ground, this could lead to under-estimates of true ground response (and vice versa).

Inclinometer boreholes HP4 and HP6 are located at the same offset of 4.7 m to the south and north of the WB tunnel axis respectively. Figures 13c and d present the subsurface horizontal displacements from both field observations and the numerical analysis. The field measurement profiles, both of which relate to when the EPBM face was 40.3 m beyond the Y-line, are not symmetrical. The magnitude of maximum displacements is greater to the north (being about 18mm at HP6 compared with about 10mm at HP4) and could be attributed to previous ground

disturbance from the construction of Central Line tunnels. In order to assess whether other small variations in time or borehole position might cause the asymmetry, additional profiles have been plotted in Figures 13c and d. As mentioned above, identifying the time or boundary when the short-term soil movements are complete and when the consolidation starts is not straightforward. The two profiles shown in Figure 13d relate to when the EPBM face was 40.3 m and 175m beyond the Y-line. These distances correspond to 42 hours and 9 days after the EPBM face was directly below the Y-line. The effect of consolidation increasing the horizontal displacements is evident.

In order to assess the sensitivity of horizontal ground displacements to spatial location, two additional profiles, determined from the results of the numerical analysis, relating to virtual borehole positions at offsets of 3.65 m and 6.0 m from the WB tunnel axis are plotted in Figure 13c (along with the HP4 position at a 4.75 m offset). It is clear that the maximum subsurface horizontal displacements vary markedly with horizontal offset distance along the monitoring line from the tunnel axis. A major implication of this is that any non-verticality of the inclinometer boreholes could significantly affect the reliability of the field measurements. The offset values of 3.65 m and 6 m can be related to horizontal ground displacements at tunnel axis level for the case when borehole HP4 deviates from the vertical by 1.8° and 2.1° respectively. As a consequence of such potential deviations in borehole HP4, the horizontal displacements measured at tunnel axis level would be underestimated by 27% (6.00 m offset) or overestimated by 44% (3.65 m offset). This example emphasises the importance of the verticality of inclinometer tubes and boreholes since comparatively small deviations can lead to significant differences in measurement from those actually taking place at the intended spatial position. For this reason the latest monitoring guide (BSI, 2016) also recommends that inclinometer tubes should be within 2° of vertical. Although not investigated here, another potential source of error could be twisting of the keyways within the inclinometer tube. The discussion above offers some potential explanations for discrepancies between the field measurements and the results from the numerical analysis.

5 Predictions prior to EB Crossrail tunnel excavation

The excavation and construction of the WB tunnel was followed by a 73-day consolidation period before the EB tunnel was excavated. The Crossrail tunnels are designed to very high standards of water-proofing and so should be essentially impermeable. The analyses performed included three different assumptions for the lining permeability: a) permeable, b) impermeable and c) permeable with a very small permeability (1% of the permeability of the adjacent soil).

Cumulative and normalised ground surface settlements just prior to construction of the EB tunnel are presented in Figure 14. Compared with the measurements on both the X- and Y-lines, the numerical analysis performed assuming the tunnel lining to be completely permeable results in significant over-predictions of settlement (Figure 14b). A better prediction of the field measurements was obtained for the case where the tunnel lining was assigned a small finite permeability rather than being fully impermeable (Figure 14a). It does not seem unreasonable

that some water ingress may have occurred or that water is flowing longitudinally along the extrados of the tunnel. The shape of the settlement trough is well predicted (Figure 14c).

The predictions of cumulative subsurface soil vertical displacements from analyses with an impermeable lining and a lining with a permeability of 1% of that of the adjacent soil (i.e. $k_{\text{lining}} = 1\%k_{\text{ground}}$) are compared with field measurements in Figure 15. The numerical analysis results in both Figures 14 and 15 are expressed in relation to a datum taken to be the base of HP24. However, the numerical analyses indicate that the base of HP24 at a depth of 40 m was subject to displacements due to the WB tunnel excavation and subsequent consolidation (see Figure 15j). If the surface settlements were expressed in relation to a horizontal line at 50 m depth, the numerical predictions result in deeper troughs that are closer to the data from the X-line (presented with respect to the base of HP21) as shown in Figure 16. This further illustrates the importance of the choosing an appropriate datum when comparing predictions with field data.

6 Predictions after EB Crossrail tunnel excavation

6.1 Surface displacements

Volume loss and trough width parameter values relating to the field monitoring lines for the EB Crossrail tunnel excavation are summarised in Table 5. The volume loss and trough width parameter values obtained from the numerical analysis (for the EB Crossrail tunnel) are presented in Table 6. The target volume loss for this excavation, 1.3%-1.4%, was obtained from the southern part of the troughs of the field measurements, in order to account for the presence of the WB tunnel.

Surface settlement and normalised settlement troughs due to the excavation of the EB tunnel are presented in Figure 17. The southern limb of the trough (i.e. that closest to the already excavated WB tunnel) has a greater volume loss and a wider trough than the northern limb. Although the northern limb would be expected to be influenced by the presence of the existing Central Line tunnels, the more recent excavation of the WB tunnel has a greater effect. Wan (2014) suggests that the surface settlements of the six northernmost points on the XSMP line (furthest away from the EB tunnel axis) did not respond in a truly greenfield fashion as they were affected by the North Carriage Drive (see Figure 2). This is evident from the field data plotted in Figure 17a.

The numerical predictions under-estimate the surface settlement irrespective of the lining permeability assumed for the WB tunnel. The numerical predictions presented are expressed relative to displacements of the line at 50 m below ground level (were a maximum heave of around 1.5 mm was predicted in all three analyses) while the field measurements are relative either to the base of HP18 (YSMP line) or to the base of HP21 (XSMP line).

The normalised shapes of the predicted settlement troughs are compared with the measured field data for the various WB tunnel lining permeability scenarios in Figures 17b, c and d. The northern limb predictions are in excellent agreement with the field measurements up to the North Carriage Drive (although there are only four SMPs extending to about 12 m from the EB tunnel axis – see

Figure 2) irrespective of the assumed WB tunnel lining permeability. The remaining points are judged not to be truly greenfield, as noted above. The shape of the southern part of the settlement trough was only well predicted for the case of fully permeable Westbound tunnel lining. In fact, only for this case was the southern limb of the trough wider than the northern limb, similar to the field measurements. This might be attributed to the fully permeable WB tunnel lining case resulting in more accurate predictions of the pore water pressure changes at the end of the short-term consolidation period prior to the EB tunnel excavation as shown in Figure 18. The implication of this is that the changes in pore water pressure prior to the EB Crossrail EPBM arrival control the soil displacements during EB tunnel excavation. Closer predictions of pore water pressure changes, imply better predicted stress-paths and therefore more representative soil stiffness response and consequent displacements.

Horizontal ground surface displacements from excavation of the EB tunnel are presented in Figure 19. The direct field measurements and indirect profiles are plotted, the latter being based on a Gaussian fit to the settlement data in conjunction with a point-sink assumption, using the same approach and assumptions as described for the WB tunnel case. For the southern limb of the X-line, zero horizontal displacements were assumed for the southernmost point of the line because the presence of the North Ride precluded continuous micrometer stick measurements in this area directly above the EB tunnel where displacements would be expected to be zero (according to the point-sink assumption). The field data are well reproduced when the existing WB tunnel lining was simulated as fully impermeable or with its permeability being 1% of that of the adjacent ground. The magnitudes of the predicted horizontal displacements lie between the values measured along the two monitoring X- and Y-lines with the maximum located at the same horizontal distance from the EB tunnel axis as for the monitoring data. Neither the field data for the YSMP line nor the corresponding numerical predictions follow the predicted horizontal displacements based on the indirect approach at offsets greater than 30m from the EB axis.

6.2 Subsurface displacements

The predictions of the subsurface vertical displacements from the three lining permeability scenarios implemented in the numerical analyses are compared with the field measurements in Figure 20. Excellent agreement is achieved for the case where the WB tunnel lining was modelled to be fully permeable when predicting the vertical subsurface movements. This again indicates that the pore water pressure changes due to the WB tunnel excavation and subsequent consolidation period were more realistically predicted when the WB tunnel lining was modelled as fully permeable which in turn affected the predictions of the ground response to the EB tunnel excavation. The other permeability scenarios also produced reasonable predictions.

Subsurface horizontal displacement measurements due to the EB tunnel excavation are shown in Figure 21. The shape and the magnitude of the numerical predictions are generally in good agreement with the field data. Field measurements from inclinometers HP9 and HP15, both located 5.2 m north of the EB axis but at different distances from the existing Central Line tunnels (see Figure 2) are compared in Figure 21f. The maximum horizontal displacement measured in HP9 is almost double that of HP15. This can be attributed to the fact that HP9 is closer to the

Central Line tunnels and the difference between the two profiles reflects the influence of the existing tunnels on the soil response due to the new Crossrail excavation. An alternative explanation for the discrepancies between measurements at HP9 and HP15 could be the verticality of the inclinometer boreholes as discussed previously.

7 Conclusions

A series of numerical analysis has been performed to model the construction of the Crossrail tunnels in London Clay beneath Hyde Park. A numerical analysis used a kinematic hardening soil model (named M2-SKH) was adopted to model the behaviour of the London Clay since this model when used for the St. James's Park greenfield site by Avgerinos et al. (2016) provided excellent predictions of tunnelling-induced ground movements. Results from the numerical analysis have been compared comprehensively with the field monitoring data provided by Wan (2014) and Wan et al. (2017). Predictions from the numerical analysis compare in most cases satisfactorily with the field monitoring data. The excellent predictions presented here and by Avgerinos et al. (2016) for the St. James's Park case study highlight that a consistent use of the M2-SKH model can predict satisfactorily ground tunnel induced movements, irrespective of dimensions and depth of the tunnels and the tunnelling technique adopted (e.g. open-face shield versus EPBM).

Interpretation of the predicted surface and subsurface vertical and horizontal displacements due to the construction of the Crossrail tunnels, exemplifies how numerical analysis can assist in explaining and identifying potential ambiguities in field measurements. In particular, the importance of selecting a truly stable point for anchoring the vertical displacements of the ground has been demonstrated. This is important when results from numerical analysis are compared with field monitoring data. The former indicates that the datum points selected for the field monitoring were not always stable with heave predicted below the invert of the tunnel.

In examining the distribution of horizontal ground surface displacements, it was observed that they extend much further from the tunnel than would be predicted based on the empirical approach of a 'Gaussian' fit in conjunction with a point sink assumption.

When consolidation stages are modelled the predictions of the numerical analysis depend not only on the calibration of the bubble model (as shown by Avgerinos et al., 2016) but also on the lining permeability itself.

With respect to horizontal displacements, the numerical analysis highlights that small potential deviations in the verticality of inclinometers might result in poor comparisons between the field monitoring data and numerical analysis predictions.

Finally, in interpreting field monitoring data, identifying the boundary between short- and longer-term movements is not straightforward. This is not the case for 2D FE analysis and thus it might be another potential reason for discrepancies.

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Table 1: Analysis sequence

Increment numbers	Analysis stages
0	Deactivation of the elements representing the linings of the two tunnels and the superficial deposits
1-19	Erosion of 180m of overburden
20	Deposition of superficial deposits and rise of the water table from the top of London Clay to 3m depth within the superficial deposits
21-30	Underdrainage of the pore water pressure profile
31-130	Excavation/lining construction of Westbound Crossrail tunnel
131-140	Consolidation period of 73 days prior to arrival of the Eastbound Crossrail tunnel
141-240	Excavation/lining construction of Eastbound Crossrail tunnel

Table 2: Parameters assumed for the J4 model for Upper and Lower Lambeth Group

Model parameters	C ₁	C ₂	C ₃ (%)	α	γ	E _{dmin} (%)	E _{dmax} (%)	G _{min}
Soil layers								
Upper Lambeth Group	987	875	0.0001	1.1	0.850	0.003	0.3	2000
Lower Lambeth Group	1200	1100	0.0001	1.30	0.62	0.002	0.3	2000
	C ₄	C ₅	C ₆ (%)	δ	λ	ϵ_{vmin} (%)	ϵ_{vmax} (%)	K _{min}
Upper Lambeth Group	404	404	0.0035	1.8	0.34	0.001	0.2	2500
Lower Lambeth Group	265	850	0.0004	1.20	0.34	0.003	0.4	2500

Table 3: M2-SKH model parameters assumed for London Clay units

Model parameters	unit B2/unit A3,A2
v ₁	2.5/2.75
λ	0.12/0.15
κ	0.06/0.063
n	0.87
m	0.28
A ₂	250/180
ϕ'	25°/20.3°
R ₂	0.01/0.005
a	0.8

Table 4: Volume loss and K values for the WB Crossrail tunnel from field measurements
(Wan et al., 2017)

WB Crossrail tunnel volume loss / K_v values		
	Southern half-trough	Northern half-trough
YSMPs	0.40% / 0.34	0.48% / 0.40
XSMPs	0.78% / 0.44	0.78% / 0.44

*reference head of the extensometers

Table 5: Volume loss and K values for the EB Crossrail tunnel from the field measurements
(Wan et al., 2017)

EB Crossrail tunnel volume loss / K		
	Southern half-trough	Northern half-trough
YSMPs	0.97% / 0.43	0.70% / 0.31
XSMPs	1.39% / 0.47	1.01% / 0.34

Table 6: Volume loss and K values for the EB Crossrail tunnel from the numerical analysis

EB Crossrail tunnel volume loss / K				
Lining permeability	<i>Unloading (%)</i>	<i>Around the tunnel perimeter</i>	<i>Southern part of the mesh</i>	<i>Northern part of the mesh</i>
Permeable	9	1.40%	1.26% / 0.49	1.21% / 0.44
Permeable - 1% of the soil permeability	11	1.41%	1.11% / 0.50	1.24% / 0.53
Impermeable	12	1.32%	1.07% / 0.51	1.18% / 0.54

Figure 1: Soil profile and tunnel geometry assumed at Hyde Park.

Figure 2: Instrumentation plan at Hyde Park site (Wan et al., 2017).

Figure 3: Cross-section through line Y (looking Northwest, see Figure 2) with the position of rod extensometers boreholes (after Wan & Standing, 2014b).

Figure 4: Cross-section (looking Northwest, see Figure 2) with the in-place inclinometer boreholes (after Wan & Standing, 2014b).

Figure 5: Finite element mesh.

Figure 6: a) Permeability profile and b) pore water pressure profile assumed in the finite element analysis.

Figure 7: Effect of previous stress history on a) p_0'/p' and b) K_0 Hyde Park profiles.

Figure 8: a) Ground surface settlement and b) normalised ground surface settlement due to the Westbound Crossrail tunnel.

Figure 9: Horizontal displacements due to Westbound Crossrail tunnel excavation.

Figure 10: Vertical displacement at various rod extensometer positions due to the westbound Crossrail tunnel excavation.

Figure 11: Subsurface settlement trough at different depths due to the Westbound Crossrail tunnel excavation.

Figure 12: Subsurface trough width parameters due to the Westbound Crossrail tunnel.

(a) HP2 (15.8m south of WB axis)

(b) HP3 (11.0m south of WB axis)

(c) HP4 (4.7m south of WB axis)

(d) HP6 (4.7m north of WB axis)

(e) HP7 (7.9m north of WB axis)

Figure 13: Horizontal displacements for different inclinometer borehole locations due to the Westbound Crossrail tunnel excavation.

Figure 14: a),b) Ground surface settlement and c) normalised surface settlement prior the Eastbound Crossrail tunnel construction for various tunnel lining permeability (relative to the base of HP24).

Figure 15: Subsurface settlement troughs prior to the Eastbound tunnel construction for various permeability scenarios for the Westbound tunnel.

Figure 16: a) Ground surface settlement and b) normalised surface settlement prior the Eastbound Crossrail tunnel construction (corrected with the line 50m below ground level).

Figure 17: a) Ground surface settlement and b), c) and d) normalised ground surface settlement due to the Eastbound Crossrail tunnel (Westbound tunnel assumed b) permeable, c) permeable with 1% of the soil's permeability and d) impermeable).

Figure 18: Pore water pressure changes during the consolidation period between the two Crossrail tunnels constructions at piezometer HP32 (12m North of Westbound tunnel axis).

Figure 19: Ground surface horizontal displacements due to the Eastbound Crossrail tunnel excavation.

Figure 20: Vertical displacement at rod extensometers due to the Eastbound Crossrail tunnel excavation.

Figure 21: Subsurface horizontal displacements due to Eastbound Crossrail tunnel excavation (various inclinometers).