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A ground movement assessment case study: validating the construction sequence to protect listed buildings during the construction phase of the Olympia development

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ABSTRACT: To provide support to the retained façade and the adjacent listed structures within the high-density Olympia development, the new basements of Central Hall and G-Gate were constructed using the top-down method. A detailed ground movement assessment was performed to verify the chosen construction sequence and confirm stability of the sensitive structures. 2D finite element sections modelled the past and future loadings of the development. These were extrapolated to 3D using Oasys XDisp software to provide settlement predictions and damage assessments for the surrounding buildings.

The soil model options were limited by the available ground investigation results and chosen based on the most realistic soil movement predictions following a parametric study. The ongoing monitoring is providing further validation of the analysis results, chosen method and soil properties. The predicted heave for Central Hall due to demolition, basement excavation and new deep foundations is compared to the predicted settlement for a basement on the neighbouring but previously undeveloped site. This case study illustrates the importance of modelling the complexities of the construction sequence and the compromises made to accommodate typical ground investigation data.

Keywords: Excavation; Basement; Monitoring; Modelling; Finite Element

1 INTRODUCTION

This London case study presents the modelling of basement construction on a high-density site with a mix of retained and new structures. The analysis method and soil model were selected due to time constrains and the limitations of the available soil testing. This combines a series of two dimensional (2D) finite element cross sections, employing PLAXIS. The three-dimensional (3D) ground movements and building damage assessment were extrapolated using the 2D profiles within the software programme Oasys XDisp.

The proximity and sensitive nature of the retained structures requires a sympathetic construction sequence and monitoring. Ground movements during the ongoing construction at Olympia are being monitored with a series of inclinometers in the secondary secant piles and fixed targets at the capping beam. This provides the opportunity to validate the predicted movements and therefore the assumptions required in adopting a simplified finite element approach.

2 PROJECT

The Olympia project is a redevelopment of the entire site in west London to ensure that it maintains its status as a world class exhibition destination. The existing buildings will be improved and enhanced whilst delivering a range of supporting and complimentary uses. Additional buildings will be constructed on the remaining open space, leaving the site constrained on all sides by roads, existing buildings and an overground railway.

The existing exhibition spaces of the Grade II* and II listed buildings Grand Hall and National Hall will be retained, with a public garden and retail space constructed between and above them. Several of the other existing buildings will undergo cut and carve refurbishment to add additional capacity or change of use, for example from a multistorey carpark to an office or a hotel.

This paper focuses on the areas named Central Hall and G-Gate. G-Gate was previously used as an open-air car park. A new theatre structure will be constructed in this space, including a new two storey basement partially shared with the neighbouring Central Hall. Central Hall is an existing 4 storey exhibition and conference hall, with a partial single storey basement. The existing façade will be retained whilst replacing the building behind and constructing a deeper one storey basement. The new building will be used as office space with incorporated exhibition space. Central Hall will share a basement with G-Gate, but in addition is surrounded by the very sensitive Grand Hall and National Hall. These structures are formed of masonry and steel, each supporting glazed barrel-vaulted roofs.

2.1 Foundations

The existing Central Hall foundations, as well as those for Grand and National Hall, are shallow pad foundations founded on the Kempton Park Gravels. The level of the pads varies between 2.4m and 7.4m below ground level. The Central Hall pad foundations and existing basement walls will all be removed to form the new basement, except for the pads and basement wall supporting the retained façade.

The new basement walls for Central and G-Gate will be formed of a secant pile wall. Central Hall's secant pile wall is located within 1m of Grand Hall's pad foundations and within 10m of the National Hall foundations. To minimise the effect on the existing structures, the basement will be constructed using the top-down method, where the ground floor is in place before excavation commences. This method provides more rigidity to the secant piles during excavation and therefore reduces the lateral movement of neighbouring areas.

The main foundations for the new buildings will be supported on piles as plunge columns are required for the top-down construction sequence. The bearing and plunge column piles range in diameter from 0.9m to 2.4m and carry loads of up to 26MN. The secant piles are typically 0.9m in diameter and the secondary secant piles 15 to 25m long. Where secondary piles are required to support columns from the superstructure they are up to 2.4m in diameter and up to 55m long. The primary piles are 9 to 12m long, providing a minimum of 2m embedment into the London Clay or 2m below the formation level of the basement slab.

2.2 Construction sequence

The planned construction sequence incorporated into the ground movement analyses was:

1) Apply loads of existing building (Central)
2) Demolition of existing building (Central)
3) Installation of perimeter secant walls and piles
4) Construction of ground level slab (+4.5mOD)
5) Excavation commences
6) Jump-start of core walls & superstructure construction
7) Complete basement excavation & construct lower basement slab.
    a. Level B2 slab in G-Gate (-4.2mOD)
    b. Level B1 slab in Central (-0.1mOD)
8) Install Level B1 slab in G-Gate

The jump-start of the core walls and superstructure allows for a reduction in overall programme but loads up the piles and secant walls during the excavation phase before the support of the basement slabs are in place. A limit is placed on the height of the core and superstructure construction prior to completion of the basement. The ground movement assessment considers the intermediate loading stages of the piles.

3 GROUND MODEL

The ground below the Olympia site is typical of west London geology. A layer of Made Ground has replaced or been placed upon the superficial deposits, in this case Langley Silt and Kempton Park Gravels. Beneath these lies a thick layer of London Clay, approximately 60m thick, followed by the Lambeth Group, Thanet Sands and Chalk.

The Kempton Park Gravels is part of the fluvial Pleistocene deposits commonly found in London and known collectively as the River Terrace Deposits. They are primarily composed of gravel and sand and often used to support shallow foundations. The layer of London Clay is of sufficient strength and depth to support deep pile foundations.

3.1 Ground investigation

The ground investigation testing was specified by the previous designers and focuses on pile design rather than ground movement assessment. Furthermore, the existing, in-use buildings limited the locations available for investigation. The deep boreholes were limited to locations on the roads between the buildings and two were completed in this area, see BH02 and BH05 in Figure 1. The ground beneath the existing basement at Central Hall was confirmed using small windowless sampler rigs that fit within the existing basement but provided lower quality samples for testing.

3.2 Soil properties

Geotechnical parameters were derived from the undertaken in-situ Standard Penetration Tests (SPTs) and laboratory tests, the latter including particle size distribution, Atterberg Limits and Undrained Unconsolidated or 'quick' Triaxial tests (UUTs). Effective strength and stiffness parameters were suggested for the strata containing mainly granular material (Made Ground and Kempton Park Gravels), while additional undrained shear strength values were derived for the cohesive strata (Langley Silts, London Clay and Lambeth Group).

For the London Clay, the undrained shear strength, c_u profile was derived using the empirical SPT correlation $c_u = N \cdot f_1$ (Stroud, 1988) with a value for f_1 of 4.5 appropriate for the measured plasticity. The results of the UUTs were used to validate the obtained profile of undrained shear strength with depth, as given in Equation (1), where z_1 is the depth (m) from -2m OD and z_2 (m) is the depth from -25m OD.

$$c_u = 90 + 7z_1 \; and \; c_u = 251 + 3z_2 \qquad (1)$$

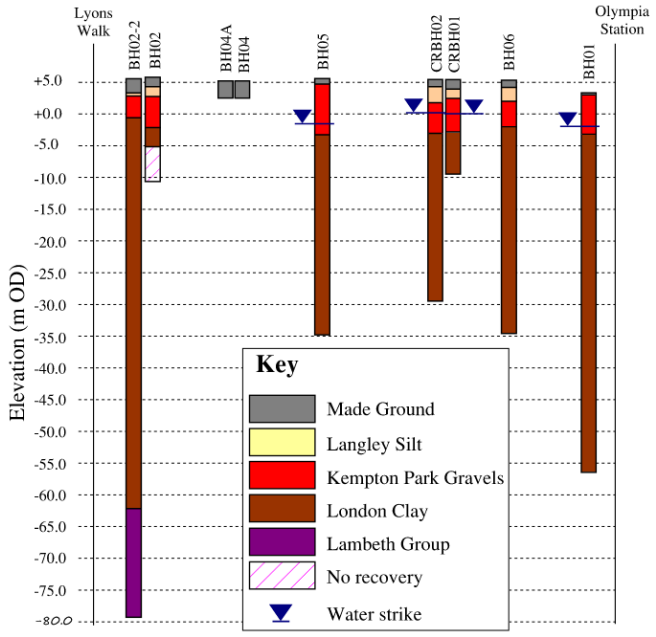

Figure 1. Cross section of boreholes from SW to NE

No tests were performed that directly provided drained soil properties or compressibility parameters for the London Clay. Stiffness values were therefore selected using the correlations with undrained shear strength recommended in CIRIA C760 (Gaba et al. 2017). The selection of soil properties in the finite element soil models are discussed further in Section 4. To determine an appropriate level for the phreatic surface, groundwater levels were monitored using dipmeter piezometers in several boreholes across the site. The groundwater within the superficial deposits was between 0.0m OD and -1m OD. A level of 0m OD was used in design.

4 ANALYSES

Due to programme constraints, a series of 2D finite element cross sections were analysed instead of developing a full 3D model. The predicted movements for these analyses were extrapolated to three dimensions employing the XDisp programme, which also facilitated an assessment of the predicted building damage categories, following Burland (2001) and Burland & Wroth (1975).

4.1 Finite element soil model

Various advanced models have been developed to better capture the complexity of modelling stiff over-consolidated clays. However, the input parameters typically require advanced soil testing (e.g. pressuremeters, advanced triaxial tests) to determine the stiffness in the appropriate strain regions. Considering the time and costs of these tests, they are often not specified in practice and only conventional soil testing to analyse behaviour at large strains is generally undertaken.

Taking into account the ground investigation data available for this project, the only material model deemed suitable apart from Mohr-Coulomb (MC) is the Hardening Soil model (HS). The MC model assumes a constant linear elastic stiffness which may lead to unrealistic movements with varying stress and strain levels. For example, heave movements following demolition and excavation are often overpredicted due to the lack of distinction between stiffness following unloading compared to primary loading. The HS model uses a hyperbolic relationship which captures the stiffness based on the stress-strain history. Despite this advantage, these models are still limited by the inability to account for the anisotropy and strain softening behaviour of the London Clay.

The HS model requires more input parameters which can be derived and justified based on standard use, literature (Lees, 2016; Obrzud & Truty, 2018) and comparison with the Mohr-Coulomb model. The HSsmall model requires two additional parameters to capture the strain-dependent stiffness at small strains. This model was not employed as the additional parameters for the small-strain shear modulus could not be derived from the available soil tests on site.

To model undrained soil behaviour in PLAXIS, one of three models are available: A, B and C. Undrained C uses undrained soil parameters; no distinction is made between effective stress and total stress, resulting in no computation of excess pore pressures. This drainage type is not suitable when assessing both the short-term and long-term response of the soil.

Undrained A and B both use effective stress analysis with the difference that the undrained shear strength is an output from the constitutive model in Undrained A and an input parameter in Undrained B. The limitation of the latter is that excess pore pressure may be inaccurate, and the shear strength remains constant, therefore it is not recommended to carry out a consolidation analysis. Undrained B is generally suitable when soil failure is driving the analysis, as the soil will always fail at the defined undrained shear strength. However, it is less accurate for ground movement assessment when structural elements are involved.

Undrained A is considered more appropriate for the computation of excess pore pressure, as the stress path is more reliable, and the undrained analysis can then be followed by a consolidation stage to predict long-term ground behaviour. It should be noted that the mobilised undrained shear strength should be checked against known data; in soft clays the strength can be overestimated resulting in catastrophic consequences, while in stiff clays the strength is typically underestimated.

The tool SoilTest was used in PLAXIS to simulate laboratory tests to compare the stiffness and strength of the London Clay at two depths for the Mohr-Coulomb and Hardening Soil models as well as Undrained A and Undrained B to validate the soil model and input

parameters. The results were analysed to confirm that the undrained shear strength and stiffness for certain strain ranges and depth were in line with the data from the soil investigation. The use of Undrained A resulted in a calculated c_u far below the specified strength, almost half in the case of the HS and Undrained A combination. The effective stress parameters were increased slightly to compensate for this effect, but still remaining within typical values adopted for London Clay with a φ' = 25° (increased from 22°) and c'=5kPa.

Despite the low mobilised undrained shear strength, Undrained A was chosen as the preferred drainage type to model the basement excavation with undrained analysis followed by a consolidation stage to assess the long-term behaviour. Soil failure was not a concern and underprediction of the undrained shear strength was therefore considered acceptable.

To determine the stiffness of the London Clay, the correlations of $E_u = 600c_u$ and $E'=0.8E_u$ were adopted, assuming that the majority of the axial strains will be below 0.1%, which is typical for retaining walls (Atkinson, 2000). Undrained triaxial tests were simulated using the Hardening Soil model to match the secant Young's modulus at 0.1% and 0.01% axial strain with typical correlations of $E_u = 400c_u$ and $E_u = 800c_u$, respectively. The unloading-reloading Young's modulus was taken as three times the reference secant modulus.

4.2 *Finite element predictions*

The location of the cross sections modelled in Plaxis 2D are presented in Figure 2. The total displacement contours at the end of the excavation phase are presented for G-Gate in Figure 3 and in Figure 4 along with the Central Hall cross sections. Later stages (including consolidation) are not included as construction is ongoing at the time of writing. The predicted movements for the one storey Central Hall basement are small with a maximum value of approximately 6mm. For the two storey G-Gate basement, up to 25mm of movement was predicted. The predicted and monitored movement at the top of the wall due to the top down construction methodology results in low movements behind the wall at shallow depths, protecting the adjacent foundations of the façade and Grand Hall building.

Table 1: Soil properties in Plaxis 2D

Stratum	**Top of Stratum (m OD)**	**φ' (°)**	**c' (kPa)**	**E'$_{ref}$ (MPa)**
Made Ground	+5.5	28	0	10
Langley Silt	+4.0	27	1	24
Kempton Park Gravels	+2.0	36	0	45
London Clay	-2.0	25	5	NA*
Lambeth Group	-62.0	28	10	192

* Non-linear stiffness with $E'_{50}{}^{ref} = 43.2$ MPa and $p_{ref} = 125$ kPa (top of London Clay)

4.3 *Oasys XDisp predictions*

The horizontal and vertical movements behind the wall in the finite element cross sections were used as input to the Oasys XDisp model to create displacements contours around the excavation and predict the movements at adjacent building footings. The stiffening from 'corner effects' has been accounted for in accordance with (Fuentes & Devriendt, 2010). Empirical literature curves from CIRIA C760 (Gaba et al. 2017) have been used to assess the impact due to the installation of the perimeter secant piled walls. For all the sensitive assets surrounding Central Hall, a damage Category 0 (negligible) was predicted, following Burland (2001) and Burland & Wroth (1975). The maximum settlements of the adjacent footings were limited to 10mm due to this methodology. The small movements predicted caused by the top-down construction of the single storey basement support the use of this construction method to protect the surrounding buildings from damage, especially when founded on shallow footings behind and close to the retaining walls.

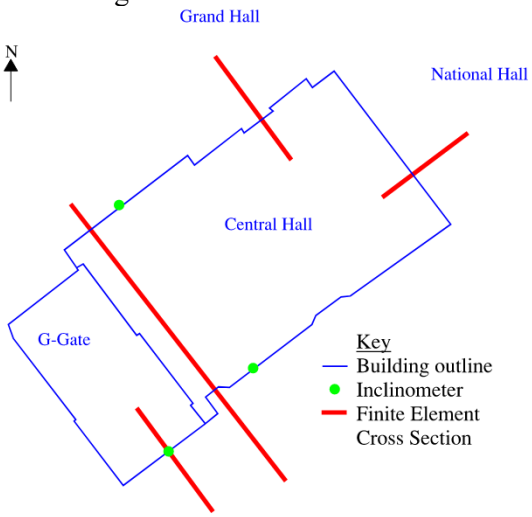

Figure 2. Relative locations of finite element cross sections and inclinometer positions

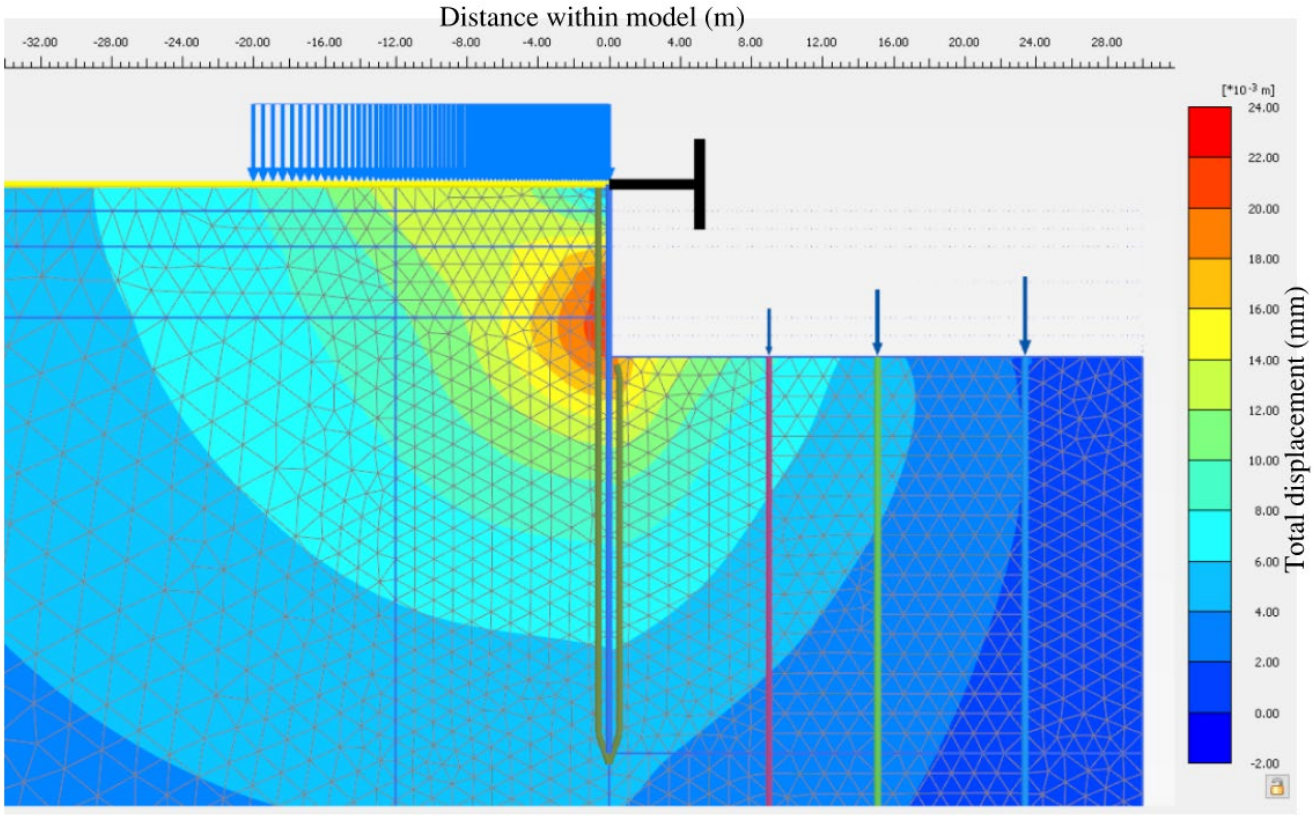

Figure 3. Total displacement contours for G-Gate cross section at the end of excavation stage

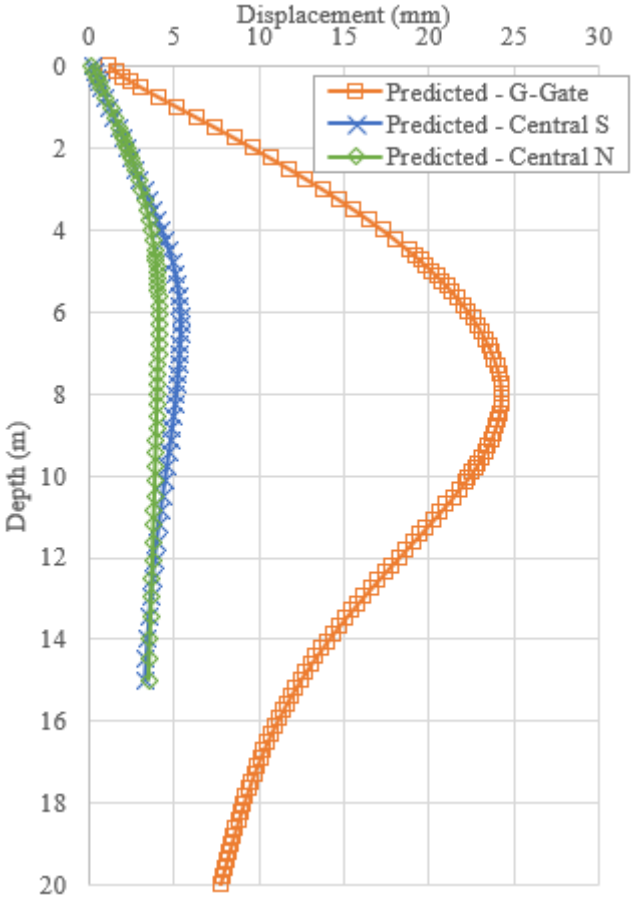

Figure 4. Lateral displacement predictions for secant wall at the end of excavation.

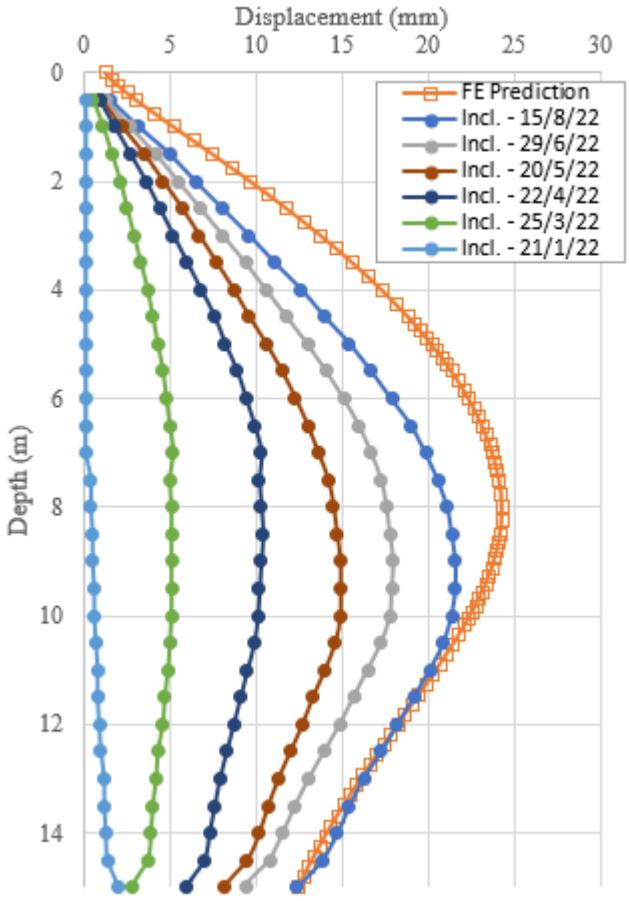

Figure 5. Comparison of inclinometer results to the prediction for G-Gate cross section, total displacements.

5 MONITORING

The existing buildings, surrounding footpaths and new secant walls are undergoing monitoring before, during and after the construction phase. This monitoring is in place to ensure that movements of the sensitive surrounding assets remain at an acceptable level and act as an early warning for preventative measures, should unexpected movements occur. The data can also be used to validate the predictions made by the finite element analyses.

5.1 Equipment

Inclinometers have been installed within the new secant pile walls surrounding the G-Gate and Central Hall basements. These are manually read at intervals, with the displacements reported relative to the movement of the top point of the inclinometer close to the capping beam. Targets at ground level and on the secant wall capping beams provide a reference point for the movements occurring at this level, that value is then applied to the top value of the inclinometer reading. The location of the inclinometers presented in this paper are shown by the dots in Figure 2.

5.2 Results

At the time of writing this paper, the excavation of the G-Gate basement is complete, but the excavation of the Central Hall basement is ongoing. The finite element prediction and inclinometer readings comparison is therefore limited to the G-Gate cross section, located as shown in Figure 2.

Figure 5 shows the increase in wall movement over several months, from the start of the basement excavation in early 2022 to the completion of excavation at the end of July 2022. Only the top 15m of the prediction is shown, as this is the length of the inclinometers. By the end of excavation, the shape of the curve from the inclinometers is similar in shape to the prediction, matching the movements from 10m below ground level and overestimating movements above this.

One cause of this overestimation of movements could be the wall stiffness. In the finite element model, the contribution to stiffness of the primary piles in the secant wall was ignored. The toe of the primary piles is approximately 10m below the top of the capping beam and corresponds to the length over which the predicted movements exceed the measured values. The spacing of the secondary piles was also altered in the final stages of design and no consideration was taken for the presence of the larger plunge column piles within the wall. The wall stiffness modelled is therefore not representative of the as-built conditions.

6 CONCLUSIONS

The monitoring data from the inclinometers show good consistency with the predictions of embedded retaining wall movements for the excavation of the basements at this stage. The differences between the modelled and final construction sequence have made comparison more difficult for some areas of the project. However, the adopted top-down construction method has clearly limited the movements around the excavation which has helped to protect existing foundations of structures behind and in close proximity to the basement retaining walls.

Ground movement assessments undertaken using 2D finite element methods often require input parameters which are uncertain, and assumptions need to be made. The analyses and monitoring data presented demonstrate that even when the available ground information limit the analysis to a more simplified approach, realistic predictions can be made by adopting informed assumptions about soil behaviour using a more advanced soil model with non-linear stiffness for London Clay. It is important to note that the user needs to be aware of the advantages and limitations of each model and analysis type and results should be compared with available empirical data and similar case studies.

7 ACKNOWLEDGEMENTS

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