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'Real-time' back-analysis of a shaft excavation in London Clay using the propped contiguous piled wall

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ABSTRACT: A square shaft excavation in the over-consolidated London Clay was supported by the contiguous piled wall with multiple levels of temporary props. The 3D finite element model was adopted for design analyses. During the shaft construction, the 'real-time' back analyses were trialled by 2D finite element models on DAARWIN® - a data platform that uses the machine learning algorithm to conduct back analyses. Challenges encountered in the back analyses trial comprise: i) 2D modelling versus actual 3D excavation behaviour; ii) the exposed London Clay surface induced by contiguous piled-wall; iii) review and implementation of the reliable monitoring data in relation to the as-built construction activities; and iv) interpretation of the back analysis outcome. This paper discusses these challenges and the proposed solutions, and also presents the calibrated 'best-estimated' London Clay stiffness values in deep excavation with a less stiff retaining wall system.

Keywords: real-time back analysis (RTBA); excavation; soil-structure interaction.

1 INTRODUTION

A 20m deep square shaft was excavated using a bottom-up construction technique with three levels of temporary props to minimise the duration of the excavation to the formation level. This arrangement was in association with the earliest construction of the base slab, hence, facilitating the connection with the constructed tunnels in advance of the other planned site construction activities, as demonstrated in Figure 1.

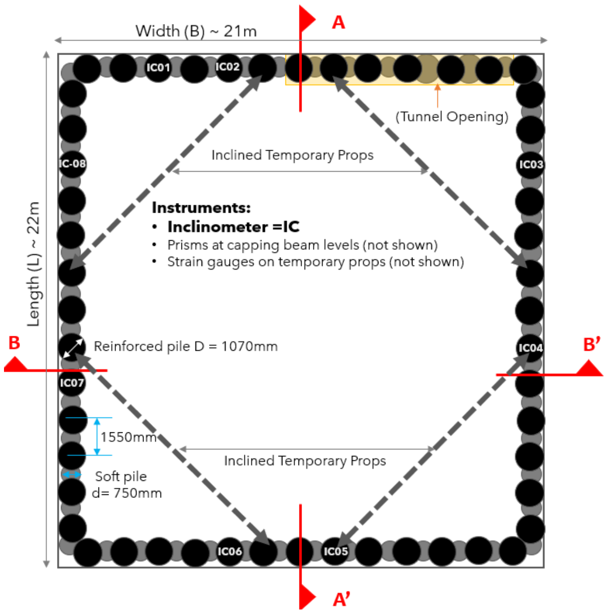

Figure 1. Layout of square shaft

The permanent shaft design was carried out by the 3D finite element model analysis. The linear elastic perfect plastic Mohr-Coulomb soil constitutive model was adopted modelling the over-consolidated cohesive soil behaviour.

A monitoring plan was designed to closely measure the excavation-induced displacement during the shaft excavation. The monitoring data review was part of the 3rd party's asset protection measures, to provide early warning if the agreed trigger values were approached. Given the monitoring data was provided in 'real-time' (available within 24 hours after measurements were taken), this allowed the trial of 'real-time' back analysis to better understand the soil-structure interaction in the actual construction environment.

2 CASE HISTORY

2.1 Confirming design

The square shaft is approximately 21m by 22m in plan with a maximum excavation depth up to 20m, except for a corner sump with an additional couple of meters of excavation. The contiguous piled-wall is adopted as the retaining structure with the reinforced pile in 1.07m diameter at 1.55m spacing c/c, and the soft grouting pile in 0.75m diameter penetrated 3.0m into London Clay. The toe level of the reinforced piles was tailored for the excavation support varying from -7.0mAOD to -10.0 mAOD. The minimum temporary prop stiffness value was assumed in design, hereby, the allowable maximum prop load at three propping levels was indicated in Figure 2.

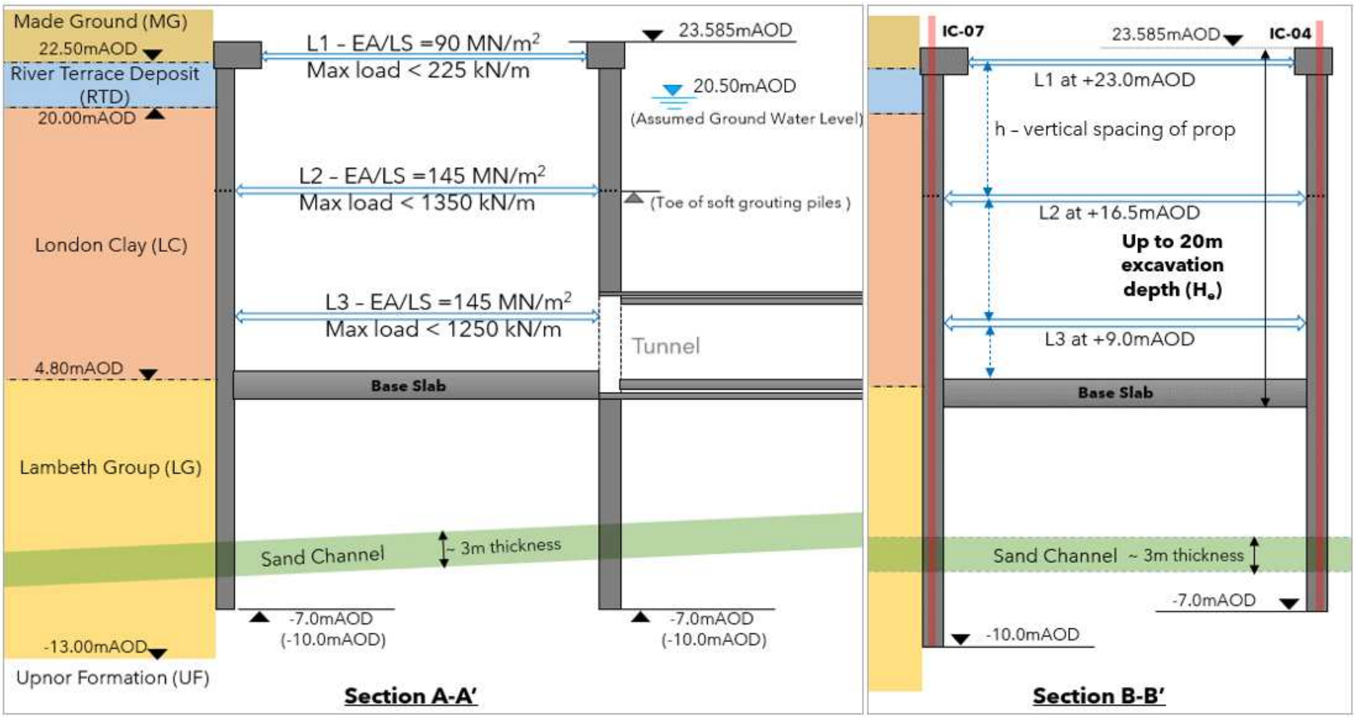

Figure 2. Shaft analysed sections

Table 1. Summary of major geotechnical design parameters

Parameters/ Stratum	RTD	LC	UMB	Sand	UF
Density γ (kN/m^2)	19.0	20.0	20.5	21.5	21.0
Friction angel ϕ'_{peak} (^{o})	33.0	24.0	27.0	33.0	33.0
c' (kN/m^2)	0	5	5	0	0
Undrained shear strength c_u (kN/m^2)	-	70+7.5z	50+10z	-	-
Poisson's ratio ν' (ν_u)	0.30	0.125 (0.5)	0.125 (0.5)	0.125	0.125
Earth Pressure Coefficient K_0	0.50	*1.0**	*1.0**	*1.0**	*1.0**
Undrained stiffness E_u (MPa)	-	1000 c_u	1000 c_u	-	-
Drained stiffness E' (MPa)	20	0.75E_u	0.75E_u	150	200

*Note: z =0 at level of +20.5mAOD. MG= Made Ground; RTD=River Terrace Deposits; LC=London Clay; UMB=Upper Mottled Beds – a layer within Lambeth Group; UF=Uponor Formation. * In-situ K_0 value from GI is >1.0, the adjusted value of 1.0 to account for stress relaxation from piling installation and excavation.*

2.2 Ground condition

The shaft ground condition is the typical London Basin geology profile, that superficial deposits overlaid above the London Clay. Based on the limited ground investigation information, it has assumed that a sand channel in average of 3.0m thick presented in the Upper Mottled Beds (UMB). The sand channel was inclined along the Section A-A' as shown in Figure 2.

A hydrostatic groundwater pressure as a cautious assumption was applied in the design. In addition, perched water from the ground level for superficial deposits (MG & RTD) was considered in the Ultimate Limit State (ULS) analysis. Given the planned excavation period was less than 6 months, the total stress analysis was then assumed for the excavation stage analysis. However, there was no piezometer installed near the shaft to validate the groundwater pressure profile.

Table 1 summarised the ground model and geotechnical design parameters used in the confirming design.

2.3 Construction sequence

Table 2 summarises the proposed excavation sequence and the as-built construction dates for key excavation stages.

Table 2. Summary of construction sequence for square shaft

Stage	Description	Start[1]	End[1]
1	Install piled-wall	*Jul 21*	*Nov 21*
2	Construct tunnel	*Nov 21*	*April 22*
3	Dig1 to +22.5 (+21.4)[2]	*Jan 22*	*n/a*
4a	Cast Capping beam	*Jan 22*	*Mar 22*
4b[3]	Prop 1 at +23.0 (+22.5)[2]	05-03-22	29-03-22
5	Dig2 to +18.8 (+18.0)[2]	*05-03-22*	*26-03-22*
6	Dig3 to +14.8 (+15.0)[2]	30-03-22	06-04-22
7[3]	Prop 2 at +16.5 (+17.0 & +16.0)[2]	07-04-22	26-04-22
8	Dig4 to 12.7 (+12.2)[2]	27-04-22	05-05-22
9	Dig5 to +9.0	06-05-22	10-05-22
10	Dig6 to +7.5 (+7.7)[2]	11-05-22	15-05-22

11[3]	Install Prop 3 at +9.0	16-05-22	20-05-22
12a	(Dig7a to +4.9)	21-05-22	28-05-22
12b	Dig7b FEL +3.3 (+3.9)[2]	06-06-22	09-06-22
13	Cast Base Slab	*10-06-22*	*16-07-22*

Note: 1) construction activity dates reviewed for Inclinometer IC-04, dates may differ at other locations. 2). as-built level given in bracket; 3). as-built temporary props with pre-loading, 24 Nos. of diagonal props installed as P2 over two levels. 4) stage 1 to 4a, 5, and 13, dates were indicative only due to uncertainty in validation.

3 ‘REAL-TIME’ BACK ANALYSIS TRIAL

Back analysis is a procedure of using field observations to reduce uncertainty and identify material parameters in the numerical model for analysis and design purpose (Gioda & Maier, 1980).

DAARWIN® is a cloud-based data platform, offering a rapid, close to ‘real-time’ process of back analysis through the built-in machine learning Genetic Optimization (Santos, 2015) algorithm. With the provided design numerical models and the reviewed monitoring data, DAARWIN creates a ‘digital twin’ to compare design against observations. Through the parametric study function the influential model parameters are identified. They can be selected as variable parameters and calibrated in back analysis to find their ‘optimal values’. In this way, DAARWIN significantly improves back analysis performance in terms of computation efficiency and finding the ‘optimal value’. Moreover, it offers an opportunity for a performance-based design approach to be adopted in fast-paced construction, such as Observational Method (Peck, 1969; Gaba *et al.*, 2017).

3.1 Initial review (2D vs 3D)

The initial review compared the 3D model predictions of the conforming design against a plane strain Plaxis 2D model predictions. Figure 3 presented the wall deflection predictions from key excavation stages at the location of inclinometer IC-07 on Section B-B’. It can be seen that there are similar deflection profiles from both 2D and 3D predictions, whilst the 2D model estimated the higher maximum deflection but a more restrained wall displacement at the top.

According to previous studies (Ou *et al.,* 1996 & 1998; Fuentes *et al.,* 2007), the geometry of excavation could affect the predicted ground response between 3D and 2D analyses. Finno *et al.*, (2007) used the plane strain ratio (PSR) to indicate this difference. This empirical ratio is in relation to excavation geometry (L/H_e, L/B), wall system stiffness ($S=EI/\gamma_w h^4$), and factor of safety against basal heave (FS_{BH}). The lower PSR ratio means a significant difference between the 3D and 2D response. The wall system stiffness was investigated and found that the stiff system produced lower PSR than the flexible system, see Figure 4. The calculated PSR for the square shaft confirming design was included in Figure 4, showing PSR ratio of 0.8 from the final dig stage. This is comparable to the ratio of the maximum wall deflections by the 3D model versus the 2D model at the final dig presented in Figure 3.

Therefore, it is considered that the Plaxis 2D predictions are representative of the shaft excavation displacement with the corresponding PSR ratios. Besides, the updated PSR ratios by the shaft as-built record indicate the negligible difference between 2D and 3D responses.

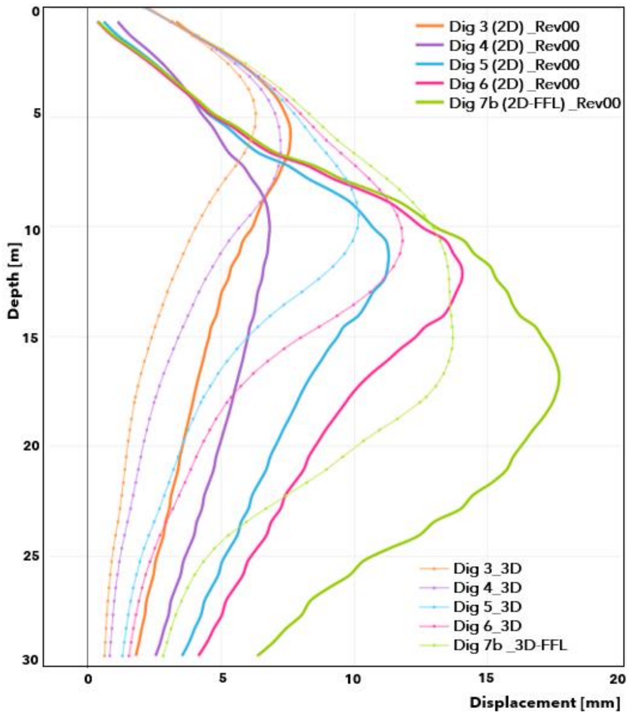

Figure 3. Comparison of wall deflections: 3D vs 2D

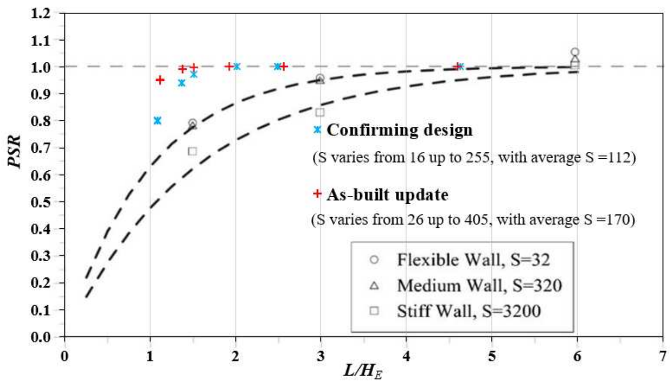

Figure 4. Effects of wall system stiffness (S) for L/B=1 (Finno et al., 2007)

3.2 Review monitoring data

Inclinometers IC04 on Section B-B’ recorded the greater horizontal displacements during the early excavation stages, the data was reviewed ensuring the quality of data and suitability for the back analysis.

The as-built construction activities were also reviewed for the individual locations. For example, the as-built dates at IC-04 were summarised in Table 1. On DAARWIN, these input dates will automatically filter

IC-04 monitoring data into corresponding construction stages for design visualisation and analyses.

3.3 Back analysis

The back-analysis trial was undertaken in the following steps: 1) compare the 2D model predictions against the measurements; 2) perform a parametric study to identify the significant influential parameters; 3) set up and run back analysis; and 4) interpret the back analysis outcome.

During this process, the 2D model was constantly reviewed and manually updated, reflecting the change from the as-built ground and construction conditions. Sometimes, model inputs and assumptions were updated to achieve matched predictions. A record of the shaft 2D model revision was summarised in Table 3.

Table 3. Summary of 2D model revision

Revision	Model Description & major changes
00	2D model on Sec B-B'
01	Add capping beam & pre-loading in props
02	Adjust LC stiffness E after back analysis at Dig 3
03	Remove Sand layer
04	Update Dig 4 level to as-built +12.2mAOD
05	Update P2A at +17mAOD, P2B at +16mAOD & Dig 6 level to as-built +7.7mAOD
06	Update LC with three sub-layers A3/A2/A1 after back analysis at Dig 6 & prop P2 stiffness
07	Update pre-stress loading at P2 and P3
08	Include a 'stress relaxation' zone behind walls & Introduce strain-related-stiffness Mohr-Coulomb stiffness E in LC and UMB
09	Apply 'best-estimated' soil stiffness E after back analysis targeted on Dig 4, 6 & 7b.

3.3.1 Comparison of model prediction vs measurement

The comparison between the 2D model predictions and measurements from the inclinometer IC-04 was presented in Figure 5. The improved predictions were obtained through the 'real-time' back analysis trial and process of updating the 2D model. For example, after the introduction of the 'stress relaxation' zone behind the walls and strain-related stiffness input for London Clay sub-layers in model rev08, the significant improvement in predictions was observed. Model rev09 adopted the interpreted 'best-estimated' parameters from the back-analysis targeted at 3 dig stages, the predictions were further refined compared to model rev08.

3.3.2 Parametric study

Parametric study on DAARWIN reviewed the major Plaxis 2D model inputs, such as soil constitutive model related parameters (e.g., stiffness E and strength of c', φ', c_u), and structural parameters (e.g., wall stiffness EI, prop stiffness EA). However, if the ground model, groundwater, or loading assumptions were affecting the prediction, a manual check of the Plaxis 2D model will be necessary.

An example of the parametric study of the undrained shear strength (c_u) for LC-A1 was demonstrated in Figure 6. The c_u value has a negligible impact on the wall deflection predictions at Dig5 (Maximum deflection variance Δ <1.0mm), however, a significant impact was observed at Dig 7b (Δ >6.0mm). Therefore, a lower c_u value of 150 kPa was recommended in the 2D model to receive better predictions at different excavation stages.

Because the total stress was applied for the shaft excavation analyses, the soil undrained shear strength (c_u) and undrained shear stiffness (E_u) were found as the most influential Mohr-Coulomb soil parameters in the back-analysis trial using the inclinometer data.

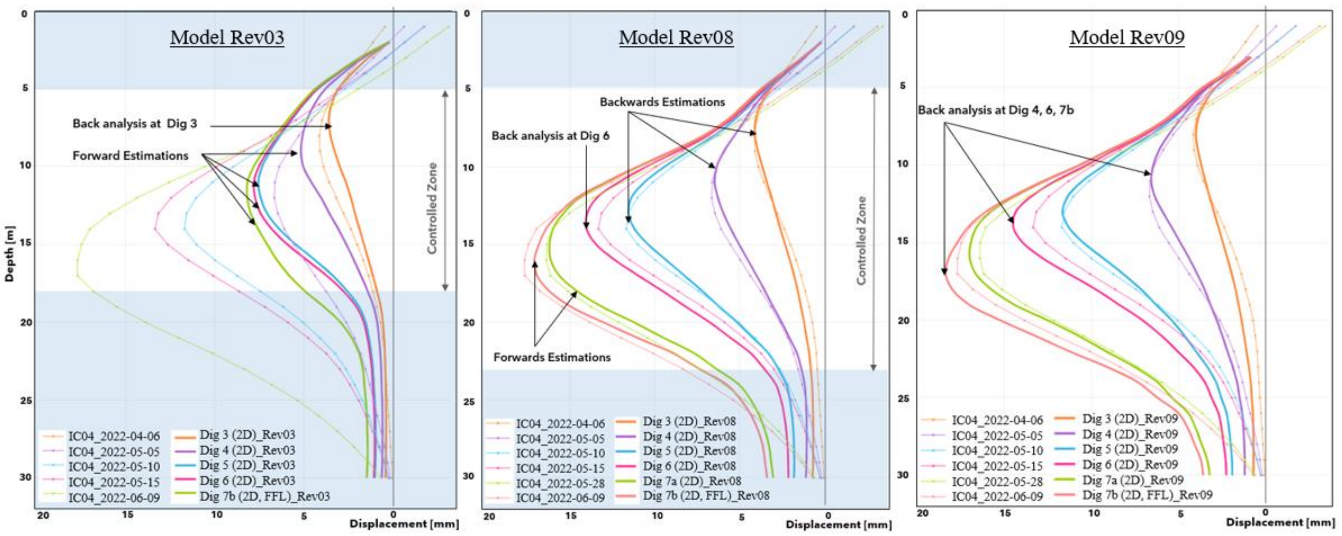

Figure 5. 2D model predictions versus measurements at IC-04 location

In addition, the influence of prop stiffness (EA) was noticed and considered in the back-analysis.

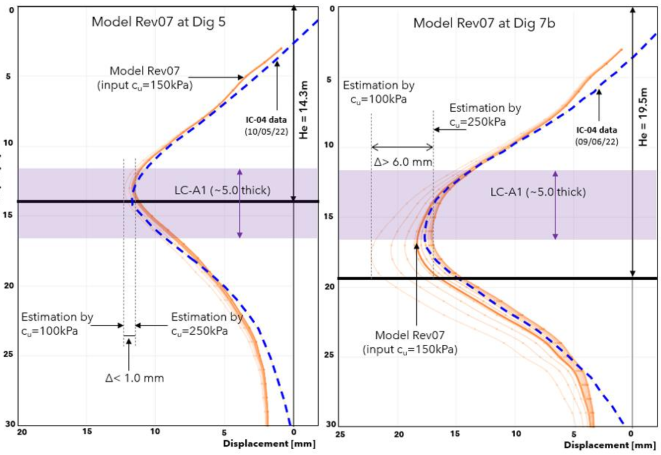

Figure 6. Parametric study of LC-A1 (c_u) by model Rev07

3.3.3 Run back analysis

Based on parametric study, the variable parameters were determined from influential parameters for back analysis using the inclinometer data. The measurements could be selected from single or multiple excavation stages. For instance, in Figure 5 the back analysis by model rev03 targeted Dig 3, whilst the back analysis by model rev08 firstly targeted Dig 6 only, and later targeted all Dig 4, 6, and 7b stages. After back analysis, the improved predictions at the targeted dig stages were obtained.

If correlations between different variables can be established, it will enhance the efficiency of back analysis. As more variables will be evaluated without additional computational load.

Besides the variable parameters, the range of parameters needs to be decided, referring to the available ground investigation information or the known empirical correlations (e.g., E_u/c_u of 1000 for retaining wall structure by Gaba *et al.*, 2017).

A controlled zone is recommended for back analysis, due to the inevitable errors in the measurements (e.g., instrument accuracy, calibration, and field condition), and potential errors induced by numerical modelling. This is a specific zone where confidence can be ascertained for both 2D model prediction and monitoring data to be robust. The controlled zone shall adjust for each construction stage depending on the 2D model performance and the quality of monitoring data, as the examples demonstrated in Figure 5.

Following the above settings, DAARWIN runs back analysis automatically, and the work-in-progress outcome is available to view.

3.3.4 Interpretation of back analysis outcome

In terms of outcome, multiple combinations of variables with different values are provided on DAARWIN, in the order of the least error to the targeted monitoring data. However, the least error index is directed to a mathematical optimistic solution, not necessarily representing the ‘best-estimated’ solution with full physical geotechnical meaning. Hence, a further review by experience engineer is required to interpret the outcome and derive the ‘best-estimated’ parameters.

Table 4 summarised the ‘best-estimated’ soil stiffness for London Clay, interpreted from the back-calculated combinations of model rev08 targeting at Dig 4, 6, and 7b. Three Mohr-Coulomb soil stiffness values at different shear strain levels were proposed to represent the non-linear soil stiffness: $E_{u,Large}$ for ε<0.02%; $E_{u,Middle}$ for ε from 0.02% to 0.05%; and $E_{u,Small}$ for ε>0.05%. The recommended shear strain range was referenced to mobilised shear strain developed in the Plaxis 2D model rev07 and calibrated by model rev08 & 09.

In the back analysis, three stiffness parameters plus the stiffness for the stress relaxation zone ($E_{u,Rel}$) and corresponding undrained shear strength (c_u) were treated as variables in the initial back-analysis. The correlations between these parameters were formulated from the outcome. Thus, the reduced variables could be further calibrated in the back-analysis.

As key variable parameters, the London Clay stiffness at large strain $E_{u,Large}$ and stiffness in relaxation zone $E_{u,Rel}$, value range and the interpretated ‘best-estimated’ values were shown in Figure 7.

Table 4 Summary of ‘best estimated’ soil stiffness for London Clay (LC) sub-layers

LC sublayers / Parameters	**LC-A3**	**LC-A2**	**LC-A1**
c_u (kN/m^2)	70+7.5z		150*
$E_{u\text{-}Large}$, (strain > 0.05%)	1000c_u	850c_u	800c_u
$E_{u\text{-}Middle}$, (strain 0.02-0.05%)	1.6$E_{u\text{-}L}$	1.25$E_{u\text{-}L}$	1.4$E_{u\text{-}L}$
$E_{u\text{-}Small}$, (strain < 0.02%)	2.0$E_{u\text{-}L}$	1.6$E_{u\text{-}L}$	3.0$E_{u\text{-}L}$
$c_{u\text{-}Rel}$ (kN/m^2)		0.5c_u	
$E_{u\text{-}Rel}$		750$c_{u\text{-}Rel}$	

** Revised c_u for LC-A1, the confirming design value refers to Table 1. z =0 at +20.5mAOD. Rel = relaxation zone.*

4 DISCUSSION & CONCLUSION

4.1 Modelling stress relaxation zone

With the progress of excavation, London Clay surface zone between contiguous piles was exposed below the soft grouting pile toe level. This exposed zone has a width of about ½ D, the reinforced pile diameter. Based on IC-04 data, the horizontal displacement trend revealed the ‘stress relaxation’ or ‘softening’ occurred in the exposed zone causing the reduction in soil stiffness. The relaxation percentage is associated with exposure time.

A geometry study suggested that a relaxation zone up to 3D width is necessary to capture this 'stress-relaxation' behaviour.

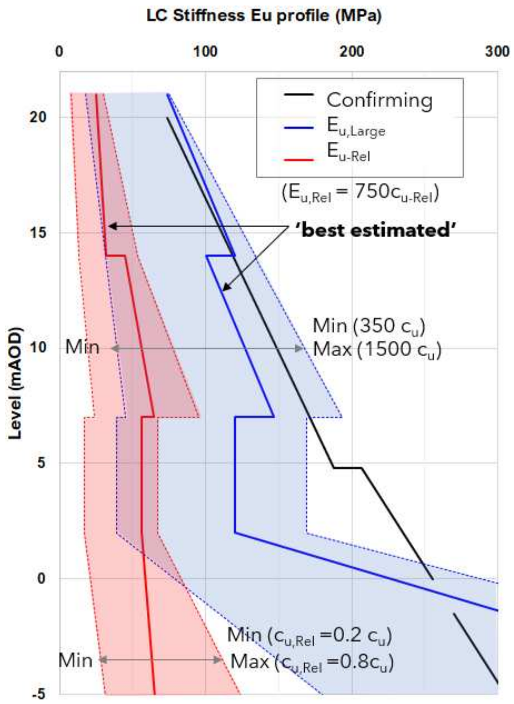

Figure 7. Range & 'best-estimated' values for key variable stiffness parameters

4.2 Modelling strain-related-stiffness by Mohr-Coulomb model

Soil experiences non-linear stiffness with varied strain (Jardine *et al.*, 1986; Atkinson, 2000). A series of advanced soil constitutive models have been developed to simulate the non-linear stiffness characteristic. However, the application of the advanced soil model involved a complex procedure to derive input parameters. In addition, the increased model parameters also added complexity to the back analysis, as well as the computational time.

This back analysis trial adopted three Mohr-Coulomb stiffness values to represent the non-linear stiffness. The matching predictions from model rev08 & 09 suggested a way forward for the simple soil constitutive model to be continually utilised in design and back-analysis.

4.3 Conclusions

The DAARWIN back analysis trial of the square shaft showcased that it is possible to conduct 'real-time' back analysis in parallel with construction. The parametric study and back analysis outcomes will contribute to a better understanding of ground behaviour in the actual construction. Moreover, the insightful information can advise on the optimisation of construction, e.g., proposal of the alternative construction method to achieve savings in cost, time, and better sustainability, and improving construction risk management.

A few modelling assumptions were tested for the efficient modelling and back analysis, such as using a 2D model to represent a 3D construction, adopting three Mohr-Coulomb stiffness values to represent the non-linear stiffness, and adding a zone behind walls to consider the stress relaxation.

The 'best-estimated' London Clay soil stiffness parameters are calibrated in excavation with the flexible supporting system, as a useful reference for future excavation projects. These parameters and the proposed modelling assumptions shall be validated further in future research.

5 ACKNOWLEDGEMENTS

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