Appendix G Ground Movement Assessment

Pell Frischmann

Saville Theatre

Ground Movement Assessment

April 2023

Pell Frischmann Project Reference: 105465

Report Ref.	105465-PEF-ZZ-XX-RP-G-000001-P02 Ground Movement Assessment.docx					
File Path						
Rev	**Suit**	**Description**	**Date**	**Originator**	**Checker**	**Approver**
P01	S3	Draft for comment	April 2023	EMR	KS	AM
P02	S3	Issued for information	05/05/2023	EMR	KS	AM
Ref. reference. Rev revision. Suit suitability. (S2 – suitable for information)						

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Contents

Figures

Tables

Executive Summary

Site Name and Location	Saville Theatre, 135 Shaftesbury Avenue, London WC2H 8AH
Existing Structure	Grade II listed six to seven storey steel framed brick building with two levels of basement supported by reinforced concrete basement retaining walls propped by floor slabs at B1 and B2 level. The ground level is used as a cinema with the upper floors used as office space and storage.
Development Proposal	Remove the internal structure of the existing building, leaving in place the existing two storey basement retaining walls and the existing facades with the exception of the north west wall, which will be dismantled and reinstated as part of the construction works. An additional two storeys of new basement will be formed within the existing basement footprint, the additional basement depth is 8.8m from to the existing B2 slab level to the top of the B4 slab, which is anticipated to be approximately 1.0m-1.5m thick to support the structural loads, therefore the total basement depth will be up to 16.5m over the four levels. Within the footprint of the B4 slab, an additional 3.4m excavation is proposed for sprinkler water tanks.
Ground Model	The ground model has been derived from previous boreholes at the site and historic boreholes in the vicinity of the site, and comprised the following.

Strata	Description	Top of Strata (m bgl)	Thickness (m)
Made Ground	Gravelly CLAY with brick	0	3.5
Lynch Hill Gravel	Medium dense gravelly SAND	3.5	1.2
London Clay	Slightly sandy stiff fissured CLAY	4.7	29.7
Lambeth Group	Very stiff slightly sandy fissured CLAY	34.4	15.6
Thanet Sands	Very dense silty clayey fine to medium SAND	50.0	4.1
Chalk	Unstructured Chalk (Grade D_M)	54.1	-

Groundwater Conditions	Groundwater monitoring undertaken in the previous borehole recorded shallow water at between 4.4m and 5.3m bgl in shallow standpipes at the site. This is anticipated to be perched water on top of the impermeable London Clay, separate from the deeper aquifer within the Thanet Sands and granular Lambeth Group beneath the London Clay.
Temporary Propping	The ground movement assessment has considered a sequence of construction and temporary propping based on Contractor's proposals. Temporary props are adopted at B1 and B2 levels to support the existing reinforced concrete walls during demolition of the existing basement slabs. During excavation, 3 No. levels of props have been considered at 9.70m, 12.80m and 16.20m below ground level.
Basement Substructure	Two forms of basement substructure were considered as part of this assessment, either piled basement walls or an underpinned solution.
Piled Basement Walls	The assessment has considered 600mm piles at 750mm centres with a total pile length of 25.0m. This results in wall movements of up to 60mm of wall deflection, the majority occurring at the formation excavation for the B4 slab, this high deflection indicates that 600mm piles may not be suitable structurally, and that 750mm piles are likely required.
Underpinning Basement Walls	The assessment has considered underpins 1.8m wide, and 1.2m deep, with a height of between 2.6m and 2.8m. The design has considered a minimum concrete curing period prior to excavation of 7 days, which should be incorporated into the sequencing for the works. This results in 50mm of movement of the basement walls, the majority occurring at the formation excavation for the B4 slab.
Piled Wall vs Underpins	The results indicate a similar long term deflection profile behind the walls, with more movement occurring during construction from the piled wall solution, associated with the relative flexibility of the 600mm diameter piles, which would reduce if a larger pile diameter was used. In the long term the structural form of the basement is similar with lining walls cast in front of either the pile or underpins, and propped by the permanent floor slabs.
Excavation Heave	The analysis indicates up to 130mm of heave occurring at the base of the excavation during construction and an additional 40mm occurring in the long term resulting in 170mm total.
Damage Category Adjacent Structures	A damage category assessment has been undertaken in accordance with CIRIA 760 and predicts damage category up to Cat 1 will occur as a result of the works. The anticipated settlement and ground movement has been summarised along adjacent facades. The assessment predicts a maximum settlement of 3mm at the base of the Crossrail tunnel. Adjacent services need to be checked on a case by case basis to form specific agreements with each provider.

Recommendations for Further Work	Geotechnical investigation with stiffness testing to confirm geotechnical parameters. Structural investigation to confirm structural capacity of existing basement walls. Structural design of either the piled wall or underpin solution to confirm the reinforcement requirements. Engagement with supply chain to understand lead times. Development of an instrumentation and monitoring plan. Party wall agreements with adjacent affected structures.

1 Introduction

1.1 Appointment and Scope of Report

Pell Frischmann (PF) were commissioned by Yoo Capital Limited to prepare a Ground Movement and Damage Assessment in support of the proposed redevelopment of the Saville Theatre site, located at 135 Shaftesbury Avenue, London, WC2H 8AH.

The scope of the report is as follows:

- Review available geotechnical information to develop a Ground Model with appropriate geotechnical design parameters:
- Review available information on adjacent structures likely to be affected by the development:
- Assess the possible impact of the proposed basement construction on the adjacent structures and the existing façade of Saville Theatre through a quantitative damage assessment.

An assessment of the geo-environmental effects of the development are outside the scope of this report.

1.2 Site Location and Description

The site is flat, roughly rectangular and approximately 1,100m^2 in area. It is bound by Stacey Street to the southwest, St Giles Passage to the northeast and New Compton Street to the northwest. The site is located in the London Borough of Camden and the approximate centre is at National Grid Reference 529977, 181149. The site location is show in Figure 1.

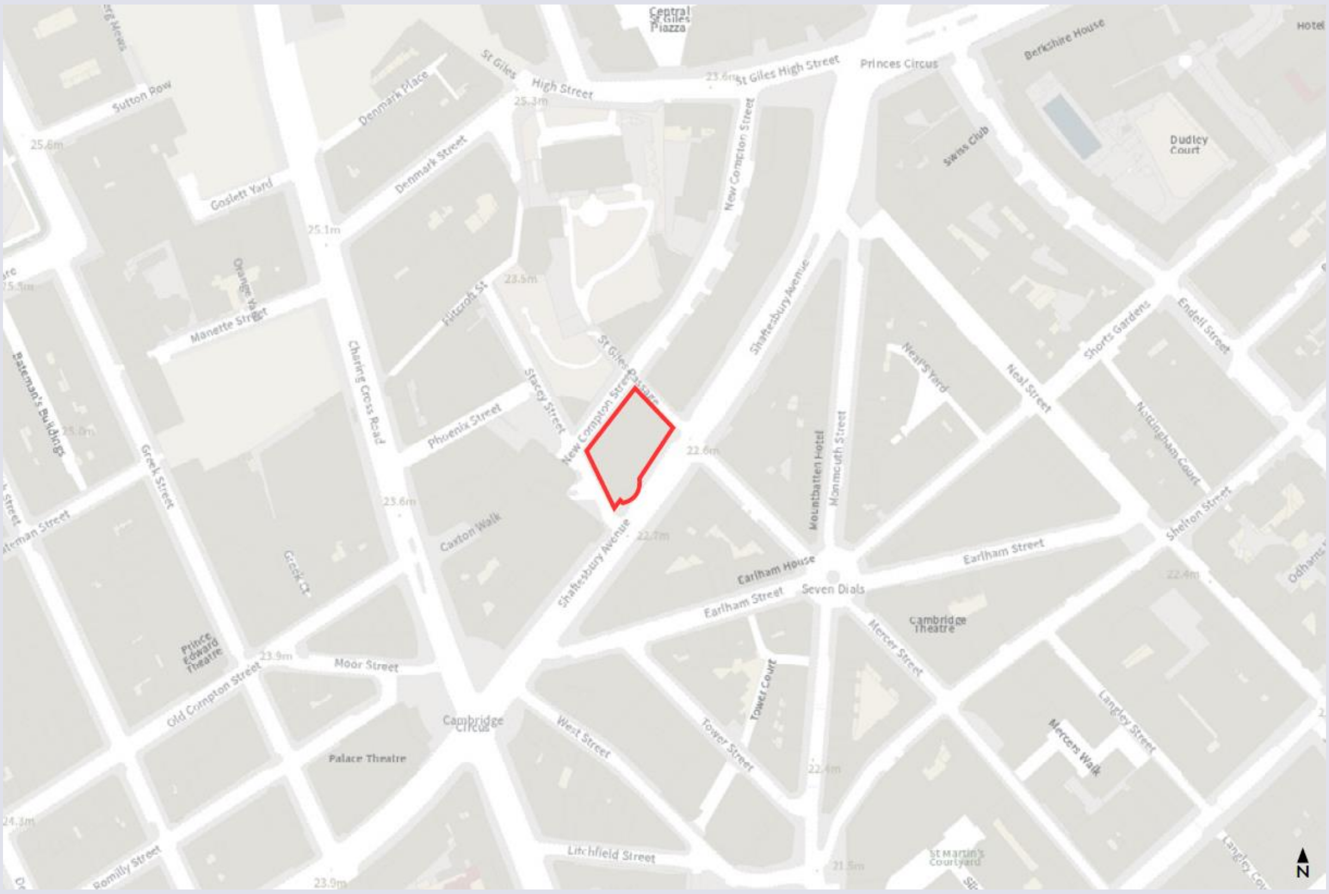


Figure 1: Site Location

1.3 Existing Structures

The site is occupied by a Grade II listed six to seven storey steel framed, masonry building with two levels of basement supported by reinforced concrete basement retaining walls. The ground level is used as a cinema with the upper floors used as office space and storage.

Figure 2: Existing Building Looking from the South (Left) and the North (Right)

Limited dimensional information on the existing structure is available. Historic structural drawings were reviewed and used to scale building and basement dimensions. Note that these existing structural dimensions are yet to be confirmed at the time of preparation of this report. The current two level basement is formed by L-shaped reinforced concrete retaining walls, which are shown to be propped by floor slabs at B1 and B2 level. The upper section has a height of approximately 3.5m from ground level to underside of the wall foundation and the lower section of the retaining wall is 4.2m from the underside of B1 to the underside of B2. For a section of the southern and eastern basement walls, the L-shaped wall steps out to form a vault designated as a “Property Store” as identified on Figure 3 below.

Based on the limited information available on the existing structure, the following is considered to be how the structure is designed to act.

- It is assumed the basement construction was formed in an open excavation and the reinforced concrete retaining walls constructed within that open excavation.
- It is assumed that once constructed, the outside ground to the retaining walls was backfilled with suitable engineering fill material.
- Based upon the general geometry of the basement and the drawn thicknesses of the retaining wall elements, it is unclear whether the retaining walls were designed to cantilever from their base to ground level.
- It is likely along the perimeter of the basement that the first basement level provides a degree of lateral propping to the retaining wall. As to whether that was assumed in the design of the external retaining walls it cannot be determined at this stage.

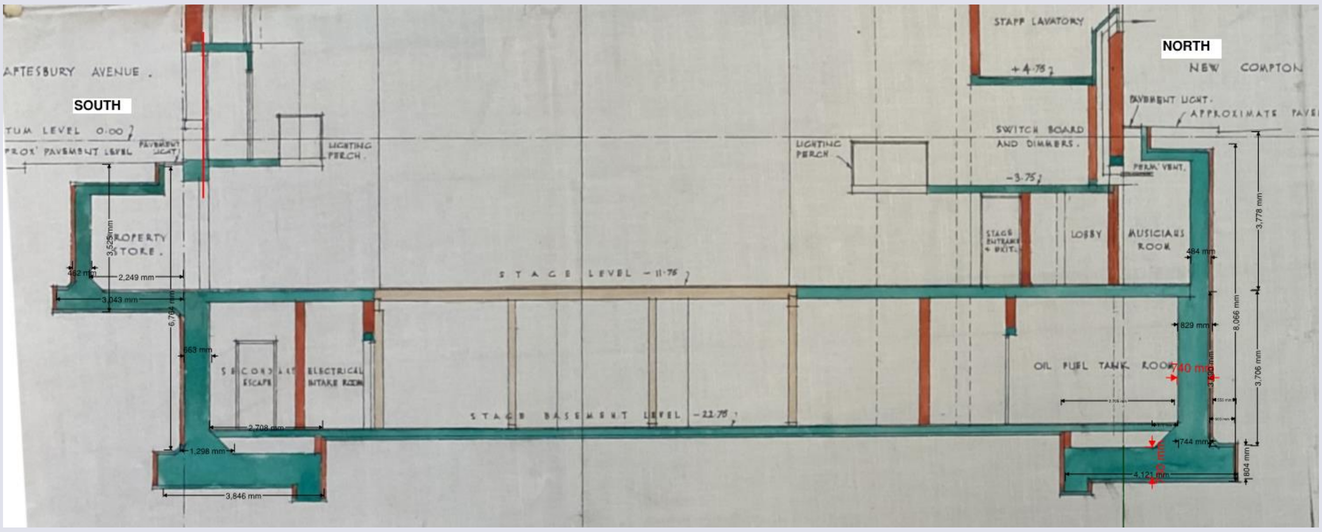


Figure 3: Existing Basement Cross Section

1.4 Proposed Works

The proposed works involve removing all of the internal structure of the existing building, leaving the façade and retaining walls to the two-storey basement in place, the northern façade will be removed and reinstated as part of the works. An additional two storeys of new basement will be formed within the existing basement footprint, extending below the existing basement retaining walls which shall remain in place. The additional basement depth is 8.8m from to the existing B2 slab level to the top of the B4 slab, which is anticipated to be approximately 1.0m-1.5m thick to support the structural loads, therefore the total basement depth will be up to 16.5m over the four levels. Within the footprint of the B4 slab, an additional 3.4m excavation is proposed for sprinkler water tanks.

The upper three levels of basement (B1 to B3) are proposed to be used as theatre space, with B4 used for shared plant. At ground level, theatre Front Of House will be provided with hotel rooms above on a number of levels, as yet unconfirmed.

It should be noted that the basement layouts are subject to revision and the layouts and dimensions presented here, and within this report are based on the available information at the time of writing. This report should be revised as required to accommodate the final basement layout. Basement construction details are presented in further detail in Section 3.

1.5 Sources of Information

The following reports were available in the preparation of this document.

Table 1: Available Information

Document Title	Date	Author	Reference
25916- 135-142 Shaftesbury Avenue, London, WC2 8AH Construction Method Statement and Basement Impact Assessment	Dec 2017	Price & Myers, GEA Ltd	25916, J17183
Existing Structure Archive Drawings	1929 to 1930	Various	-
Utility Report	July 2021	Envirocheck	LM / 97761
The Saville Theatre, 135 Shaftesbury Avenue, Basement Construction Appraisal	Dec 2022	Kier	-

2 Ground Conditions

2.1 Geological Mapping

The British Geological Survey (BGS) map of the area (Sheet 256) indicates the site is underlain by Lynch Hill Gravel overlying London Clay which forms the site bedrock.

The Lynch Hill Gravel is described as generally comprising "*sand and gravel, with lenses of silt, clay or peat"*.

The London Clay is described as "*homogenous, slightly calcareous silty clay to very silty clay, with some beds of clayey silt grading to silty fine-grained sand"*.

2.2 Previous Site Investigation

A single cable percussion borehole, BH1, was completed in 2017 by GEA Limited. The borehole was located adjacent to the site on New Compton Street and was drilled to a depth of 35.0m. In-situ testing comprised Standard Penetration Testing (SPT), and samples were taken for laboratory testing. Groundwater monitoring was undertaken following the site investigation.

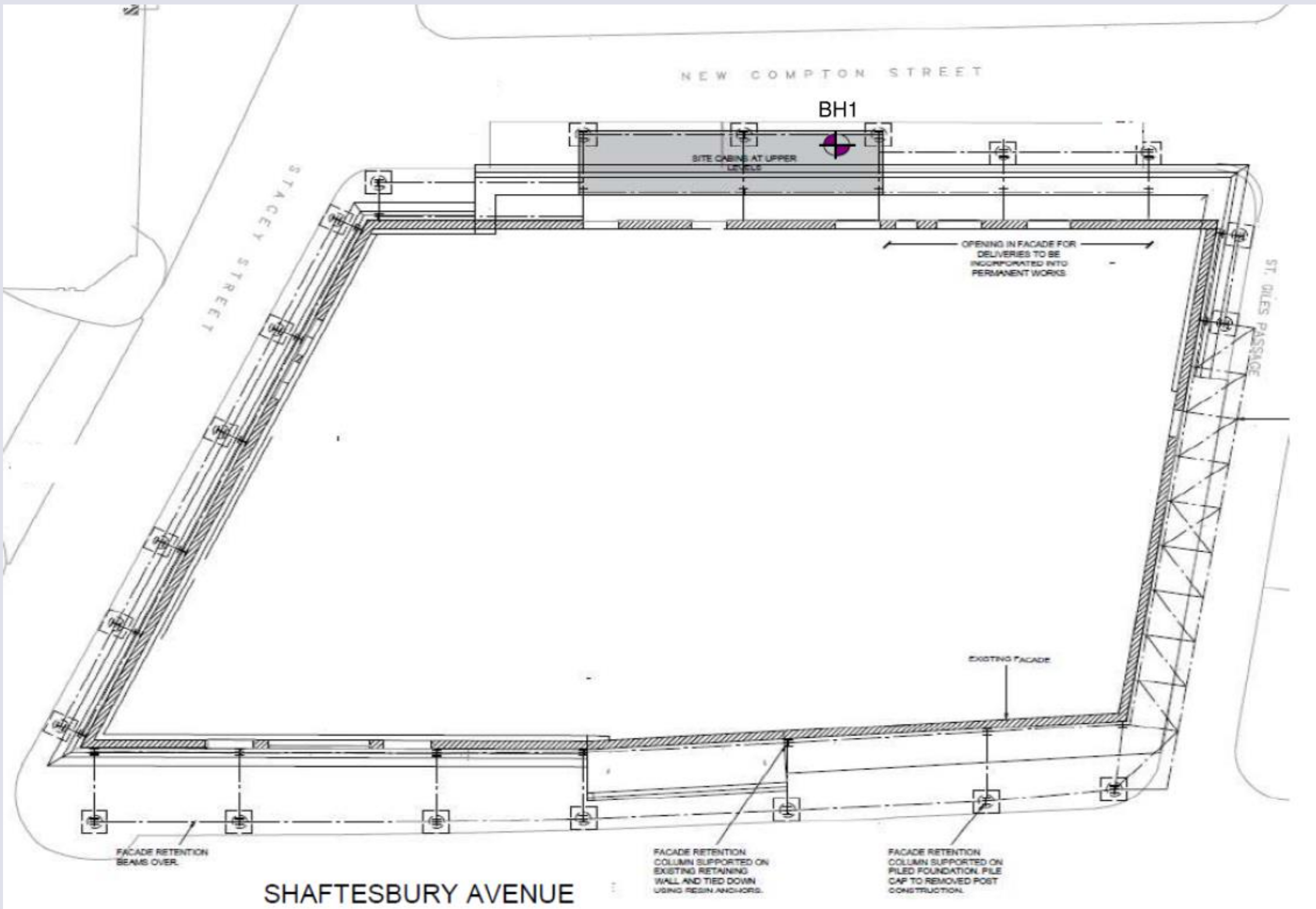


Figure 4: Borehole Location

2.2.1 Strata Encountered

The following strata was encountered within the GEA investigation.

- Made Ground comprising dark brown gravelly CLAY with brick was encountered to 3.5m bgl.
- Lynch Hill Gravel was found to comprise medium dense orange-brown gravelly SAND and was 1.2m in thickness.
- London Clay comprising firm becoming very stiff, dark brownish grey slightly sandy CLAY with occasional selenite crystals, claystones, shell fragments and pyrite nodules, was encountered from 4.7m bgl to 34.4m bgl.
- Lambeth Group soils described as very stiff slightly sandy CLAY were encountered from 34.4m bgl to the base of the hole at 35.0m bgl.

2.2.2 In-situ Testing

13 No. SPTs were undertaken in BH1, these are plotted in Figure 5 for all strata encountered. The SPTs were corrected for energy ratio, therefore the results presented are N_{60} values.

2.2.3 Laboratory Testing

4 No. Atterberg limit tests were undertaken on samples of London Clay, summarised below. An average Plasticity Index of 46% was calculated indicating it is a clay of medium to high plasticity.

12 No. water content tests were undertaken on samples of London Clay and 1 No. test was undertaken on Lambeth Group soils. An average water content of 26% was derived for the London Clay and the Lambeth Group test returned a result of 19%.

8 No. Undrained Unconsolidated (UU) triaxial tests were undertaken on samples of London Clay and 1 No. test was undertaken on a sample of a Lambeth Group soil. As the UU tests were undertaken on 100mm dia. samples, no reduction to the reported value was made to account for the effects of structure and fissuring.

The undrained shear strengths (S_U) from UU test results are plotted on Figure 6 with an S_U inferred from the SPT results using an f_1 factor of based on the plasticity index as recommended by Stroud (1974). The following f_1 factors were adopted:

- 5m bgl to 20m bgl: $f_1 = 4.5$, based on average plasticity index of 46%
- 20m bgl to 35m bgl: $f_1 = 6.0$, this value correlated better with the UU test results as outlined in White et. al (2019).

The resulting design line for the London Clay is defined below:

From 5m to 20m below ground level, $S_U = 50 + 6z$, where z is the depth below top of stratum at 4.7m bgl

From 20m to 35m below ground level, $S_U = 140 + 12z$, where z is the depth below 20m bgl

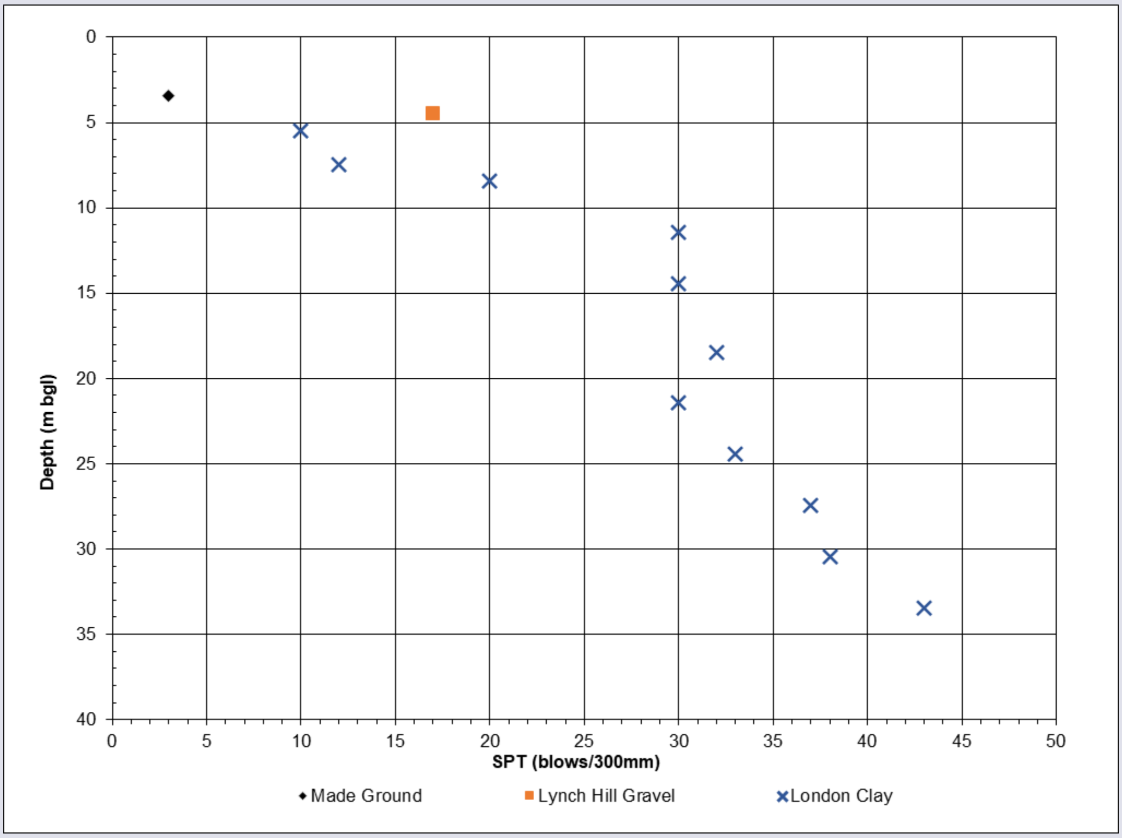


Figure 5: SPT Results (All Strata)

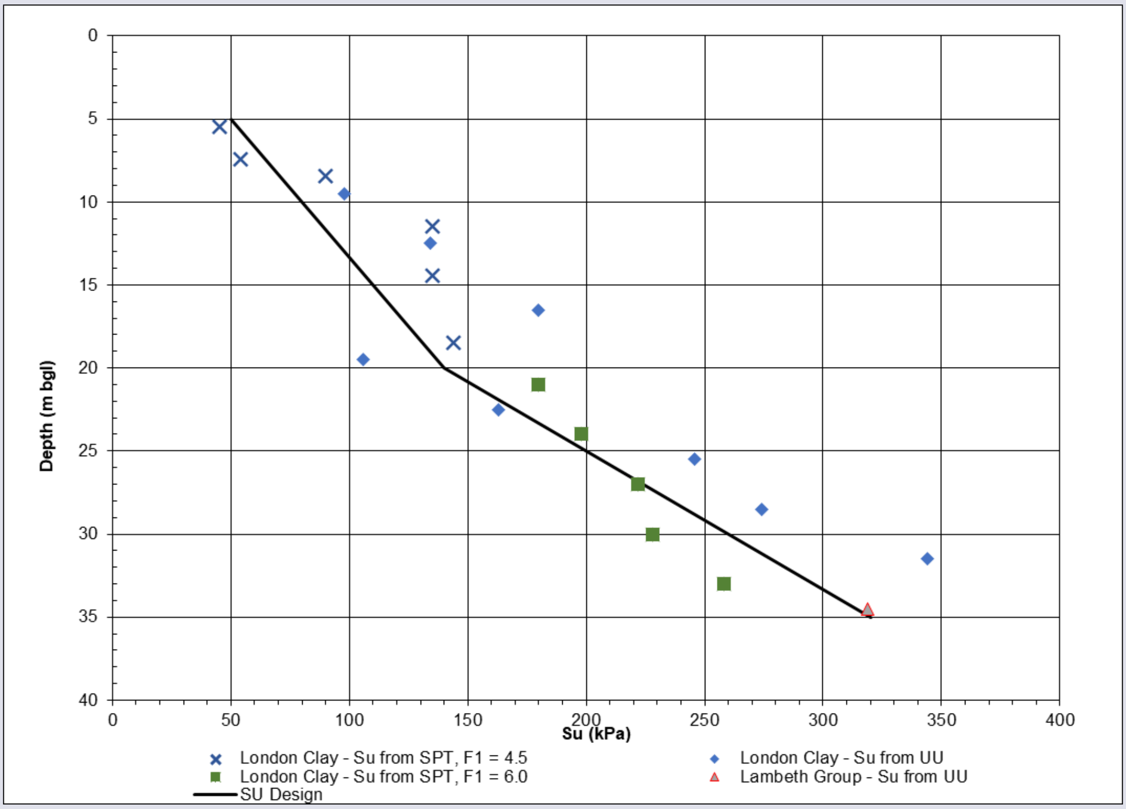


Figure 6: Undrained Shear Strength

2.3 Ground Model

BGS Borehole records were reviewed adjacent to the site to understand the deeper strata. Borehole TQ38SW5470 located 130m north of the site encountered the same sequence of strata, with chalk bedrock at 54.1m bgl (-33.88mOD).

Considering the site-specific ground investigation and nearby historical boreholes, the following Ground Model was adopted for design.

Table 2: Ground Model

Strata	Description	Top of Layer (m bgl)	Top of Layer (m OD)[1]	Thickness (m)
Made Ground	Gravelly CLAY with brick	0	122.0	3.5
Lynch Hill Gravel	Medium dense gravelly SAND	3.5	118.5	1.2
London Clay	Slightly sandy stiff fissured CLAY	4.7	117.3	29.7
Lambeth Group	Very stiff slightly sandy fissured CLAY	34.4	87.6	15.6
Thanet Sands	Very dense silty fine to medium SAND, locally clayey	50.0	72.0	4.1
Chalk	Unstructured Chalk (Grade D_M)	54.1	67.9	-
1: Estimated elevation of borehole				

2.4 Geotechnical Parameters

Geotechnical parameters have been derived for each of the strata, based on the available in-situ testing, parameters recommended from the previous Ground Investigation Report (GIR) from GEA and experience of similar materials. The geotechnical parameters presented in the following sections are applicable to a simple linear elastic idealisation of stiffness where the stiffness is derived as a function of the soil strength. This material model is appropriate for preliminary ground movement assessment and is routinely used in the design of retaining structures and consequently for assessing the performance of the existing basement walls. However, it should be noted that the use of a linear elastic model for ground movement assessment is only applicable for a preliminary assessment since the results are known to be conservative. Consequently, in later sections a more advanced constitutive soil model is derived for use in the detailed GMA incorporating strain-dependent stiffness relationships for the London Clay and Lambeth Group strata.

2.4.1 Made Ground

One SPT was undertaken in Made Ground returning a value of 3 blows per 300mm. Historic boreholes in the area were also reviewed and the SPT values ranged from 3 to 6 blows per 300mm, therefore a design value of N = 4 blows per 300mm was adopted for design.

N = 4 blows per 300mm

An undrained shear strength can be derived assuming a high plasticity for the soil and a correlation factor of 4.5 based on Stroud (1974). This results in an S_U of 18~20kPa.

An undrained stiffness was derived using the following correlation as recommended in Tomlinson (2001):

$$\textit{Undrained Stiffness, } E_u = 450 * S_u$$

$$\textit{Undrained Stiffness, } E_u = 8.1 \text{ MPa}$$

A drained stiffness was taken as 80% of the undrained stiffness.

The 2017 Price and Myers Basement Impact Assessment report recommends an effective cohesion of 0kPa, and a friction angle of 25˚ for design for the Made Ground, which was considered appropriate.

2.4.2 Lynch Hill Gravel

The Lynch Hill Gravel was described as a medium dense gravelly sand. One SPT was undertaken at 4.2m bgl, returning a value of 16 blows per 300mm. Correcting for overburden with a C_N factor of 1.1, a design $(N_1)_{60}$ value of 18 blows per 300mm is derived.

Using the correlation of SPT N to angle of friction proposed by Mitchell et al. (1978), the design SPT correlates to an angle of friction of 33 degrees.

The stiffness of the Lynch Hill Gravel has been derived using the following relationship between uncorrected SPT "N" value and stiffness (Stroud, 1989): $E' / N = 1 - 2$ MN/m^2.

Based on the design value for SPT N the following stiffness value is recommended for design purposes and is generally consistent with published values for normally consolidated granular soils in Section 8.1.2 of CIRIA C143.

Stiffness, E' = 24 MPa

2.4.3 London Clay

London Clay was described as firm becoming very stiff to high strength with depth.

The following undrained shear strength profile was adopted for design:

From 5m to 20m below ground level, $S_U = 50 + 6z$, where z is the depth below top of stratum at 4.7m bgl

From 20m to 35m below ground level, $S_U = 140 + 12z$, where z is the depth below 20m bgl

The stiffness of the London Clay has been calculated using the following conventional correlations to undrained shear strength:

- Vertical Undrained Stiffness:

$E_u = 450\ S_u$

$E_u = 22.5 + 2.7\ z$ MPa, where z is the depth below top of stratum at 4.7m bgl

$E_u = 63.0 + 5.4\ z$ MPa, where z is the depth below 20m bgl

- Horizontal Undrained Stiffness:

$E_u = 1000\ S_u$

$E_u = 50 + 6\ z$ MPa, where z is the depth below top of stratum at 4.7m bgl

$E_u = 140 + 12\ z$ MPa, where z is the depth below 20m bgl

- Drained Stiffness:

$$E' = 0.8\ E_u$$

Effective strength parameters for the London Clay were derived considering previous experience with the London Clay, the in-situ testing and correlations to friction angles for cohesive soils as per BS 8002:2015. The design friction angle recommended as per below:

$$\emptyset' = 25°$$

Effective cohesion was considered as a function of depth within the London Clay as follows:

$$c' = 5\ kPa$$

2.4.4 Lambeth Group

The Lambeth Group was described as very stiff to extremely high strength silty, sandy clay. No SPT values were available within the strata. One UU triaxial test was undertaken at 34.5m bgl returning an undrained shear strength of 319kPa.

Considering the material description within the logs and the anticipated strength profile with depth, the following undrained shear strength is considered appropriate for design:

$$S_U = 320\ kPa$$

The stiffness of the Lambeth Group beneath the proposed excavation will be largely dependent on the strain levels which are anticipated to be low considering the top of the Lambeth Group will be at 10m below the base of any excavation. Therefore the following correlation to undrained shear strength has been adopted based on Tomlinson (2001):

$$E_u = 1{,}000\ S_u$$

$$E_u = 320\ MPa$$

2.4.5 Summary of Geotechnical Parameters

Table 3: Geotechnical Parameters

Strata	γ_{bulk} (kN/m^3)	c' (kPa)	Φ (˚)	S_U (kPa)	E_U (MPa)	E' (MPa)
Made Ground	17	0	25	20	8.1	6.5
Lynch Hill Gravel	19	0	33	-	-	24
London Clay	19	5	10	$50+6z_1$ $140 + 12z_2$	V:$22.5 + 2.7z_1$ H:$50.0 + 6.0z_1$ V:$63.0 + 5.4z_2$ H:$140 + 12.0z_2$	V:$18.0 + 2.2z_1$ H:$40.0 + 4.8z_1$ V:$50.4 + 4.3z_2$ H:$112 + 9.6z_2$
Lambeth Group	20	10	30	320	320	260
Thanet Sands	22	0	42	-	-	320
Chalk	-	-	-	-	-	-

1 Depth below 4.7m bgl
2 Depth below 20.0m bgl

2.5 Groundwater

Groundwater wasn't encountered during the GEA investigation during drilling. Groundwater monitoring was undertaken within two installations within borehole BH1, described as:

- *19mm piezometer was installed to 20m with a response zone at 19.21m*
- *50mm diameter standpipe installed to 5m depth*

The available monitoring results are summarised in Table 4 below.

Table 4: Groundwater Monitoring Results

Hole	Top of Hole Elevation	Installation	Date	Screen Depth (m bgl)	Water Depth (m bgl)	Screened Stratum
BH1	-	19mm Piezometer	06/11/2017	19.21m	5.25m	London Clay
			30/11/2017		5.28m	
BH1	-	50mm Standpipe	06/11/2017	0.0m to 5.0m	Inaccessible	Made Ground Lynch Hill Gravel London Clay
			30/11/2017		4.39m	

Generally in London, the groundwater regime is characterised by the presence of an upper aquifer in the surficial soils and gravels, perched on top of the very low permeability London Clay that acts as an aquiclude. The Lambeth group comprises a mixture of cohesive and granular layers and therefore can act as both an aquiclude and part of a lower aquifer hydraulically connected to the Thanet Sand and Chalk strata.

The groundwater monitoring results recorded groundwater levels at the upper boundary of the London Clay, which is interpreted to represent the presence of perched surficial water. It should be noted that monitoring was only undertaken on two return visits and that additional monitoring including piezometers within the lower aquifer should be considered if the development foundations will extend to this depth, to confirm the groundwater regime.

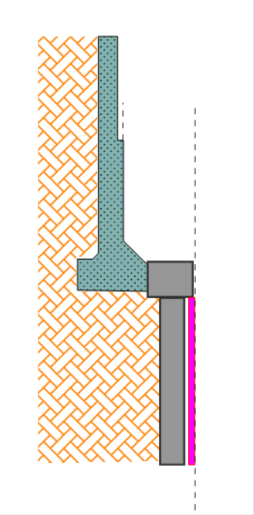
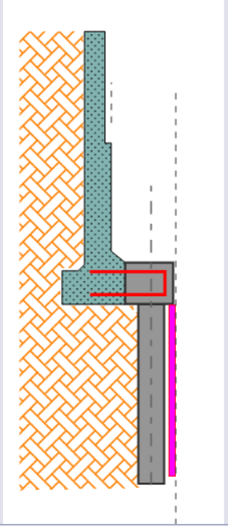
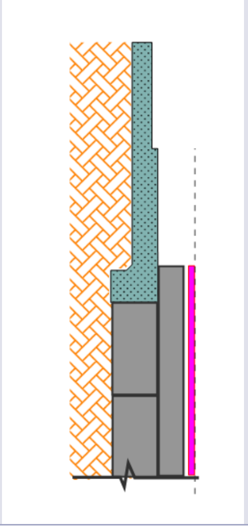
3 Proposed Development

As discussed in Section 1.4, the proposed development plans are under development and the following sections reflect the current proposals. Future revisions of this assessment may be required where changes to these proposals are made.

3.1 Basement Retention

This Ground Movement Assessment has considered the three options for the proposed basement deepening as outlined in Pell Frischmann Sketch: Basement Structural Plans dated November 2022, reproduced below in Table 5.

Table 5: Basement Construction Options

Option	Option 01	Option 02	Option 03
Description	Separate embedded retaining wall (secant or contig) drilled offset from the base of the existing basement wall.	Hybrid embedded retaining wall (secant or contig) drilled offset from the base of the existing wall and dowelled into it.	Mass concrete underpinning with lining wall to underside of toe of retaining wall with internal RC lining wall.
Sketch			

3.1.1 Piled Wall Option

For the piling option, Options 01 and 02, it is understood that to meet the basement width requirements, 600mm diameter piles are the largest diameters available, with a 300mm structural lining wall cast in front. From a GMA perspective, provided similar stiffness of the embedded secant retaining wall are maintained during excavation, then similar induced ground movements are likely to occur for both Option 01 and Option 02.

3.1.2 Underpinning Option

The underpinning Option 3 assessment has considered sequencing of the works and dimensions of the pins in general accordance with Kier's Basement Construction Appraisal for Saville Theatre, dated December 2022.

This proposed underpinning is made by forming pins 1.8m in width around the basement footprint in a 1 in 5 hit and miss sequence, resulting in 70 No. separate underpins per structural level. The pins are proposed to be 2.6m in height and 2.0m in thickness. The pins are to be constructed in three phases, where Phase 1 forms the corner sections, Phase 2 forms the intermediate sections and Phase 3 forms the largest span length across the basement wall, refer Figure 7 (right), where Phase 1 is shown in yellow, Phase 2 is shown in blue and Phase 3 in purple.

Three levels of temporary propping are proposed below the existing basement, with temporary props also being installed to support the existing basement on demolition of the floor slabs, refer to Figure 7.

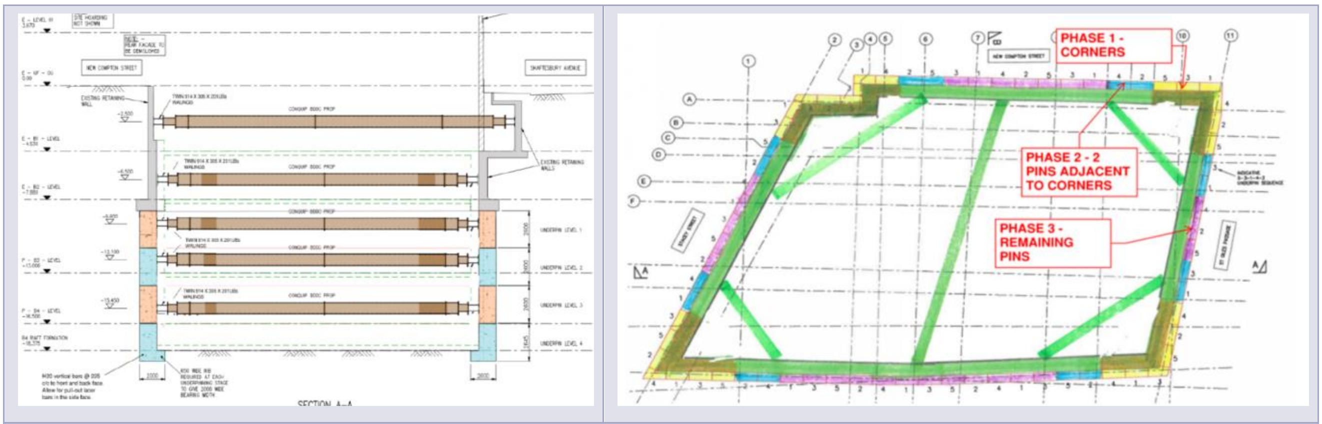


Figure 7: Underpinning Arrangement (Kier Basement Appraisal)

3.2 Floor Slab Layout

The proposed floor slab layout at the time of preparation of this report is as follows.

3.2.1 Ground Floor Slab

A permanent ground floor slab across the whole basement footprint is proposed. No significant slab openings are anticipated. Structural cores are proposed in the north-east, west and south-east corners of the slab. The slab is assumed to prop the top of the existing reinforced concrete basement walls. At this stage a 350mm thick slab has been adopted.

3.2.2 Basement 01 (B1) Floor Slab

A partial floor slab is proposed to accommodate the theatre auditorium and viewing areas at both B1 and B2 level. This slab is proposed to be located at 3.9m below ground level (below top of existing basement walls). The layout of the slab is as shown in Figure 8. At this level all sections of the basement are propped except for the eastern façade adjacent to St Giles Passage which spans unsupported approx. 14.5m between the structural cores. In the absence of a developed scheme a 350mm thick slab has been assumed.

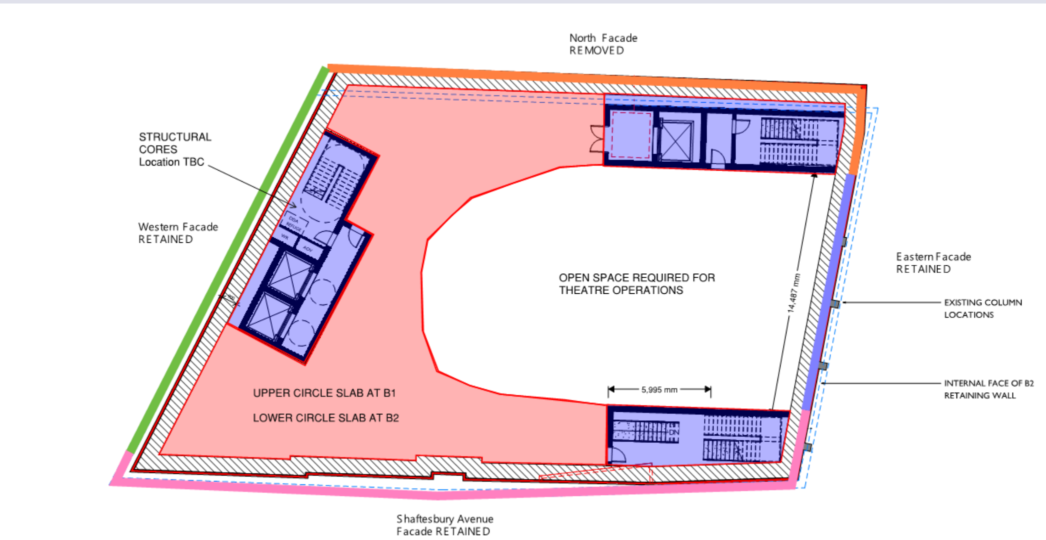


Figure 8: Slab Layout at B1 and B2 Level

3.2.3 Basement 02 (B2) Floor Slab

The partial floor slab at B2 will follow a similar outline to the B1 floor slab, located at a level of 7.3m below ground level. In the absence of a developed scheme a 350mm thick slab has been assumed to the B2 floor plate..

3.2.4 Basement 03 (B3) Floor Slab

At B3 level (top of slab at approx. 13.0m below ground level) a floor slab will be present, which extend across the entire footprint except for a slab opening to accommodate removable flooring for the staging. The floor opening dimensions are not known at this stage so an assumed outline of 7.5m by 12.0m has been adopted, with a 1.0m wide thickened section of slab (1.0m thick(placed in front of the basement wall forming a deep horizontal beam to provide support to the wall, refer Figure 9. At this stage a 350mm thick slab has been adopted.

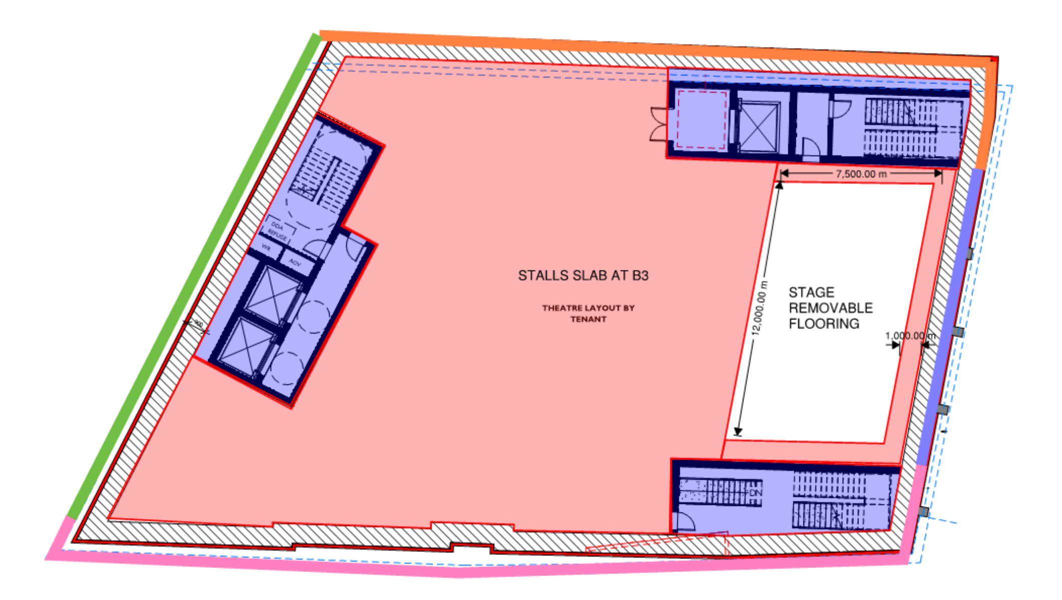


Figure 9: Slab Layout at B3 Level

3.2.5 Basement 04 (B4) Floor Slab

The basement slab at B4 level will be ground bearing and will support the structural loads via both internal columns and the perimeter basement walls. The slab will be integral with the basement walls thereby providing a flexural fixity and restraint both horizontally and vertically. The top of the slab will be located at 16.50m below ground level, and is assumed to be between 1.0m and 1.5m thick at this stage, therefore a raft foundation formation level of 18.30m below ground level was adopted to accommodate up to 1.8m of slab excavation.

The extent of the B4 slab is as shown in Figure 10.

3.2.6 Sprinkler Tank Slab

Further excavation within the B4 footprint for the proposed sprinkler tanks will extend another 3.4m. This excavation is assumed to be 6.0m from the edge of the basement wall as per plan in Figure 10. At this stage of the design it has been assumed that the tank walls will be formed from a 350mm thick concrete wall, with a 350mm thick slab at the base of the tank. The slab design will need to consider the effects of heave and strip foundations to balance the heave and bearing pressures or a suspended slab may be required.

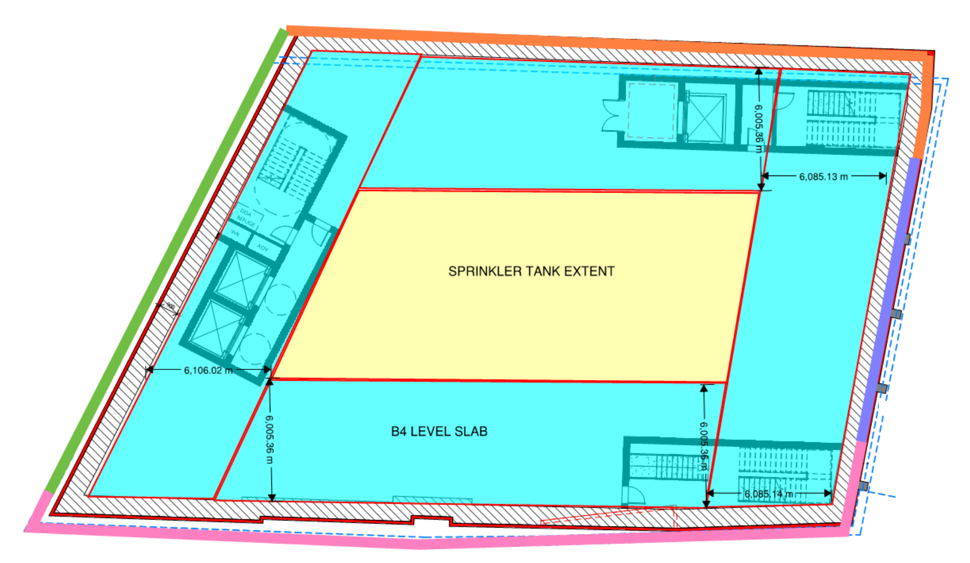


Figure 10: Slab Layout at B4 Level

3.2.7 Floor Slab Layout Summary

A summary of the proposed layout is shown below in Figure 10.

This also shows temporary propping levels that were adopted for preliminary design. These will need to be confirmed with input from the Contractor pending their design.

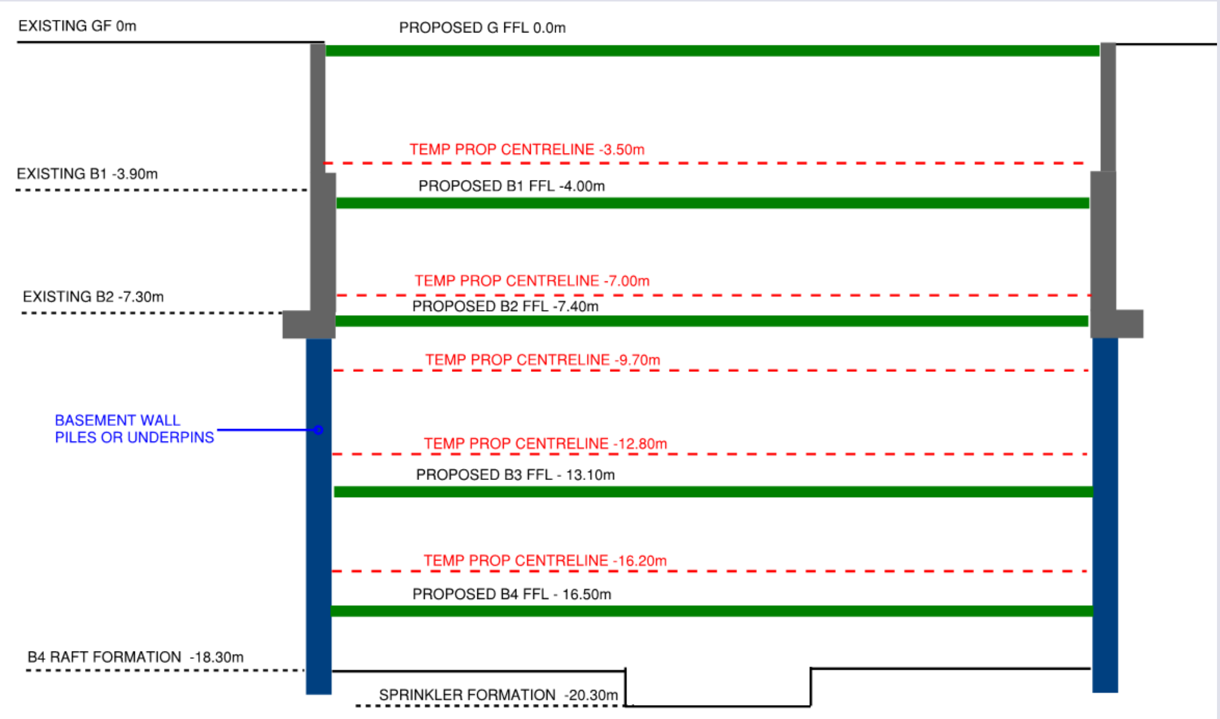


Figure 11: Proposed Layout of Floor Slabs and Temporary Props

3.3 Construction Sequence

The current proposed construction sequence at the time of preparation of this report is summarised below.

Piled Wall Option:

1. Demolition of existing internal structure.
2. Installation of façade supports to existing retaining wall stem (in reality parallel with 1 above).
3. Installation of walers and temporary props between Ground and B1 levels.
4. Demolition of existing B1 slab.
5. Installation of walers and temporary props between B1 and B2 levels.
6. Demolition of existing B2 slab and toe of existing retaining wall toe cut back to 400/600mm from internal face of retaining wall.
7. Piling works from B2 level and construction of capping beam at existing wall toe.
8. Excavation works to level of B3 temporary props.
9. Installation of walers and temporary props.
10. Excavation works to B4 temporary prop level.
11. Installation of walers and temporary props.
12. Excavation to B4 slab formation level and construction of B4 raft slab dowelled into piled wall.
13. Removal of B4 level walers and temporary props.
14. Construct lining wall between B4 and B3 levels.
15. Construction of B3 slab dowelled into piled wall.
16. Construction of B2 slab dowelled into piled wall capping beam.
17. Removal of B3 level walers and temporary props.
18. Construct lining wall between B3 and B2 levels.
19. Construct B1 slab dowelled into existing basement walls.
20. Removal of temporary props at B1 and B2 level.
21. Complete super structure.

3.3 Construction Sequence (cont'd)

Underpinning Option:

1. Demolition of existing internal structure.
2. Installation of façade supports to existing retaining wall stem (in reality parallel with 1 above).
3. Installation of walers and temporary props between Ground and B1 levels.
4. Demolition of existing B1 slab.
5. Installation of walers and temporary props between B1 and B2 levels.
6. Demolition of existing B2 slab and toe of existing retaining wall toe cut back to 400/600mm from internal face of retaining wall.
7. Excavation sequence of the Phase 1 (corner sections) underpins in a 1 to 5 hit and miss sequence:
    - Excavate and install trench box
    - Excavate underpin beneath existing reinforced concrete retaining wall
    - Install reinforcement and shuttering for the underpin
    - Remove formwork and remove trench box
    - Backfill trench box excavation with lean mix concrete
8. Install Phase 2 (section surrounding corner) underpins
9. Install Phase 3 (long span sections)
10. Once all underpinning installed, excavate to level of temporary props
11. Install temporary propping and walers
12. Excavate to next underpinning level and install 2nd level underpins, Phase 1 to Phase 3
13. Excavate to next level temporary props and install temporary propping and walers
14. Excavate to next underpinning level and install 3rd level underpins, Phase 1 to Phase 3
15. Excavate to next level temporary props and install temporary propping and walers
16. Excavate to next underpinning level and install 4th level underpins, Phase 1 to Phase 3
17. Excavate to next level temporary props and install temporary propping and walers
18. Excavation to B4 slab formation level
19. Cast permanent lining wall and construct B4 raft slab dowelled into wall
20. Removal of B4 level walers and temporary props.
21. Construct lining wall between B4 and B3 levels.
22. Construction of B3 slab dowelled into lining wall.
23. Construction of B2 slab dowelled into lining wall.
24. Removal of B3 level walers and temporary props.
25. Construct lining wall between B3 and B2 levels.
26. Construct B1 slab dowelled into existing basement walls.
27. Removal of temporary props at B1 and B2 level.
28. Complete super structure.

4 Adjacent Structures

Adjacent structures that could potentially be affected by the proposed works are described below and presented in the plan view in Figure 13.

4.1.1 Existing Façade

The existing Saville Theatre perimeter will be retained as part of the proposed works on the western, eastern and southern sides. The northern façade will be removed and reinstated as part of the works.

4.1.2 Adjacent Buildings

Limited information regarding the foundation systems of adjacent structures is available. Therefore in the absence of confirmation, they have conservatively been assumed to be founded on shallow foundations with no basement present for smaller structures. A more accurate assessment of the foundation impacts can be achieved once further information regarding their foundation type (deep or shallow) and founding depth is confirmed. Information regarding adjacent structures is presented below in Table 6, the heights of neighbouring buildings have been estimated from observation and where the depths of foundations or the heights of buildings are not known due to restricted access, these dimensions have been estimated.

Table 6: Adjacent Structures

Ref	Site Name	Distance from Site (m)[1]	Max. Est. Height Above Ground Level[2]	Depth of Basement BGL	Foundation System Type
A	Shaftesbury House 151 Shaftesbury Ave	5.0	28.0m	4.0m[3]	Shallow Foundations[3]
B	Pendrall House 1 & 2 St Giles Passage	10.0	17.5m	Unknown	Unknown
C	Phoenix Garden	11.0	3.5m	Unknown	Shallow Foundation Likely
D	121 to 125 Shaftesbury Ave	6.0	38.5m	4.0m[3]	Shallow Foundations[3]
E	158 Shaftesbury Ave	17.0	15.0m	Unknown	Unknown
F	164 Shaftesbury Avenue	17.0	28.0m	Unknown	Unknown
G	Chinese Church in Soho	17.0	15.0m	Unknown	Unknown
H	170 Shaftesbury Avenue	24.0	15.0m	Unknown	Unknown
I	Crossrail Tunnel	25.0	-	Crown of tunnel at 17.5m bgl	7.0m tunnel diameter

1: Measured as the shortest distance from the edge of the Site to the closest edge of the adjacent structure
2: Estimated based on number of floors visible from street level
3: As per Kier Basement Appraisal

4.1.3 Crossrail Tunnel

A Crossrail tunnel is present approx. 25m from the north eastern corner of the site to the centreline of the tunnel. The tunnel crown is at 17.5 m below ground level and the tunnel has a diameter of 7.0 m.

The site is located outside the extents of the Crossrail safeguarding limits therefore any piling works do not require consultation with Crossrail.

4.1.4 Utilities and Services

Service plans obtained from Envirocheck in July 2021 were reviewed as part of this check, and the main utilities adjacent to the site are shown below in Figure 12, this does not include all smaller services affected. The ground movements predicted from this assessment can be used to produce a specific impact assessment for each service to form an agreement to safeguard these assets during the works.

- Cadent Gas (top left on Figure 12) is shown along New Compton Street and Shaftesbury Avenue as a Low Pressure gas mains.
- GTT fibre optics (top right on Figure 12) are shown running along Shaftesbury Avenue and Stacey Street.
- Thames Water mains (bottom right on Figure 12) are shown as a 125mm diameter HPPE ducts along New Compton St, St Giles Passage and Shaftesbury Avenue. A 6 inch and 12 inch CI (Cast Iron) pipe are also shown in the centre line of Shaftesbury Avenue. Based on the diameters the pipes are likely to be within 900mm of the ground surface.
- Thames Water foul water sewer (bottom left on Figure 12) are shown along all surrounding streets, no indication is given to the type of pipe material or depth.

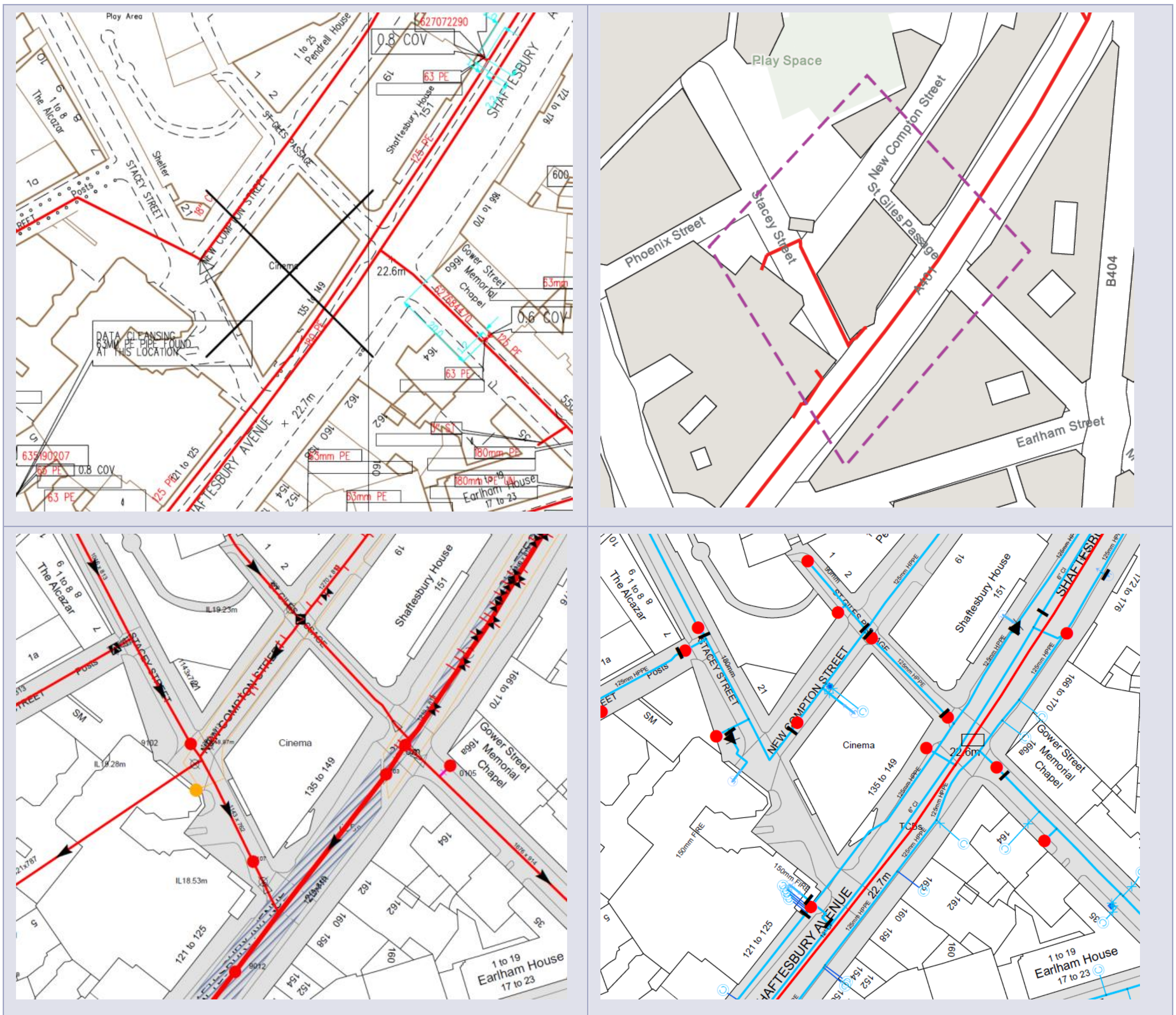


Figure 12: Utilities and Services

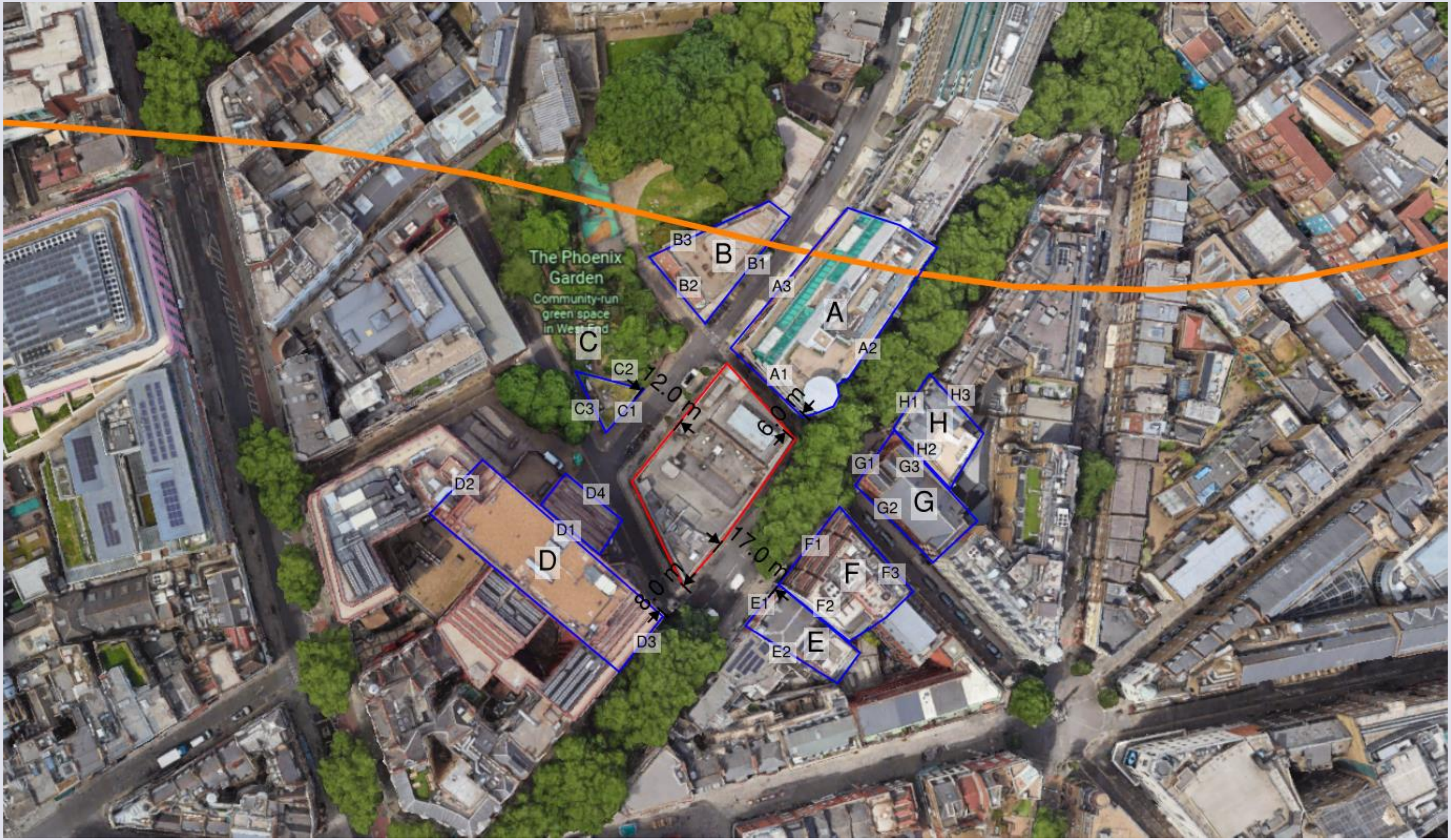


Figure 13: Plan of Adjacent Structures and Facades (Crossrail shown in Orange)

5 Ground Movement Assessment

Ground movements occur as a result of construction activities undertaken in the formation of basements. In order to estimate the extent and magnitude of the ground movements associated with installation and excavation, guidance from the CIRIA C760 guide *Embedded Retaining Walls – Guidance For Economic Design* is used. The guidance in this document is based on case studies of walls embedded in stiff over consolidated clays which typically have near surface deposits such as River Terrace Deposits and Made Ground. Many of the case studies reported relate to excavations in London Clay.

5.1 Ground Movement due to Installation

The CIRIA 760 guide provides some empirical guidance on ground movements associated with bored pile installation but does not present any information on embedded retaining wall construction using a sequential underpinning methodology. It is noted that recent studies reported by Ball et. al. (2014) demonstrates that for piles installed to current standards and workmanship and quality, horizontal ground movements are negligible and maximum vertical movements (which occur at the pile wall) are about 0.025% of the depth of the wall, with negligible movement occurring about 1.5 times the depth of the wall. For the piled option the maximum length of the piles is approx. 25m which would result in maximum vertical ground movements at the wall location of about 6mm and negligible movement at a distance of about 37m from the wall.

As already noted, there is no guidance relating to the prediction of ground movements associated with wall installation based on underpinning techniques since in this case the wall installation and excavation take place sequentially rather than the wall installation preceding the excavation phase.

5.2 Ground Movement due to Excavation

Ground movements associated with the excavation process consist of (i) movements induced by the deflection of the embedded retaining wall and the subsequent settlement of the retained soil, and (ii) movements associated with the unloading of the ground due to excavation. In this latter case the unloading of the ground induces instantaneous elastic unloading which may induce some nominal upward displacement of the ground at the base of the excavation, and as the soil is assumed to behave as an elastic solid, there will be some theoretical settlement of the soil adjacent to the excavation, although this movement associated with the elastic unloading is generally considered to be negligible.

Following this initial elastic phase the soil will undergo heave movements associated with volumetric expansion of the soil as the water and soil come to a new state of equilibrium with the reduced vertical loads. The 'heave' is confined to the base of the excavation where the reduction in vertical stress has occurred and would not contribute to ground movements beyond the footprint to the excavation.

The CIRIA C760 guide presents several methods for estimating excavation induced ground movements, namely empirical relationships based on excavation depth, relating vertical settlement behind the wall to the deflected shape of the wall or undertaking a detail numerical analysis. It is noted that neither of the first two approaches are suitable to the underpinning option and consequently in order to predict ground movements a numerical analysis is required.

6 Plaxis 3D Geotechnical Assessment

A geotechnical model of the existing and proposed basement was modelled in the Three-Dimensional Finite Element (3D FE) geotechnical software package PLAXIS 3D. This software is used worldwide to analyse a range of geotechnical soil-structure interaction problems such as excavations and foundations.

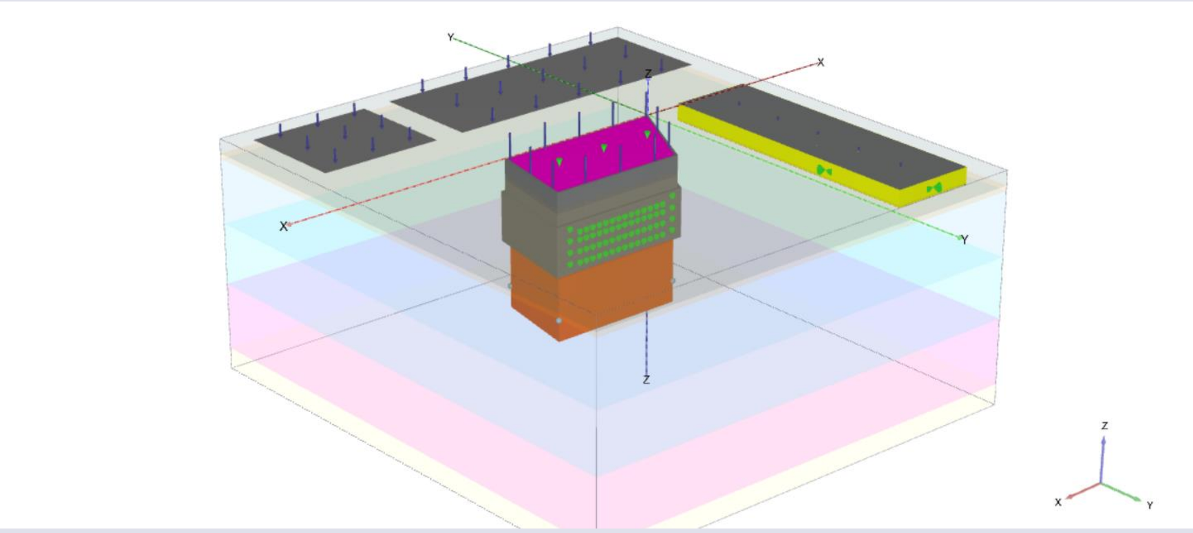


Figure 14: View of Plaxis 3D Finite Element Model

6.1 Ground Model

The ground model adopted was as per Table 2. Non-linear soil models were adopted to more accurately capture the soil response to excavation unloading, when compared to a linear elastic-perfectly plastic Mohr Coulomb (M-C) model. The model used is the hardening soil with small strain stiffness (HSS) model (Schantz et al. 1999). The model incorporates three input stiffness parameters: the triaxial loading stiffness, E_{50}, the triaxial unloading-reloading stiffness, E_{ur}, and the oedometer loading modulus, E_{oed}, as presented in Figure 15 (left).

The Hardening Soil model assumes elastic material behaviour during unloading and reloading. However, the strain range in which soils can be considered truly elastic, i.e. where they recover from applied straining almost completely, is very small. With increasing strain amplitude, soil stiffness decays nonlinearly according to a hyperbolic law. Figure 15 (right) gives an example of such a stiffness reduction curve with outline of the characteristic shear strains that can be measured near geotechnical structures.

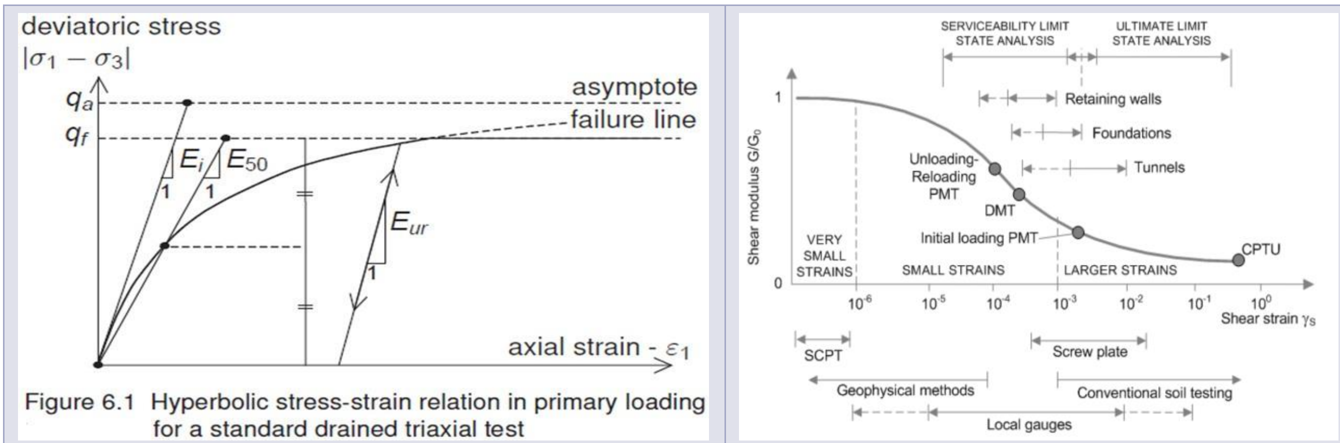


Figure 15: Small Strain Stiffness Models

For the near surface soils (Made Ground and Lynch Hill Gravel) and deeper strata (Thanet Sands), Mohr-Coulomb linear elastic soil models were adopted. Parameters adopted in the Plaxis 3D model are presented in Table 3, note that a small value of cohesion was added to the near surface soils (Made Ground and Lynch Hill Gravel) to prevent the unrealistic shallow failures occurring.

Table 7: Plaxis 3D Geotechnical Parameters

Strata	Depth to Top (m bgl)	Drainage	Material Type	Unit Weight (kN/m^3)	Cohesion (kPa)	Critical State Friction Angle, φ (deg)	Dilatancy Friction Angle, ψ (deg)	K0	Stiffness, E (MPa)
Made Ground	0.0	Undrained	Mohr-Coulomb Undrained B	17	25	0	0	0.577	10
Made Ground	0.0	Drained	Mohr-Coulomb Drained	17	2	25	0	0.577	8
Lynch Hill Gravel	3.5	Drained	Mohr-Coulomb Drained	18	2	33	0	0.455	24
London Clay	4.7	Undrained	HS Small Undrained B	20	45 + 9z	0	0	Refer Table 8	Refer Table 8
London Clay	4.7	Drained	HS Small Drained	20	5	25	5	Refer Table 8	Refer Table 8
Lambeth Group	34.4	Undrained	HS Small Undrained B	20	320	0	0	Refer Table 8	Refer Table 8
Lambeth Group	34.4	Drained	HS Small Drained	20	10	30	5	Refer Table 8	Refer Table 8
Thanet Sands	50.0	Drained	Mohr-Coulomb Drained	22	0	42	2	0.331	300
Chalk	54.0	Taken as the boundary of the model, therefore incompressible at depth.							

Parameters adopted for the HSS model are presented below in Table 8. In the absence of site specific triaxial testing these values were derived from published data and previous experience with the use of the HSS model. Since a reference pressure for stiffness correlations is not commonly given, a reference pressure of 100 kPa was adopted as broadly, many triaxial tests can be assumed to be performed with 100 kPa confining pressures.

Table 8: Hardening Soil Small Strain Stiffness Model Parameters

Soil and Depth Range	m bgl	London Clay 4.7 to 20	London Clay 20 to 34.4	Lambeth Group 34.4 to 50
Coefficient of earth pressure at rest[1]	K0	2.0[1]	1.5[1]	1.25[1]
Secant stiffness in standard drained triaxial test	E_{50}^{ref} MPa	10.5	12.2	13.5
Tangent stiffness for primary oedometer loading	E_{OED}^{ref} MPa	7.4	8.5	9.5
Unloading/reloading stiffness	E_{UR}^{ref} MPa	31.6	36.5	40.6
Power for stress-level dependency of stiffness	m	0.84[2]	0.84[2]	0.84[2]
Shear modulus at very small strain	G_0 MPa	79.1[3]	79.1[3]	79.1[3]
Shear strain at which Gs/ G0 = 0.722	$\gamma_{0.7}$	0.00035	0.00035	0.00037
Poisson's ratio for unloading/reloading	ν_{ur}	0.2	0.2	0.2
Reference confining pressure	P_{ref} kPa	100	100	100
Failure ratio, qf / qa	R_f	0.9	0.9	0.9

1: Hight et al. (2003)
2: Viggiani et al. [1997] and Hicher [1996]
3: Based on shear wave velocity = 200m/s for very small strains

6.2 Groundwater

As discussed in Section 2.5, the London Clay strata is considered an aquiclude and therefore groundwater was not modelled as being present within this layer. Groundwater was modelled as being present within the Lambeth Group soils at 36m bgl.

6.3 Existing Basement

The existing two storey basement walls comprise reinforced concrete structures with a small heel, and large toe that is propped at ground floor, B1 and B2 level. Figure 16 shows how the wall geometry was modelled in Plaxis, conservatively assuming the property store (where the basement wall is stepped back) is not considered. During demolition, the toe of the wall (Section 4 on Figure 16) was removed.

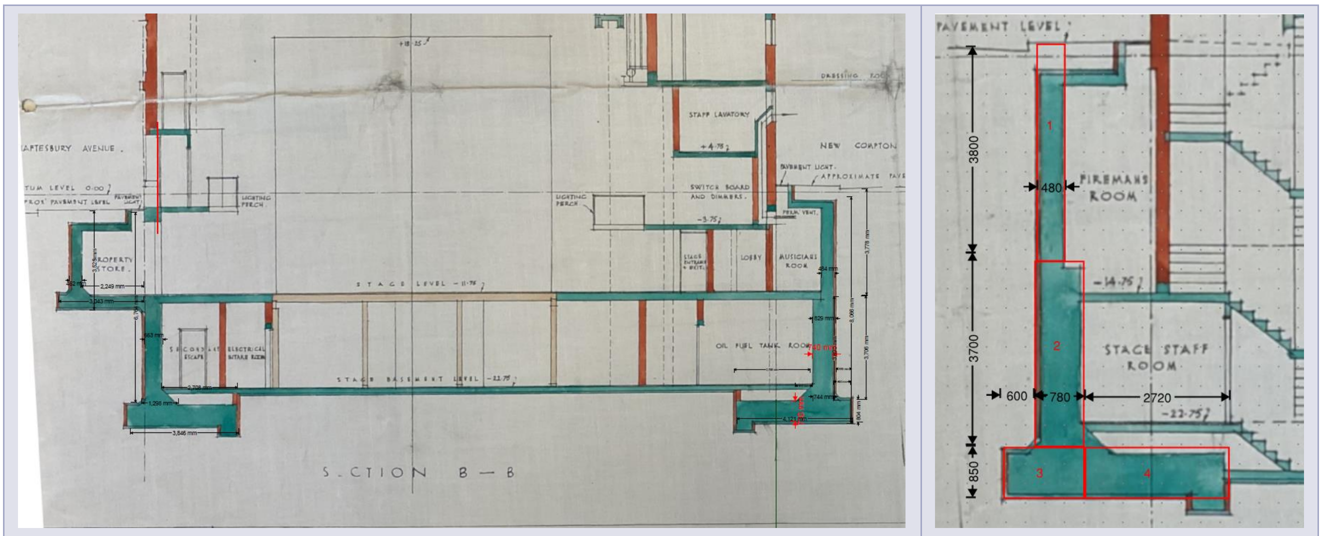


Figure 16: Existing Basement Cross Section (Left) Model Used in FE (Right)

6.4 Structural Inputs

6.4.1 Existing Basement Walls

The existing basement was modelled as a soil volume as per the dimensions in Figure 16, with the parameters of a linear elastic material, therefore in the model the concrete walls could deform under changes in loading but could not fail. The capacity of the concrete sections will be assessed through the design stages by the structural team once additional investigations have been undertaken, and is not considered a part of this scope.

Table 9: Existing Basement Wall - Soil Parameters

Soil Model	Drainage Type	Unit Weight, γ	Stiffness, E_U	Poisson's Ratio, V_U
Linear Elastic	Drained	25 kN/m^3	28 GPa	0.15

6.4.2 Piled Wall

As discussed in Section 3.1 the proposed basement may be formed from 600mm diameter piled walls, likely contiguous as they will be supporting only London Clay. The top of the pile will be at the existing basement wall toe level, taken as -7.5m in the Plaxis 3D model, and a pile length of 25.0m was adopted. Plate elements were used to model the basement walls, which are considered rectangular in the model, therefore the depth of the element in Plaxis was modified to reflect the equivalent moment of inertia value 'I'; refer Table 10.

Table 10: Plaxis 3D Piled Wall Parameters

Pile Diameter	Pile Spacing	Elastic Modulus, E	Moment of Inertia, per m of Wall (I)	Plaxis Inputs			
				Elastic Modulus, E	Depth, d	Unit Weight[1], γ	Poisson's Ratio, ν
600mm	750mm	32 GPa	0.0085m^4/m	28GPa	0.48m	2.68kN/m^3	0.15
1: Unit weight is the weight of the concrete over the weight of the soil							

6.4.3 Liner Wall

In the permanent case, a lining wall will be cast in front of the piled wall to form the permanent basement structure, the size of the lining wall will depend on the type of basement retention used during construction either underpinning or which diameter of piled wall.

Table 11: Liner Wall Plate Parameters

Liner Wall Thickness	Elastic Modulus, E	Depth, d	Moment of Inertia, per m of Wall	Unit Weight[1], γ
300mm	32 GPa	0.30m	0.0023 m^4/m	1.59kN/m^3
1: Unit weight is the weight of the concrete over the weight of the soil				

To consider the effects of the lining wall, a combined plate stiffness was derived by summing the pile and liner walls parameters as per the table below. The flexural parameters of the combined lining wall and piles were not derived using a combined depth of the section, instead summed the individual flexural parameters, therefore does not assume there is flexural fixity between them, so the lining wall is cast on top the piles without the need for bars dowelled into the piles. Plate elements were used to model the basement walls, which are considered rectangular in the model, therefore the depth of the element had to be modified to reflect the equivalent moment of inertia value.

Table 12: Combined Pile and Liner Wall Parameters

Pile Diameter	Pile Spacing	Liner Wall Thickness	Moment of Inertia, per m of Wall	Equivalent depth, d	Unit Weight[1], γ
600mm	750mm	300mm	0.011 m^4/m	0.52m	4.27kN/m^3
1: Unit weight is the weight of the concrete over the weight of the soil					

6.4.4 Underpinning Works

The assessment aimed to consider the cumulative ground movement effects of the underpinning works during construction and the difference between the underpinned and piled wall solutions. The final completed long-term basement form will structurally be similar to the piled wall solution.

As discussed in Section 3.1.2 the proposed underpinning sequence will be formed by 4 No. rows of pins each 2.6m in height with temporary props at each underpin level. Within each row, the pins are proposed to be formed in a 1 to 5 hit and miss sequence, with individual underpins 1.8m in width, and 2.0m in total thickness. The pins are proposed to be constructed by excavating a 2m deep, 1.0m thick and 1.8m wide, trench box supported excavation in front of the existing reinforced concrete retaining wall / previous row of underpins. The underpin excavation is then accessed from the trench box supported area, which once the underpin is excavated and reinforcement and concrete have been replace, is backfilled with lean concrete to be excavated as part of the next dig.

Underpins were modelled in Plaxis 3D by sequentially deactivating / activating and changing the material type of Soil Volumes surrounding the excavation. To expedite the calculations and reduce computation time, the underpin sequence was modelled along the northern and southern facades for Phase 3 of the excavation (away from the corners), as this is considered the most onerous case where the basement has the longest side and therefore the longest span.

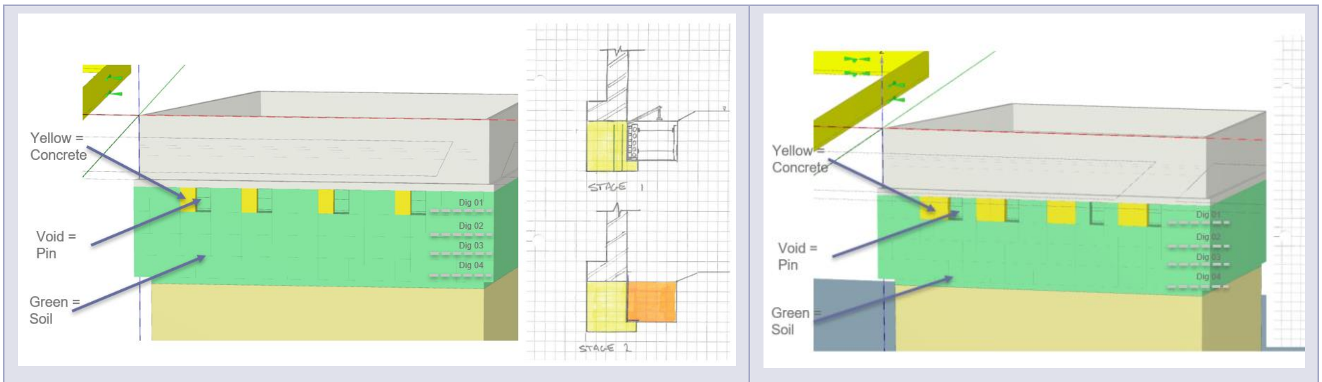


Figure 17: Underpin Soil Volumes

As the concrete strength and stiffness are key input parameters and will vary with time, the design considered the value the concrete will likely reach at the time of excavation. The variation with stiffness was considered as per Table 13. The assessment considered concrete with a final compressive strength of 40MPa and the stiffness of the concrete was related to the compressive strength as follows:

$$E_C = 4700 \sqrt{f_c'} \text{ MPa}$$

Table 13: Concrete Parameters with Time

Parameter	1 day	3 days	7 days	14 days	28 days
Concrete Strength (%)	16%	40%	65%	90%	99%
Concrete Strength (MPa)	6.5	16.0	26.0	36.0	39.6
Concrete Stiffness (GPa)	11.9	18.8	23.9	28.2	29.6

The analysis considered a concrete stiffness of 25GPa for the underpins, therefore this requires a time period 7 to 10 days has elapsed before excavation occurs in front of the underpins.

Table 14: Underpin Dimensions

Dig No.	Underpin Top	Underpin Bottom	Underpin Thickness	Dig Top	Dig Bottom	Dig Thickness	Temporary Prop Level
01	8.35m	11.2m	2.85m	8.35m	11.0m	2.65m	9.7m
02	11.2m	13.8m	2.60m	11.0m	13.6m	2.60m	12.8m
03	13.8m	16.4m	2.60m	13.6m	16.2m	2.60m	16.2m
04	16.4m	19.2m	2.80m	16.2m	18.3m	2.10m	-

6.4.5 Temporary Propping

As temporary propping layouts are to be confirmed by the contractor, specific arrangements were not modelled in Plaxis, instead the temporary propping was modelled using a uniform slab across the basement footprint at the temporary propping elevation with a uniform stiffness. This was considered reasonable as waling beams will run the perimeter of the basement, providing continuous support. Temporary prop stiffness adopted was as per Table 15, the spacing adopted is the centre to centre spacing between a spanning prop and another spanning or corner prop.

Table 15: Temporary Prop Stiffness

Strut Parameters							Plaxis Plate Inputs		
Prop Level	Diameter	Thickness	E_{steel}	EA_{PROP}	Spacing	Length	Thickness	E_{plate}	Poisson Ratio
3.5m	800mm	20mm	200GPa	100.5×10^6kN	7m	22m	300mm	5.2 GPa	0.15
7.0m	800mm	20mm	200GPa	100.5×10^6kN	7m	22m	300mm	5.2 GPa	0.15
9.7m	800mm	20mm	200GPa	100.5×10^6kN	7m	22m	300mm	5.2 GPa	0.15
12.8m	800mm	20mm	200GPa	100.5×10^6kN	7m	22m	300mm	5.2 GPa	0.15
16.2m	800mm	20mm	200GPa	100.5×10^6kN	4m	22m	300mm	9.0 GPa	0.15

6.6 Structural Loading

A preliminary raft foundation design of the B4 slab was undertaken using hand calculations to evaluate the bearing pressure at B4 level. Based on supplied structural loads and a reasonable load spread through the retaining wall and a mobilised slab width of 6.0m, resulted in a bearing pressure of 330kPa, not including the slab self-weight. This load was applied at the B4 level in the permanent case.

6.7 Surcharge Loading

6.7.1 Traffic Loads

Traffic loading was not included in the ground movement assessment, due to its transient nature, and the relatively small zone of influence of it that would be unlikely to cause significant effects on larger scale ground movements.

6.7.2 Existing Buildings

The existing Saville Theatre structure was modelled as a surface load at B2 level of the existing basement. An approximate surface load was applied across the footprint equal to 10kPa multiplied by the height of the structure in storeys, equal to 60kPa.

6.8 Construction Sequence

The behaviour of soils and structures during various construction stages and post-construction has been investigated using a deformation analysis mode, based on “undrained” and “drained” soil parameters whether it is short or long term loading, refer the following tables.

Table 16: Proposed Basement Construction Sequencing – Piled Wall Substructure

Stage	Description	Soil Conditions
A01	Existing conditions	Drained Soil
A02	Demolish existing building and remove surface load	Undrained Soil
A03	Demolish existing B1 and B2 slabs and install temporary propping	Undrained Soil
A04	Demolish toe of existing basement wall	Undrained Soil
A05	Install pile walls and capping beam	Undrained Soil
A06	Excavate to layer of temporary props	Undrained Soil
A06	Install first row of props and excavate to B3 level (13.55m bgl)	Undrained Soil
A07	Excavate to next layer of temporary props	Undrained Soil
A08	Install next prop row and excavate to B4 raft formation level (18.30m bgl)	Undrained Soil
A09	Construct B4 level permanent slab	Undrained Soil
A10	Excavate for sprinkler tanks	Undrained Soil
A11	Install sprinkler tank walls	Undrained Soil
A12	Install permanent floor slabs and remove temporary props	Undrained Soil
A13	Add in structural loads (raft and sprinkler tank) and change piled wall plate parameters to reflect addition of liner wall	Undrained Soil
A14	Long term conditions	Drained Soil Consolidation Analysis

The underpinning option was modelled as follows with Stage 01 to 04 as per the piled wall sequence.

Table 17: Proposed Basement Construction Sequencing – Underpinned Substructure			
Stage	**Dig No.**	**Description**	**Soil Conditions**
B05	01	Install corner pins (Phase 1). Pins “wished into place” in Phase 1 corners. Excavate every 1 in 5 pins beneath existing retaining wall along Phase 2&3. Fix internal basement side to model trench box internally.	Undrained Soil
B06	01	Replace excavated pins from Stage B05 with concrete. Excavate next pin adjacent to pins from previous stage. Fix internal basement side to model trench box internally.	Undrained Soil
B07	01	Replace excavated pins from Stage B06 with concrete. Excavate next pin adjacent to pins from previous stage. Fix internal basement side to model trench box internally.	Undrained Soil
B08	02	Excavate to top of next dig level at 11.0m. Install temporary props as per Table 14.	Undrained Soil
B09	02	Repeat Stage B05 at Dig No. 02	Undrained Soil
B10	02	Repeat Stage B06 at Dig No. 02	Undrained Soil
B11	02	Repeat Stage B07 at Dig No. 02	Undrained Soil
B12	03	Excavate to top of next dig level at 13.6m. Install temporary props as per Table 14.	Undrained Soil
B13	03	Repeat Stage B05 at Dig No. 03	Undrained Soil
B14	03	Repeat Stage B06 at Dig No. 03	Undrained Soil
B15	03	Repeat Stage B07 at Dig No. 03	Undrained Soil
B16	04	Excavate to top of next dig level at 16.2m. Install temporary props as per Table 14.	Undrained Soil
B17	04	Repeat Stage B05 at Dig No. 04	Undrained Soil
B18	04	Repeat Stage B06 at Dig No. 04	Undrained Soil
B19	04	Repeat Stage B07 at Dig No. 04	Undrained Soil
B20	04	Excavate to final dig level at 18.8m.	Undrained Soil
B21	-	Construct B4 level permanent slab	Undrained Soil
B22	-	Excavate for sprinkler tanks	Undrained Soil
B23	-	Install sprinkler tank walls	Undrained Soil
B24	-	Install permanent floor slabs	Undrained Soil
B25	-	Remove temporary props	Undrained Soil
B26	-	Add in structural loads (raft and sprinkler tank)	Undrained Soil
B27	-	Long term conditions	Drained Soil Consolidation Analysis

6.9 Plaxis Outputs

Plaxis outputs are presented for cross sections running north to south (A-A') and west to east (B-B') for the cases immediately following construction, and for long term considering the effects of drained unloading heave.

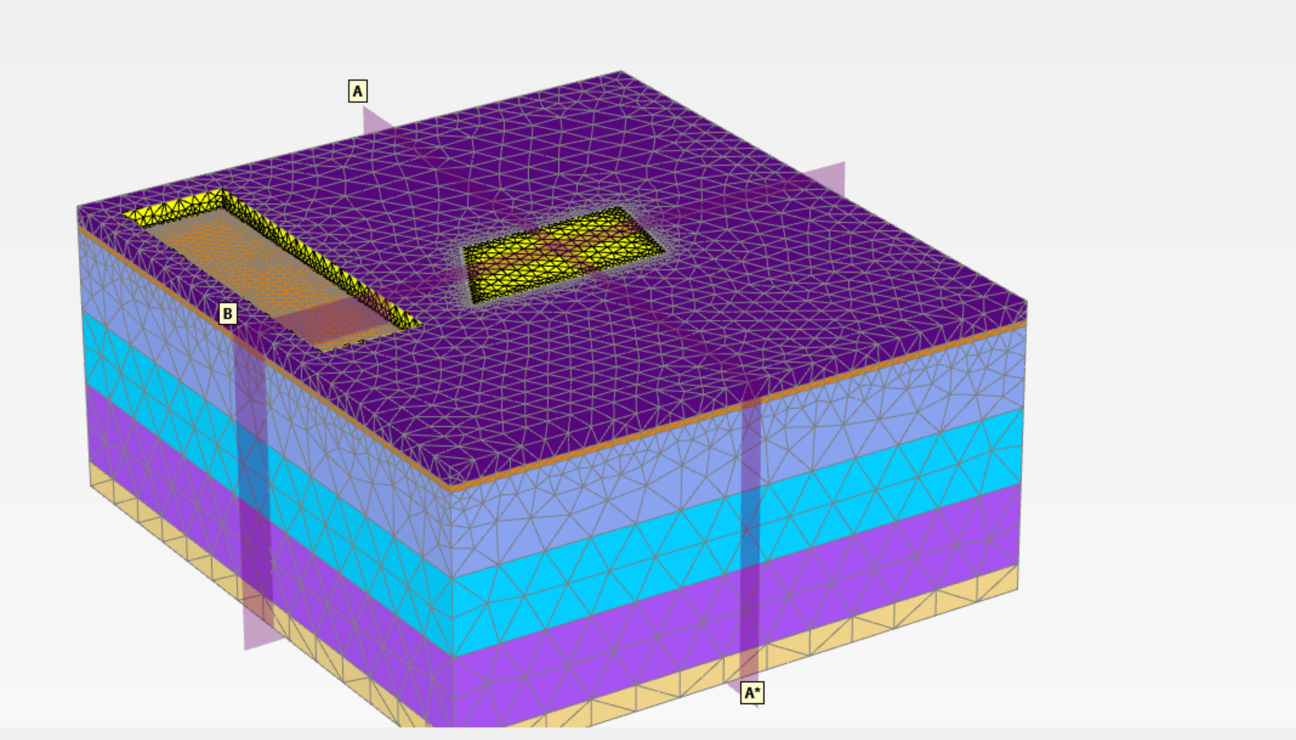


Figure 18: Model Cross Section Locations

6.9.1 Piled Wall Case

Movements of the piled wall shown below indicate up to 60mm of wall movement at the base of the excavation (B4 formation level) for the 600mm diameter pile case. Movement vectors show the ground moving into the excavation and heave occurring beneath the slab formation level.

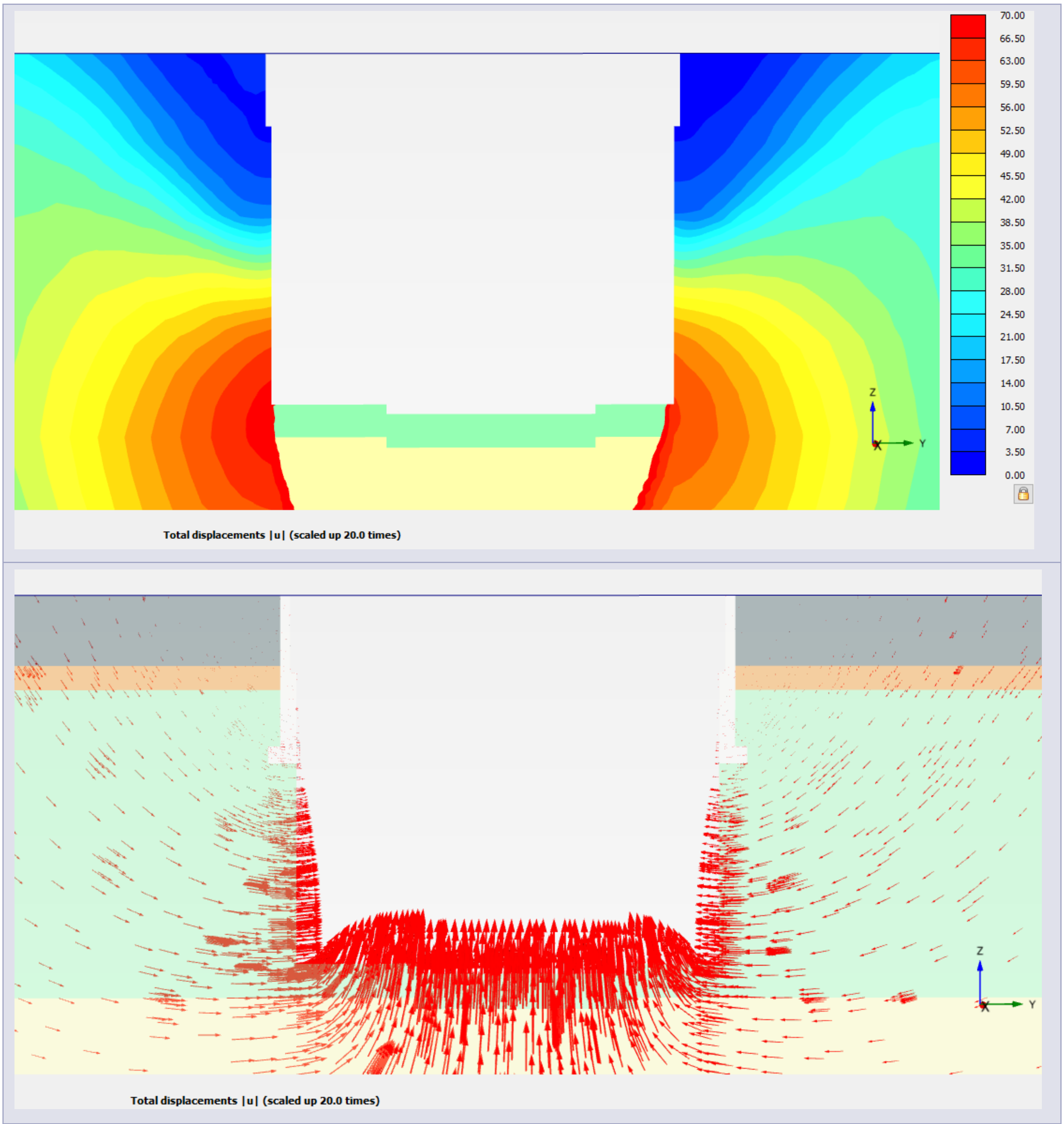


Figure 19: Section A-A' Piled Wall Total Displacements– Excavation to B4 Formation

Long term movements increase as excess pore pressures dissipate and the soil returns to drained conditions. Considering the low permeability of the London Clay and cohesive Lambeth, it is considered likely that this will take approx. 20+ years to occur. Movements increase to a maximum heave of 170mm beneath the centre of the sprinkler tank excavation, with an increase of approximately 5mm in basement wall deflections.

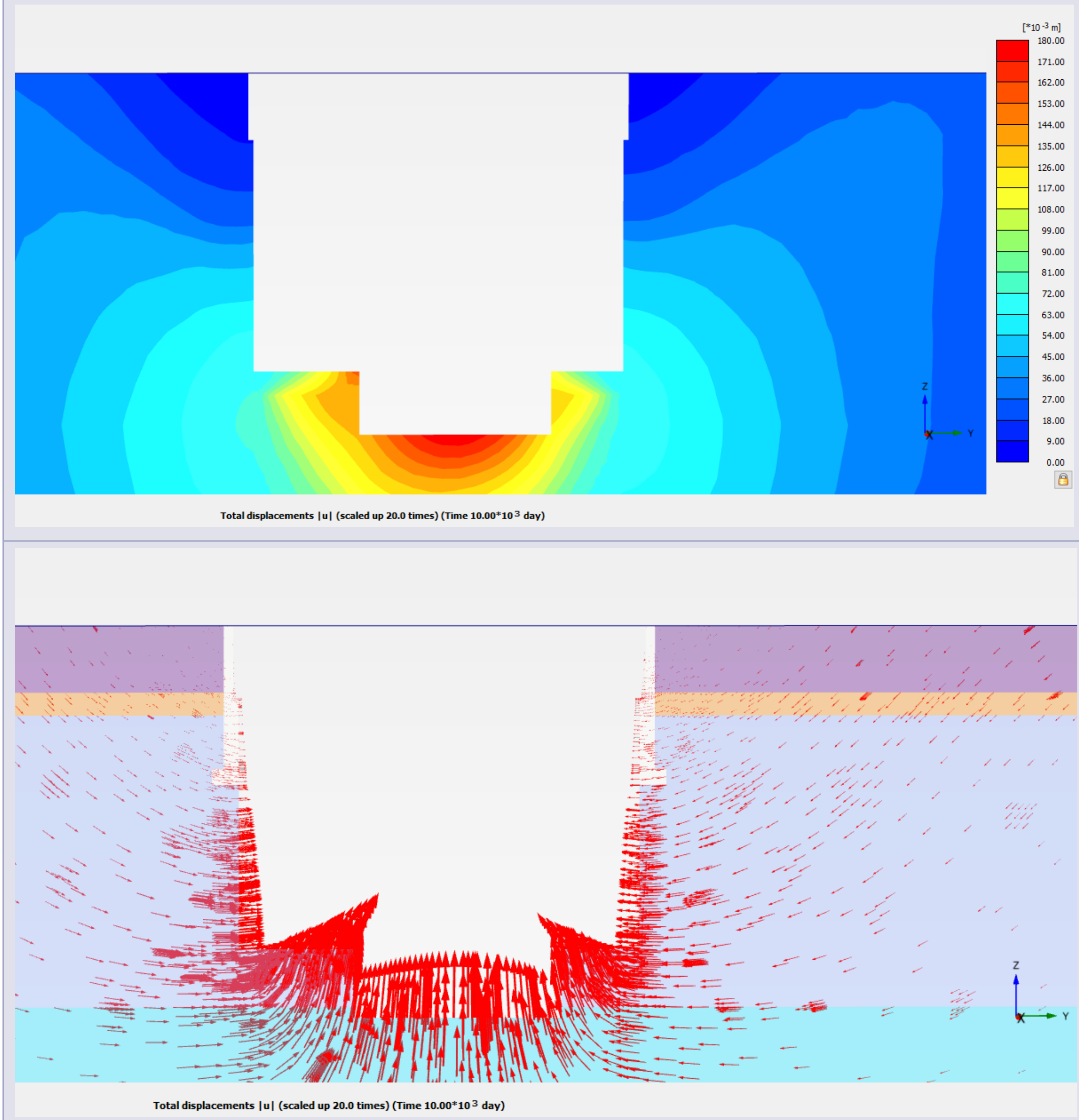


Figure 20: Section A-A' Piled Wall Total Displacements – Long Term

The west to east section exhibits lower piled wall movements, up to 40mm at the formation level. These lower deflections are due to the shorter span along the basement walls running along Stacey Street and St Giles Passage thereby providing more corner restraint.

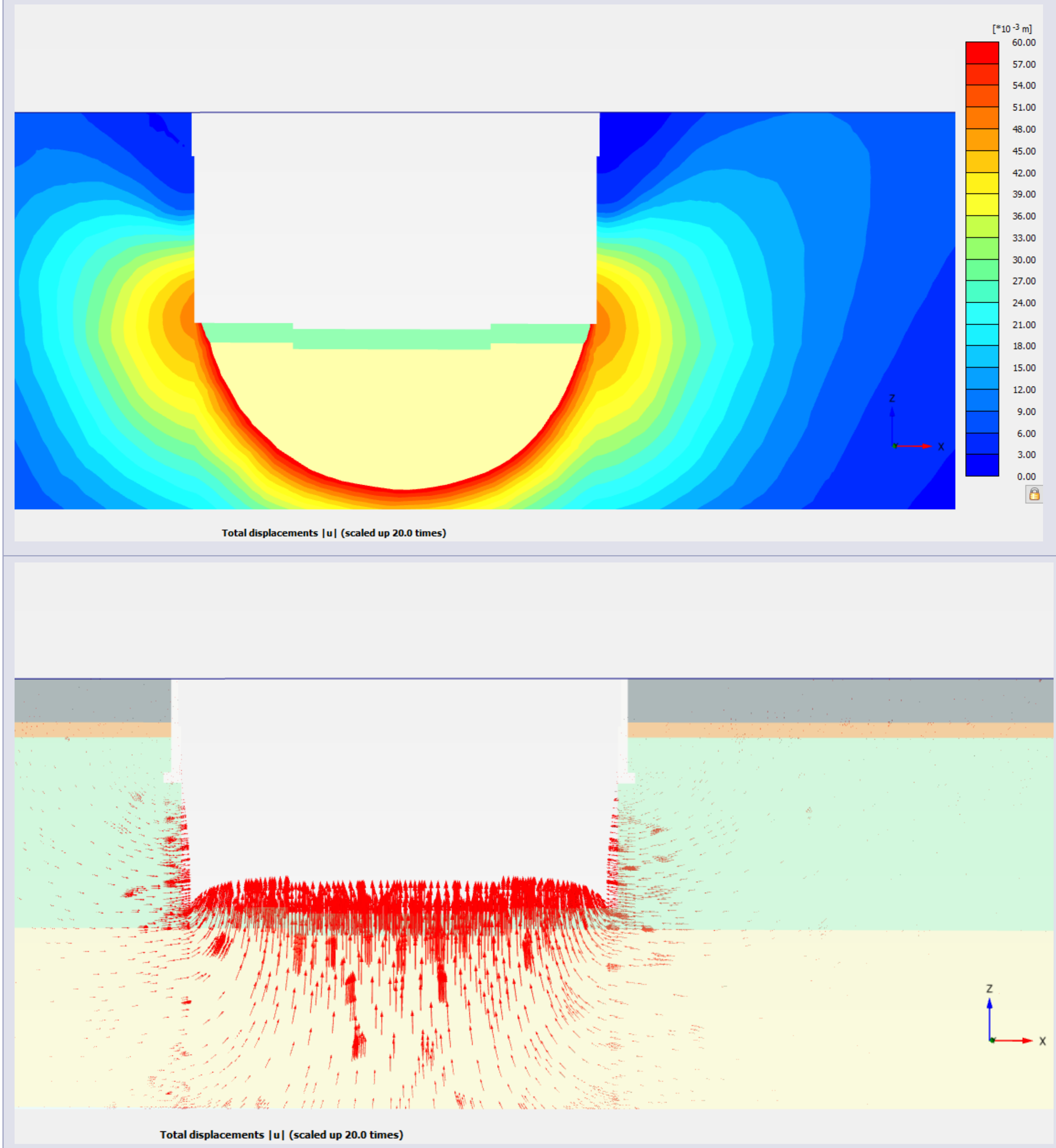


Figure 21: Section B-B' Piled Wall Total Displacements – End of Construction

In the long term, heave increases to approx. 170mm at the centre of the excavation, with an increase in wall deflections along the eastern façade to 60mm.This is likely to be as a result of the unsupported section of eastern basement wall where the theatre stage is located, the western wall does not experience the same amount of movement, refer Figure 8.

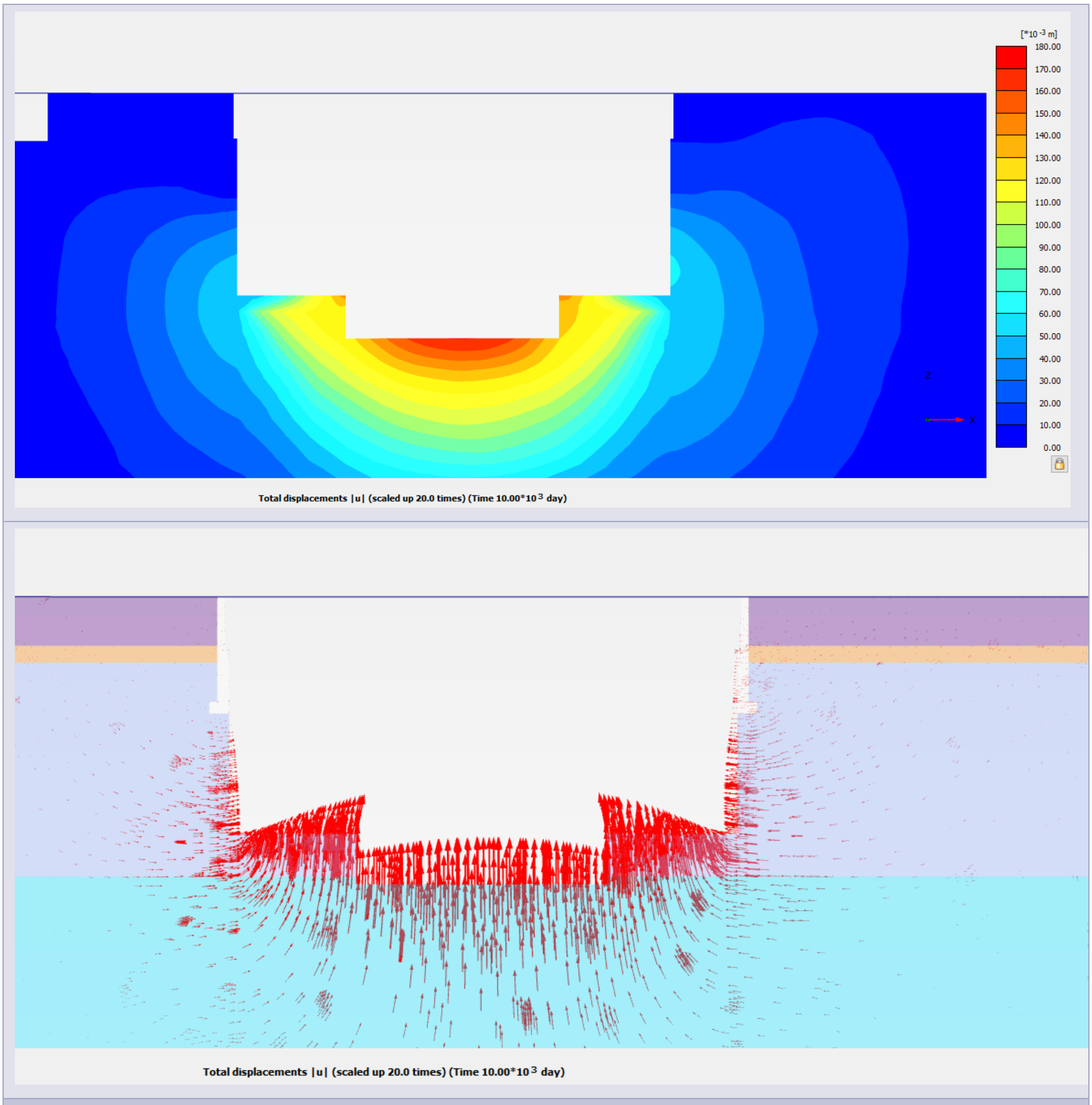


Figure 22: Section B-B' Piled Wall Total Displacements – Long Term

6.9.2 Underpinning

The following sections will present the same outputs as for the piled wall case to allow comparison between the ground movement sections. It will also show anticipated ground movements caused by the underpins to understand the effects of the sequential excavation. An additional cross section is included, C-C' running behind the basement walls along the Shaftesbury Avenue boundary to show the effects of the underpinning during construction.

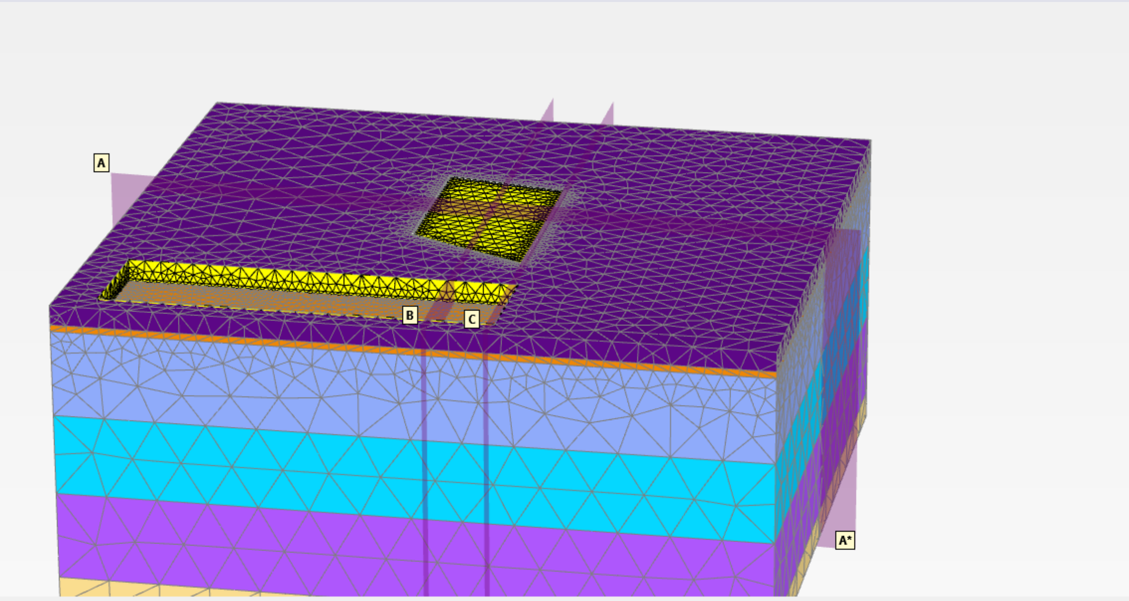


Figure 23: Model Cross Section Locations

The total displacement experienced by the walls during the underpinning works is shown in Figure 24. This generally shows the influence of the underpinning on the ground movements is localised to the area around the underpinning, as a result of the small width of the underpins. Contours of movement from the underpins do not progress past the underside of the existing retaining wall, and therefore the underpinning construction is not anticipated to significantly increase ground movements outside of the area of the underpins. The movement vectors are generally into the underpin excavation and the resulting heave/ settlement.

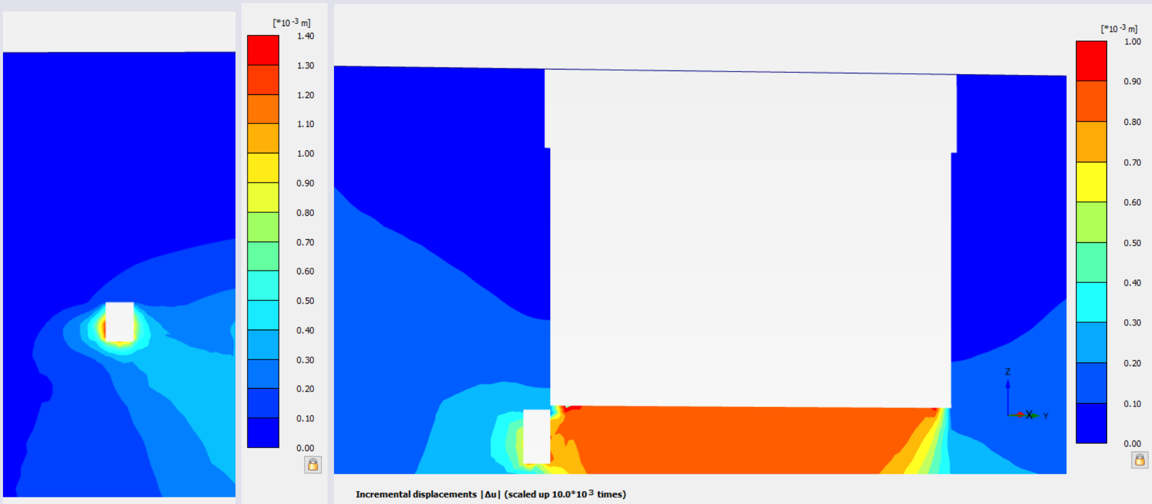


Figure 24: Dig 04 Incremental Displacements During Construction

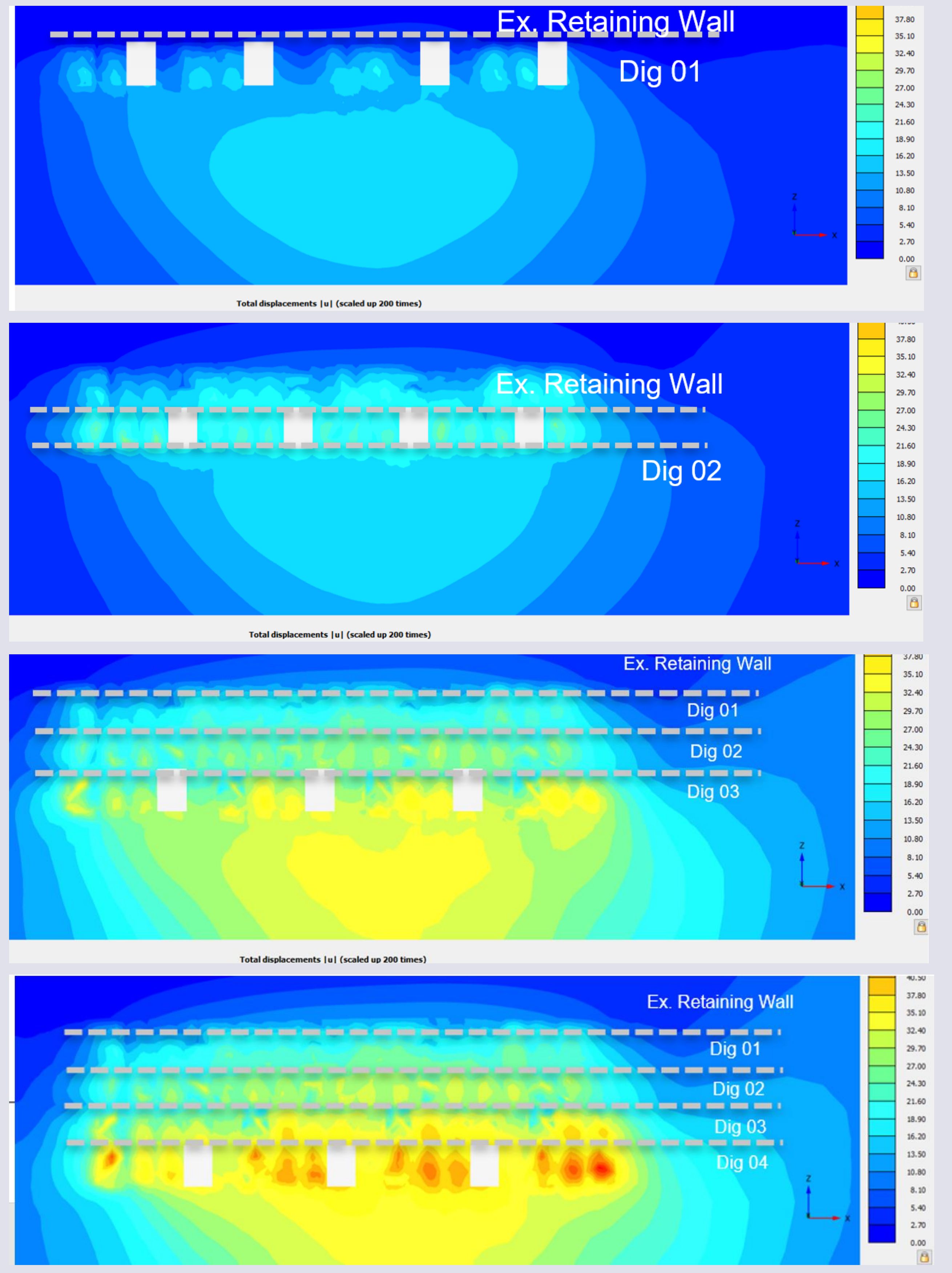


Figure 25: Section C-C' Total Displacements Along Underpins During Construction

Movements of the underpinned wall shown below indicate up to 45mm to 50mm of wall movement at the base of the excavation (B4 formation level) for the excavation to B4 formation level case. Movement vectors show the ground moving into the excavation and heave occurring beneath the slab formation level.

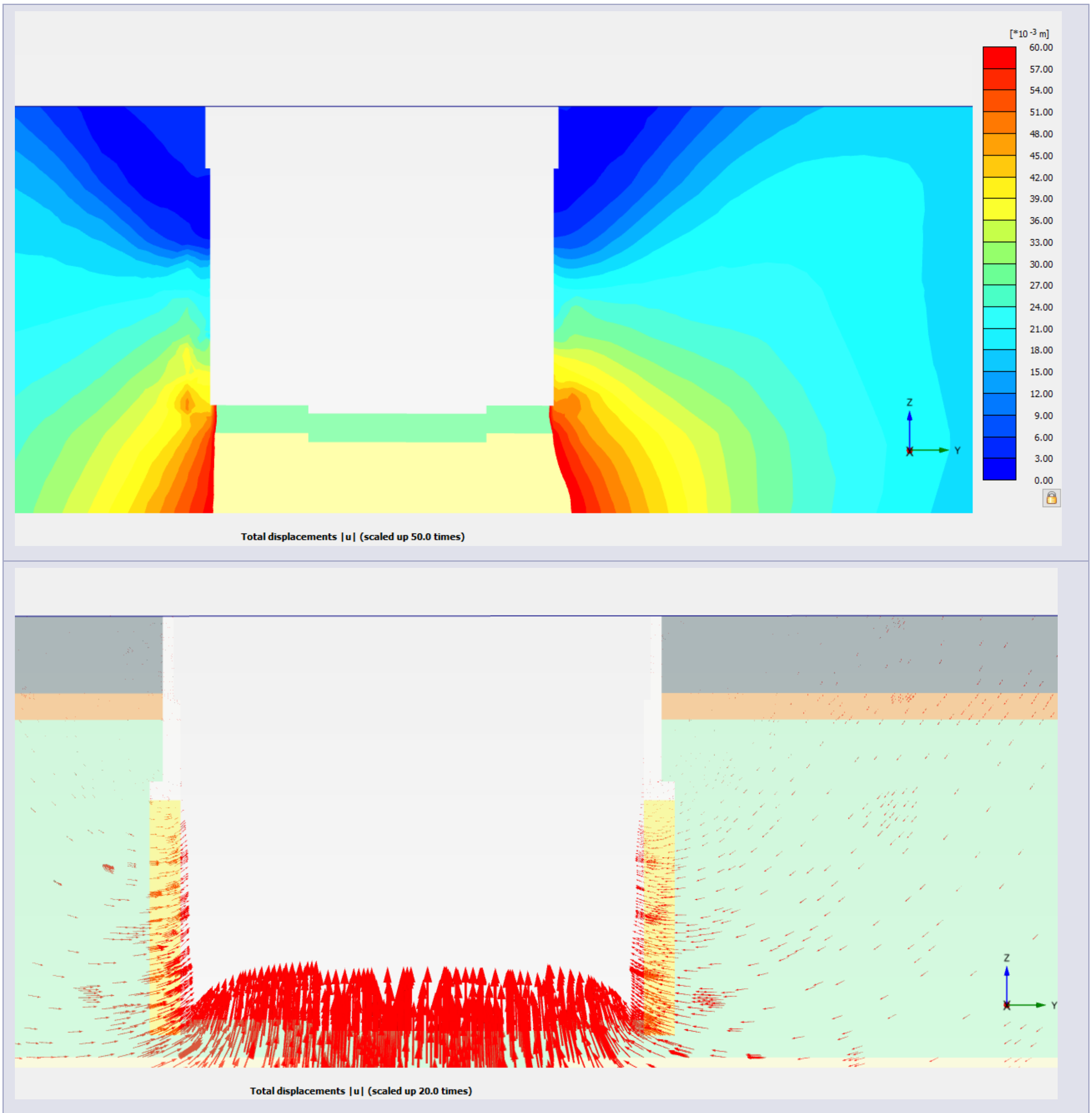


Figure 26: Section A-A' Underpin Total Displacements – Excavation to B4 Formation

Long term movements increase as excess pore pressures dissipate and the soil returns to drained conditions. Considering the low permeability of the London Clay and cohesive Lambeth, it is considered likely that this will take approx. 20+ years to occur. Movements increase to a maximum heave of 170mm beneath the centre of the sprinkler tank excavation (as per the piled wall case), with no significant change in basement wall deflections.

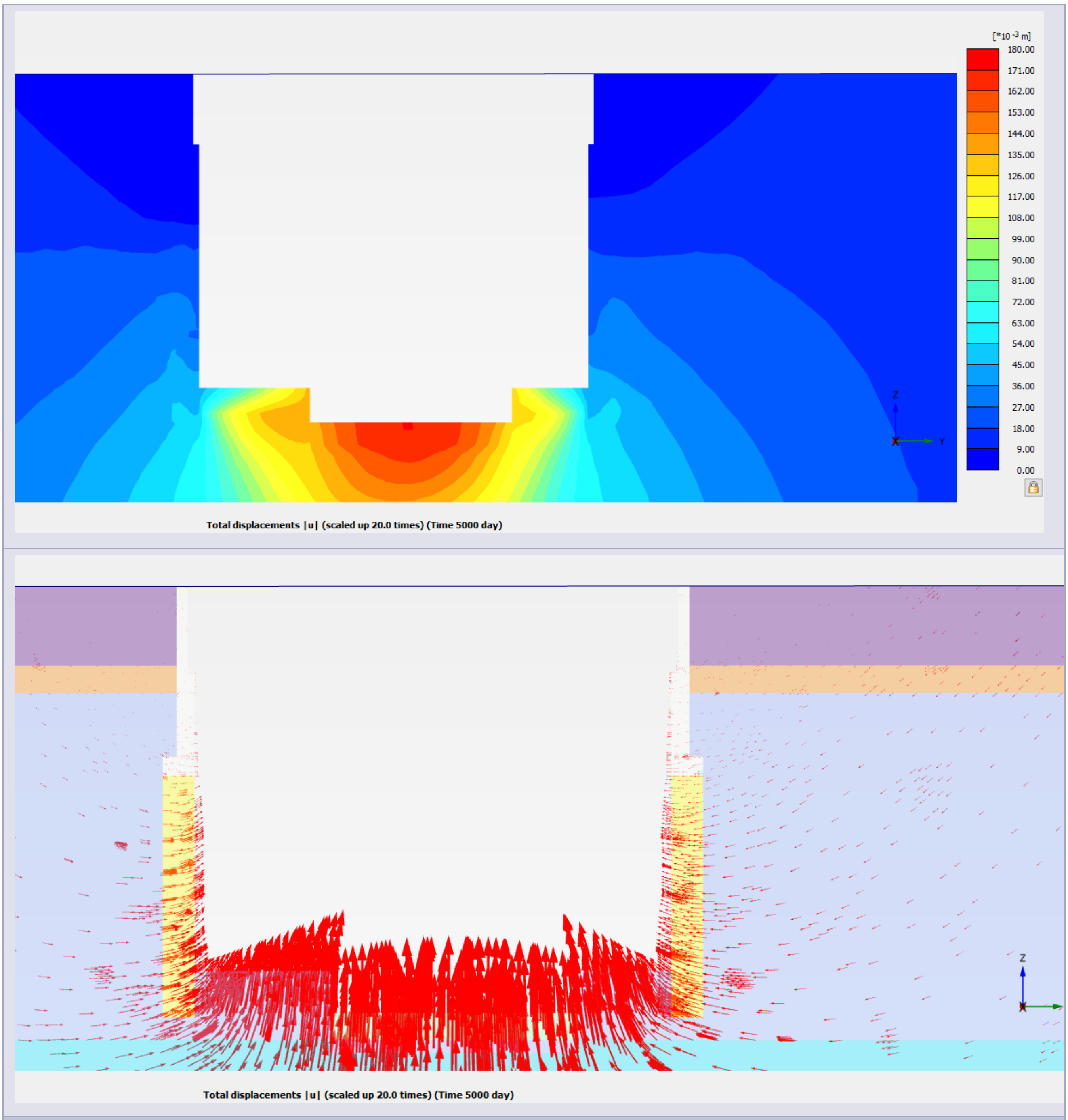


Figure 27: Section A-A' Underpin Total Displacements – Long Term

Movements of the underpinned wall shown below indicate up to 35mm to 40mm of wall movement at the base of the excavation (B4 formation level) for the underpin case. The slightly lower deflections are likely due to the shorter span along the basement walls running along Stacey Street and St Giles Passage thereby providing more corner restraint to the walls. Movement vectors show the ground moving into the excavation and heave occurring beneath the slab formation level.

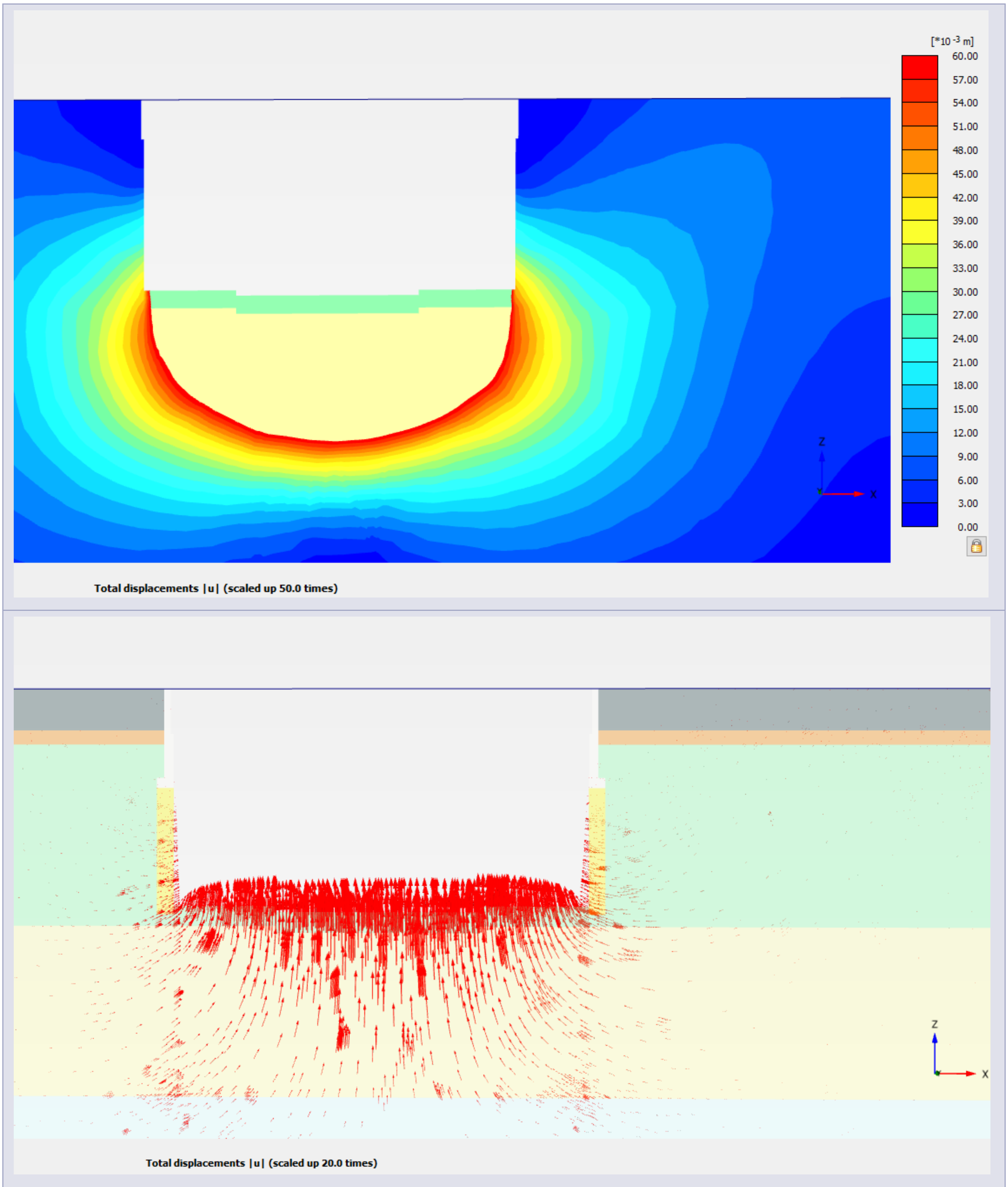


Figure 28: Section B-B' Underpin Total Displacements – Excavation to B4 Formation

In the long term, heave increases to approx. 170mm at the centre of the excavation, with an increase in wall deflections along the eastern basement wall to approx. 45mm, likely as a result of the unsupported section of basement wall where the theatre stage is located.

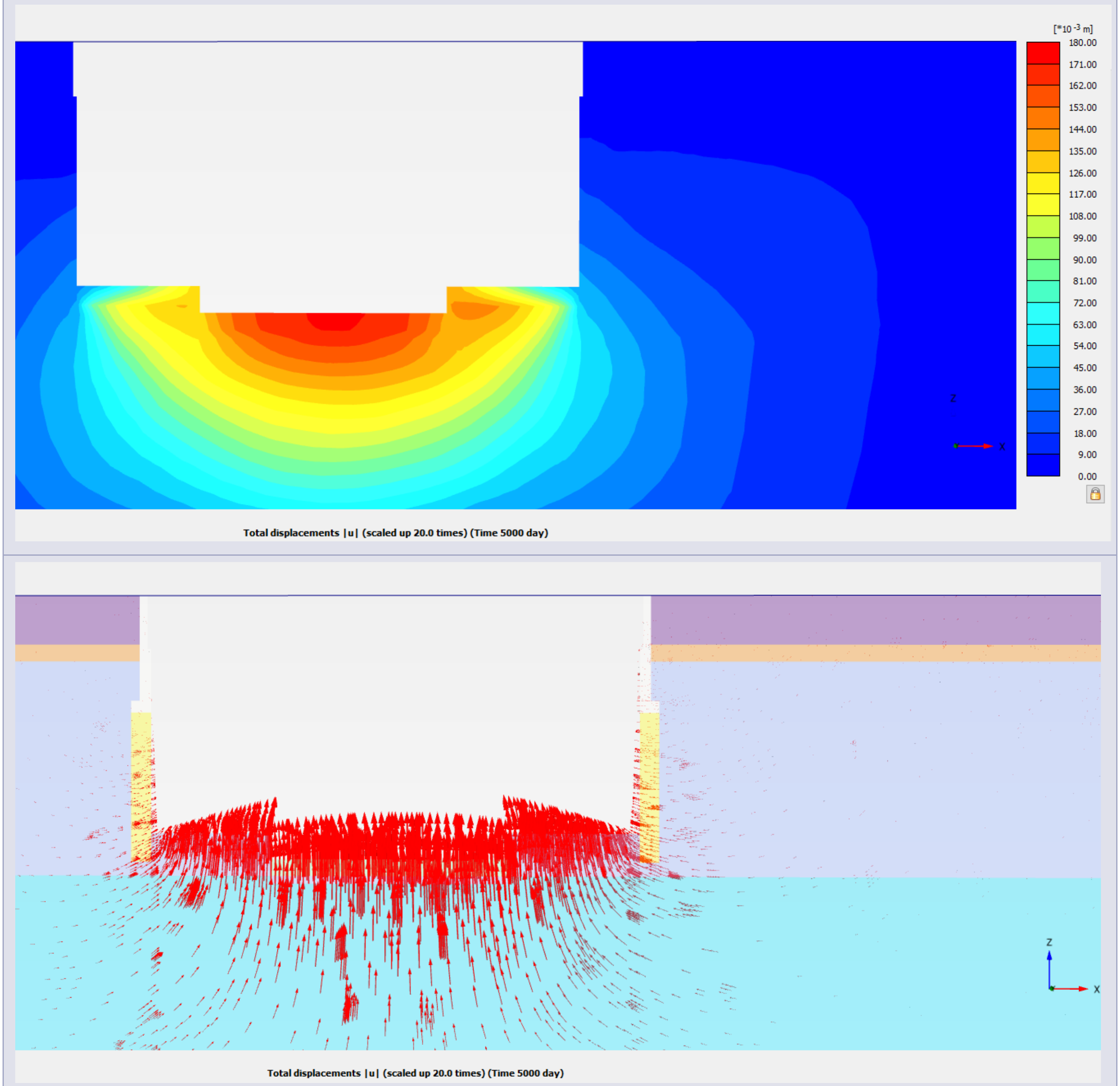


Figure 29: Section B-B' Underpin Total Displacements – Long Term

6.9.3 Comparison of Underpins and Piled Wall

The same cross section is compared side by side below in Figure 30, and shows minor changes in the ground deformation profile behind the wall for the two cases.

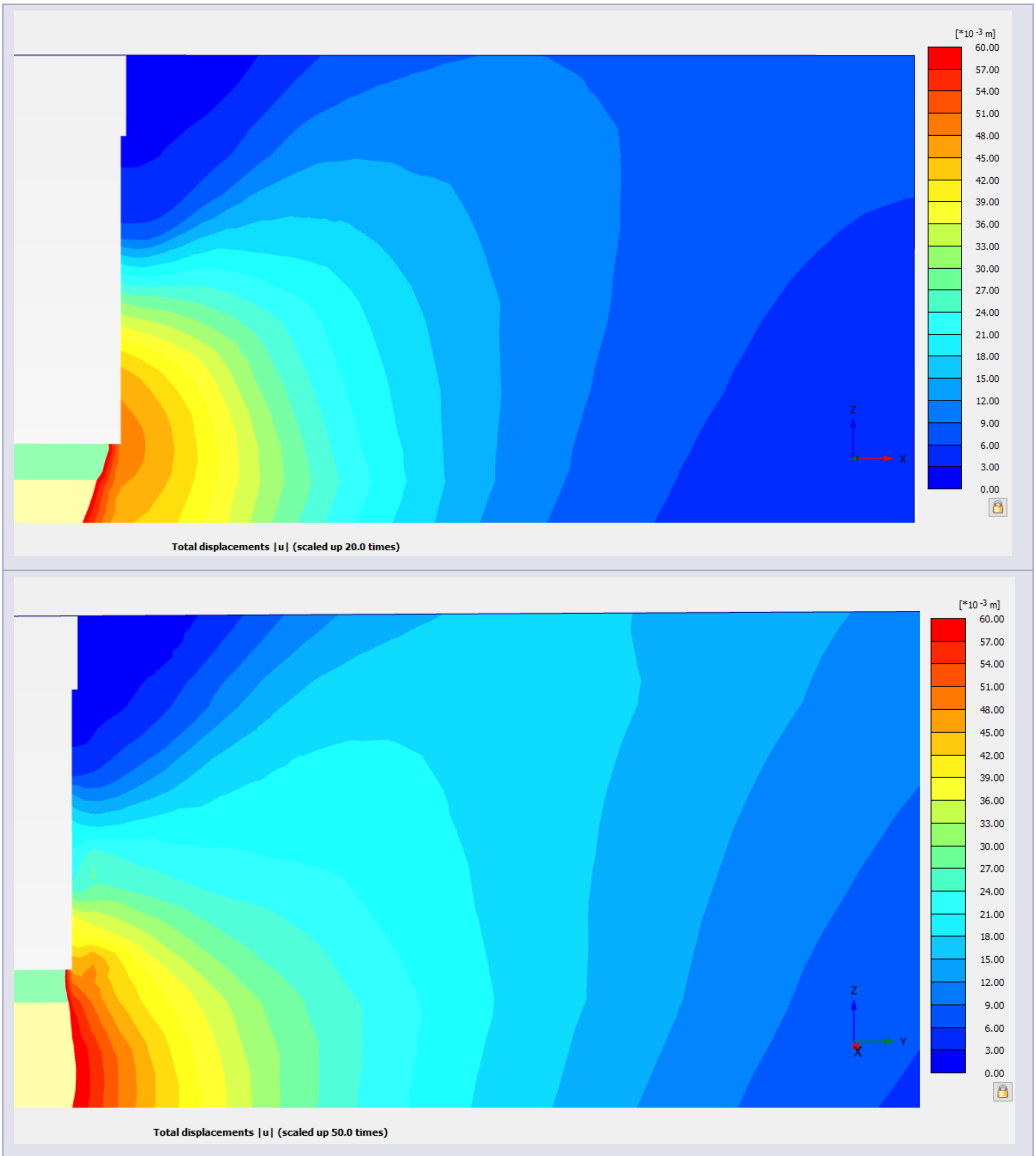


Figure 30: Section B-B’ B4 Formation Excavation, Piling (Top) and Underpinning (Bottom)

The settlement areas around the basement is compared below for the piling and underpinning case in the figure below. Similar vertical settlements are observed surrounding the excavation for the two cases with settlement up to 20mm recorded for the piling case, and up to 16mm for the underpinning case.

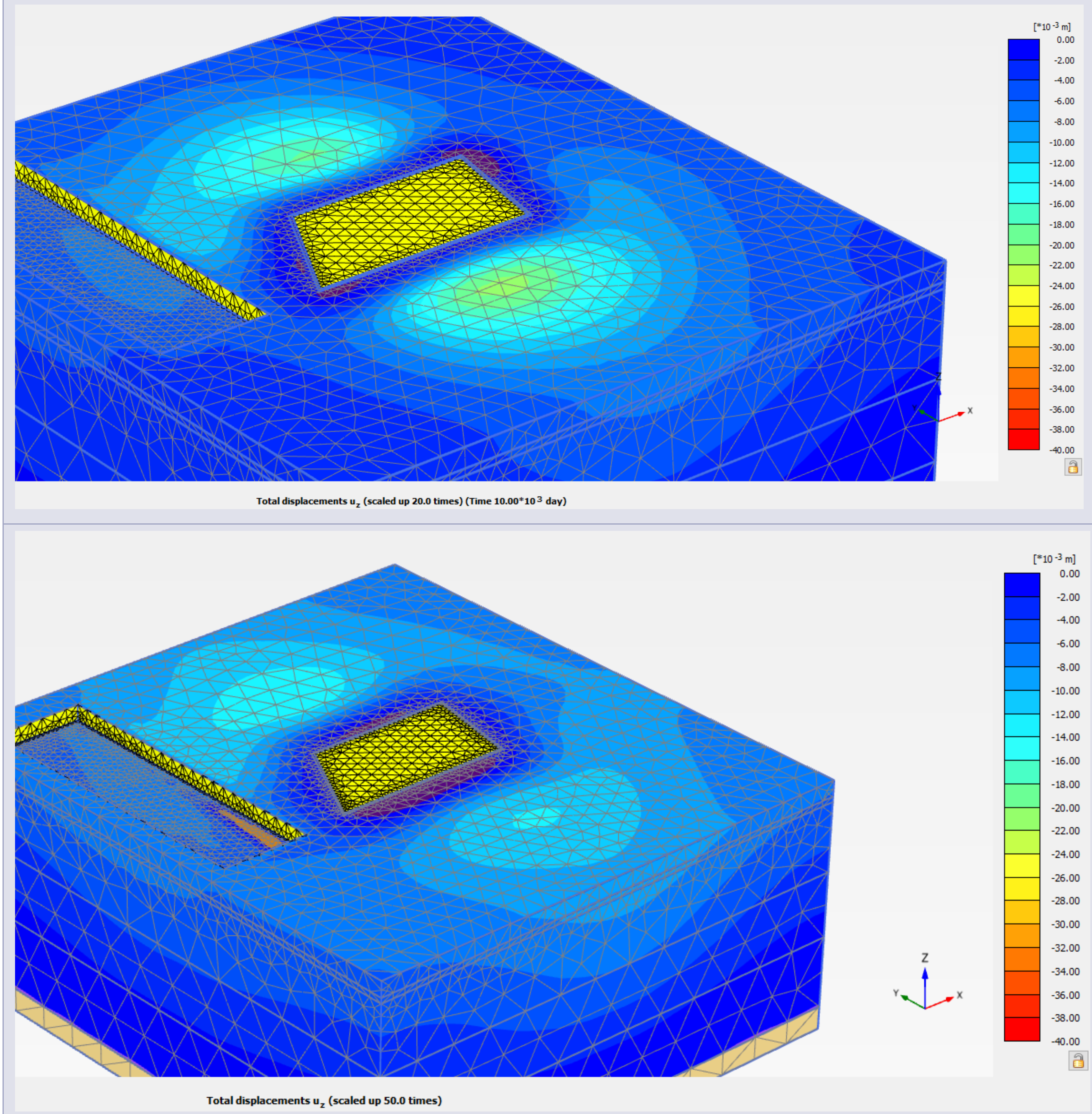


Figure 31:Total Vertical Settlements around Basement for Piling (Top) and Underpin Case (Bottom)

7 Damage Category Assessment

A damage category assessment has been carried out to investigate possible levels of damage to structures affected by the proposed Saville Theatre development, the effects on existing utilities have not been considered as part of this assessment, but can be included in the damage assessment once engagement with utility providers and their specific requirements is obtained.

The 3D Finite Element Model has been used to derive differential movements and strains along the façade lines of the structures, and damage classifications derived from those in accordance with CIRIA C760, refer to Table 18.

Table 18: Damage Classifications as per Burland et al. (1977)

Category	Description and Typical Damage	Approximate Crack Width (mm)	Limiting Tensile Strain ε_{lim} (%)
0 Negligible	Hairline cracks of less than about 0.1 mm are classed as negligible	< 0.1mm	0.0 to 0.05
1 Very slight	Fine cracks that can easily be treated during normal decoration. Perhaps isolated slight fracture in building. Cracks in external brickwork visible on inspection	< 1	0.05 to 0.075
2 Slight	Cracks easily filled. Redecoration probably required. Several slight fractures showing inside of building. Cracks are visible externally and some repointing may be required externally to ensure weathertightness. Doors and windows may stick slightly.	< 5	0.075 to 0.15
3 Moderate	The cracks require some opening up and can be patched by a mason. Recurrent cracks can be masked by suitable lining. Repointing of external brickwork and possibly a small amount of brickwork to be replaced. Doors and windows sticking. Service pipes may fracture. Weathertightness often impaired.	5 to 15 or a number of cracks greater than 3	0.15 to 0.30
4 Severe	Extensive repair work involving breaking-out and replacing sections of walls, especially over doors and windows. Windows and frames distorted, floor sloping noticeably. Walls leaning or bulging noticeably, some loss of bearing in beams. Service pipes disrupted.	15 to 25 but also depends on number of cracks	> 0.30
5 Very Severe	This requires a major repair involving partial or complete rebuilding. Beams lose bearings, walls lean badly and require shoring. Windows broken with distortion, Danger of instability.	Usually greater than 25 but depends on number of cracks	-

Buildings affected by excavations can experience different types of deformation including sagging, hogging or rigid body movement, and the amount of damage experienced can also depend on the construction type and material stiffness, the number of openings and joints etc.

Table 19 below summarises the facades considered in the assessment and the resulting damage category based on the work of Burland (2001) whereby for different ratios of height to length, settlement profiles are used to derive deflection ratios (essentially a measure of bending strain) and horizontal strains, and the resulting interaction is used to determine a potential damage category for the structure. This assessment conservatively considers the structures as masonry, however reinforced concrete-framed structures will be more flexible in shear than masonry and are consequently less susceptible to damage.

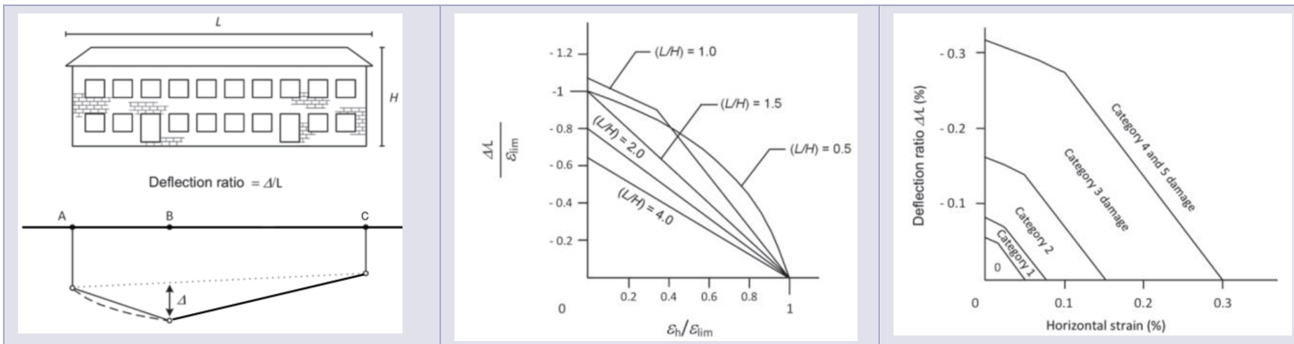


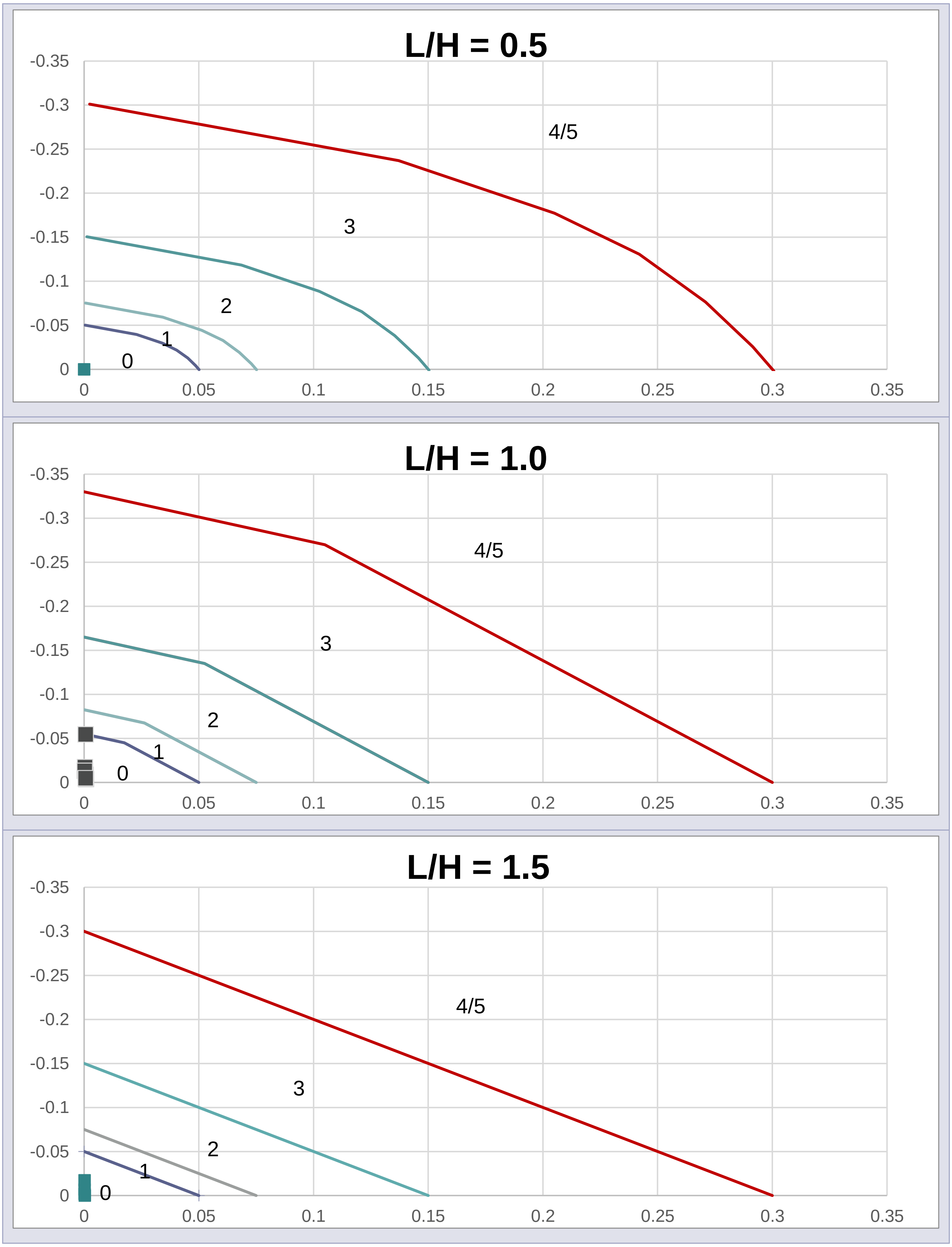
Figure 32: Deflection Ration (Left), Relationship between Deflection and Strain (Centre and Right)

The facades considered for the damage category assessment are as per **Figure 13**, and the calculated damage category for each is as per below.

Table 19: Adjacent Structures Movements

				Point A			Point B		Point C		
Facade	Length (m)	Height (m)[1]	L/H Ratio	Length along Façade (m)	Settlement (mm)	Horizontal Movement (mm)	Length along Façade (m)	Settlement (mm)	Length along Façade (m)	Settlement (mm)	Horizontal Movement (mm)
A1	25.0	25.0	1.00	0	8	17	5	9	25	2	2.5
A2	50.0	25.0	2.00	0	8	18	5	12	50	0	2
A3	50.0	25.0	2.00	0	8	18	5	12	50	0	2
B1	35.0	20.0	2.00	0	7	12	8	8	35	8	5
B2	20.0	20.0	1.00	0	7	12	10	11	20	8	6
B3	30.0	20.0	1.50	0	6	11	15	4	30	2	4
C1	15.0	3.5	4.50	0	15	21	10	24	12	21	25
C2	20.0	3.5	5.50	0	22	26	2	24	20	16	19
D1	30.0	40.0	1.00	0	10	11	7	14	30	10	16
D2	10.0	40.0	0.50	0	8	8	3	9	10	3	5
D3	70.0	40.0	2.00	0	6	4	18	11	60	6	4
D4	20.0	40.0	0.50	0	4	6	10	4	20	4	6
E1	15.0	15.0	1.00	0	21	26	8	20	15	18	14
E2	25.0	15.0	1.50	0	15	21	2	14	21	8	12
F1	25.0	20.0	1.50	0	21	26	10	16	20	16	22
F2	20.0	20.0	1.00	0	21	26	10	16	20	9	15
F3	25.0	20.0	1.50	0	15	8	19	14	25	8	12
G1	15.0	20.0	1.00	0	14	22	8	18	15	8	14
G2	30.0	20.0	1.50	0	14	14	15	12	30	6	10
G3	30.0	20.0	1.50	0	10	14	15	9	30	7	8
Crossrail	-	-	-	Maximum settlement of 3mm at the base of the tunnel							
1 Includes the height above ground level and basement depth											

Damage categories from the assessment are presented below. Damage Categories are generally fall into Category 0, except for facade E1, where the change in differential movement over the relatively short façade span (15.0m) resulted in Category 1 damage.



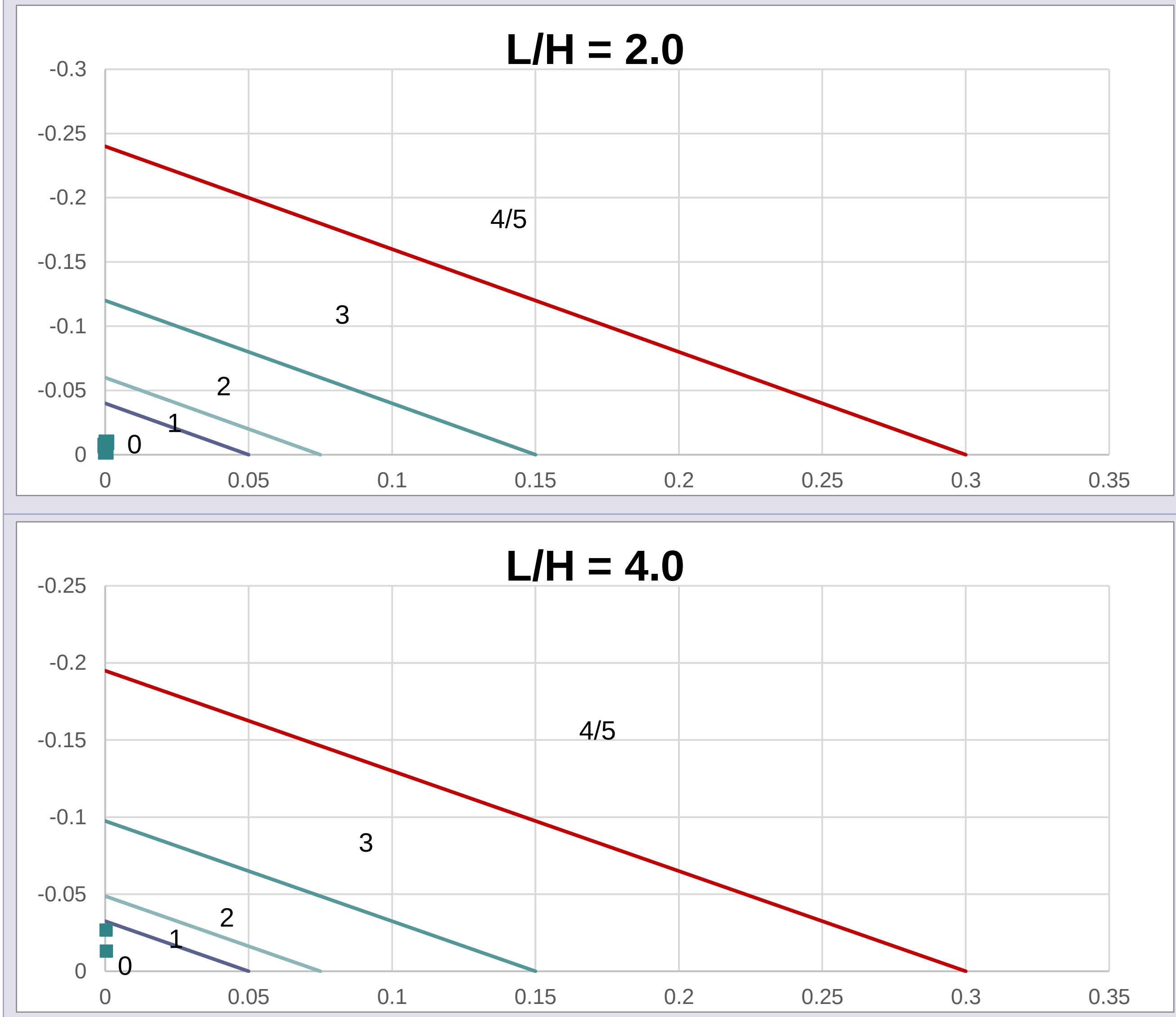


Figure 33: Deflection Ration (Left), Relationship between Deflection and Strain (Centre and Left)

8 Ground Movement Related Risks

The following risks, relating to parameters the ground movement assessment is predicated upon and associated third party risks are outlined below.

Table 20: Ground Movement Assessment Risk Profile

Risk	Mitigation Measures
Information Requirements	
Substructure information to structures A1 to G3 and Crossrail, as outlined in Figure 13.	Engagement with adjacent property holders to confirm structural arrangement of buildings
Site specific geotechnical information; this document is reliant on archive information only.	Undertake site specific geotechnical investigation (as tendered)
Clarification of development geometry and application of load to the ground.	Finalised structural layouts and progressed superstructure design
Clarification on geometry and structural capacity of existing retained theatre footings, retained walls etc.	Undertake trial puts and intrusive investigation works
Modelled stiffness of either the piled wall or underpinning solution.	Development of RIBA Stage 2/3 scheme to enable appropriate wall stiffens to be incorporated into FE model
Further development of temporary propping arrangements, ground deflections dependant on temporary prop stiffness.	Further development of temporary works design by Contractor.
Third Party Risks	
Party wall agreements with adjacent affected structures A1 to G3 and Crossrail, as outlined in Figure 13	Engage with affected property owners to develop party wall agreements where required
Establishment of Damage Categorisation (as outlined in Table 18: Damage Classifications as per Burland et al. (1977) limits on third party structures. This impacts on required design stiffnesses to Saville Theatre basement structures, foundation structure size, geometry and costs	Engage with affected property owners to develop party wall agreements where required
Establishment of Crossrail requirements. This impacts on required design stiffnesses to Saville Theatre basement structures, foundation structure size, geometry and costs	Engage with affected property owners to develop party wall agreements where required
Third party agreements with utility providers – Thames Water	Engage with utility providers to prepare agreements.
Costs associated with monitoring requirements	Engagement with key stakeholders to establish form/monitoring requirements and period of time over which monitoring is required. Development of an instrumentation and monitoring plan.
Supply chain engagement and agreement to proposed construction sequence	Further development of temporary works design by Contractor.
Other	
Ground movement assessment technical aspects	Review by third party geotechnical team appointed by the Client

9 Conclusions and Further Work

A 3D Finite Element analysis was undertaken to investigate the effects of the proposed development on the adjacent structures and to investigate the difference between a piled wall basement substructure and a underpinned basement solution. The results of the check and anticipated movements are summarised in Section 6.9 and the results of the damage assessment is summarised in Section 7.

The results of this assessment do not indicate a significant difference in predicted ground movements and resulting damage category to adjacent structures and 3rd party assets between the piled wall and underpinning basement cases. The damage category assessment has predicted a damage category of Category 1 for adjacent structures, indicating *Fine cracks that can easily be treated during normal decoration. Perhaps isolated slight fracture in building. Cracks in external brickwork visible on inspection.*

Adjacent services affected by the zone of influence of the basement excavation are outlined in Section 4.1.4, this ground movement assessment has not undertaken a damage assessment for each utility and service affected, as this will require engagement with the 3rd party asset owner and will be the subject of a separate report forming an agreement for the works.

As much of the final structural layout and construction details and sequencing is still to be confirmed, this report has made a number of key assumptions that are outlined in the Plaxis 3D inputs in Section 6.4. The majority of ground movements occur during construction when excavation progresses, and this is a function of the stiffness of the wall element, the embedment of the wall element beyond the excavation level, the location of the temporary propping and the stiffness of the temporary propping (number and size of props at each level). The stiffnesses and locations adopted for the temporary props is as outlined in Table 15 and based on preliminary calcs indicates that the lowest level of temporary propping will require props at a spacing of 4m to control deflections, therefore likely requiring 2 No. circular hollow sections across the longest span with corner props.

For the underpinning case the stiffness of the underpin concrete elements was adopted as per Section 0, and therefore the sequencing for the underpinning works should consider the time elapsed between casting concrete and excavating in front of these underpins. The toe embedment considered for each underpin beyond the depth of excavation is summarised in Table 14. For the piled wall case, the structural inputs are outlined in Section 6.4.2 and comprised 600mm piles at 750mm centres with a pile length of 25m.

This assessment has considered a serviceability limit state check and has not considered the structural loading against the reinforcement capacity of the structural elements, either the underpinning, the piles, the liner walls or the existing basement walls. The design of reinforcement of these elements will be considered in future design stages.

Further work for the ground movement assessment includes the following: engagement with adjacent property owners to create party wall agreements, engagement with service and utility providers to confirm the type of service and the movement tolerance, e.g. Thames Water. A monitoring specification should be developed to understand the required scope and timeline of monitoring and estimate the costs involved so these can be included in the project budget. Additional investigation is required into the structural capacity of the existing basement walls, and additional soil testing to confirm the stiffness values adopted such as a borehole with pressure meter testing.