

Chapter 9

Numerical Modelling of Building Response to Tunnelling: 3D Case Studies

9.1 Introduction

Numerical models in civil engineering are developed as tools for use in the design and analysis of engineering problems. A key part of this development is the establishment of the validity of new methods by comparing their numerical output (whether this is predictions of loads, movements, damage or other results) with case study data. Unfortunately, access to high quality case study data for this purpose is often difficult to obtain, particularly in the geotechnical field. For the analysis of building response to tunnelling in soft ground, however, a significant amount of data have recently become available. These data were collected during the construction of the Jubilee Line Extension (JLE) project on the London Underground as part of a research project specifically focused on building response to tunnelling. The JLE works were completed in 1996 and an extensive two volume record of the research undertaken throughout the construction: *Building Response to Tunnelling - Case studies from the construction of the Jubilee Line Extension, London* (edited by

Burland, Standing and Jardine) was published in 2001. The published research includes 27 case histories, 21 of which detail the response of buildings or groups of buildings, with the remainder describing greenfield sites. From this source, the buildings chosen as case studies for this thesis are Murdoch, Clegg and Neptune Houses in Moodkee Street, Rotherhithe. These buildings, and the tunnelling beneath them, are described in detail in this chapter along with an outline of the JLE and the associated research projects. Details of the JLE construction and research are from the Burland et al. (2001) volumes; the authors of individual chapters of which are cited where appropriate. The numerical methods described in preceding chapters are used to develop a numerical model of tunnel construction at the Moodkee Street site to investigate the response of the three buildings to tunnelling. Two models are analysed, one greenfield model for comparison with observed greenfield settlements (at sites near the chosen buildings) and one with the buildings represented by equivalent masonry beams. The predictions of building response from the finite element model are compared with the observed response from the case history.

9.2 The Jubilee Line Extension and associated research projects

9.2.1 Overview of the JLE construction project

The JLE project comprised 15.5km of new twin-tube railway and 11 stations built between 1994 and 1996 to extend the Jubilee Line on the London Underground network from the existing Green Park station in the West End to Stratford in East London. For a distance of approximately 11.5km, the western portion is in twin tunnels of 4.4m inner diameter located from 20 to 30m below ground (Jardine, 2001).

The ground conditions along the route vary from London Clay in the west to the Lambeth Group further east. London Clay is a stiff to very stiff, heavily overconsolidated clay with a typical undrained shear strength in the region of 200kPa. The Lambeth Group consists of interbedded layers of stiff clays, sands, silts and gravels (Withers et al., 2001).

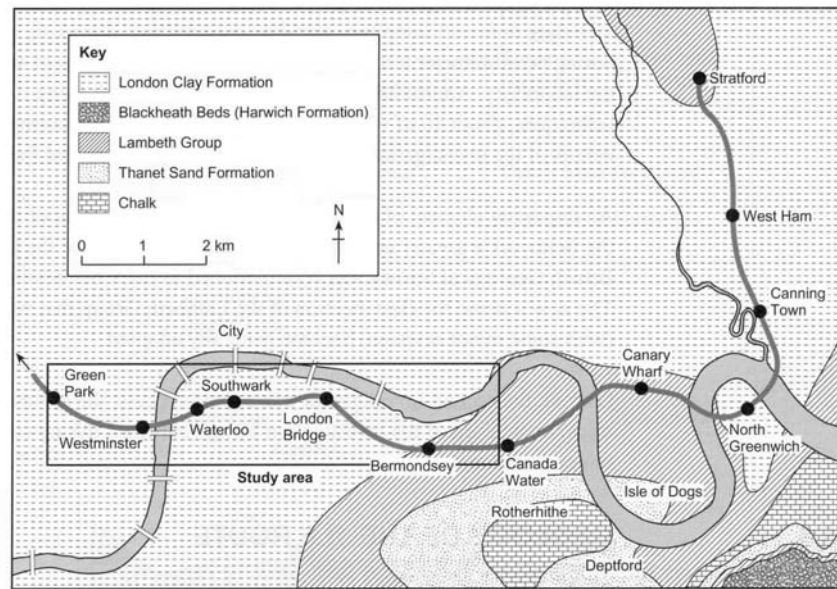


Figure 9.1: **Geological plan of the JLE route (after Withers et al., 2001)**

A description of the Lambeth Group and its engineering properties is given in section 9.3 of this thesis. Figure 9.1 shows the route outline over a plan of the geology of London and also indicates the area between Green Park and Canada Water in which case studies were conducted.

Tunnelling methods employed along the route differed with the ground conditions. In the London Clay the running tunnels were constructed using open-faced tunnelling shields with a backhoe or roadheader excavator mounted within the shield. The sprayed concrete lining method was used for the construction of the Waterloo and London Bridge station tunnel enlargements, also in the London Clay. In the section to the east of London Bridge, tunnelling was in the Lambeth Group, a much more difficult medium in which to tunnel due to its waterbearing nature and the different properties of each of its component units. Tunnelling here was undertaken using closed-face shields of both slurry and earth pressure balance (EPB) types (Mair and Jardine, 2001). Further details of the soil conditions and tunnel construction at the Moodkee Street case study site are given in section 9.3 below.

9.2.2 Overview of the JLE related research project

The two volumes of *Building Response to Tunnelling* (Burland et al., 2001) record the results of three related research projects. The main project focused on the response of buildings to the tunnelling works and was funded under the Construction Maintenance and Refurbishment (CMR) programme of the LINK scheme sponsored by (what are now called) the Department of the Environment Transport and the Regions (DETR) and the Engineering and Physical Sciences Research Council (EPSRC). The other two research projects were funded by an EPSRC grant and London Underground Limited (LUL) sponsorship and were concerned with the monitoring of surface and subsurface displacements at the St James's and Southwark Park greenfield sites (Jardine, 2001a).

The research was a collaborative effort between the scientific partner, Imperial College, the lead partner, the Construction Industry Research and Information Association (CIRIA), and initial industry partners, LUL and the Geotechnical Consulting Group. Other industry partners included: AMEC Piling, Arup, Mott MacDonald, TRL, Union Railways and Trafalgar House Technology.

The objectives of the research were to use field measurements to improve the understanding of building response to ground movements due to tunnelling, damage resulting from such movements and measures for the protection of buildings (Jardine, 2001a). Measurements were made of surface and sub-surface settlements, observed cracking and damage to buildings, horizontal strains, and full facade monitoring. The measurements were undertaken using precise surface levelling, precision tape extensometers, inclinometers and total surveying stations. A full description of the methodology is contained in Jardine (2001a).

The case study sites lie between Green Park and Canada Water as shown in figure 9.1. Among the 21 case histories of buildings in the project, a range of building and footing types are represented. Fourteen buildings are load-bearing masonry; five are reinforced concrete; and two steel and masonry framed structures. The most common footing types are pad or strip footings, with five buildings on piles and five on a raft footing. At many of the sites, compensation grouting was employed to protect the buildings. The ground

conditions can be generally categorised as either London Clay or the Lambeth Group.

The range of data collected and reported thus provides researchers with the opportunity to select appropriate case study sites for further research from the large number of case histories reported.

9.3 Moodkee Street case study buildings

9.3.1 Choice of buildings for case study

The choice of case study buildings for this thesis from the range of case histories available was made using criteria based on the objectives of the current research (as described in Chapter 3) and consideration of both the numerical methods to be validated and those in use at Oxford University. These criteria included that the buildings should be:

- Constructed of load bearing masonry to enable the use and validation of the equivalent masonry beam model for surface beams representing masonry facades;
- Built on simple strip or spread footings to avoid the modelling of pile footings which are not part of this research;
- Unprotected by compensation grouting as the modelling of this protection method is not part of this research; and
- Oriented asymmetrically with respect to the tunnel to test fully the 3D capabilities of the numerical methods developed.

The buildings chosen for this case study, based on the above criteria, are Murdoch, Clegg and Neptune Houses, a complex of three buildings in Moodkee Street, Rotherhithe. These buildings are described in detail in section 9.3.2 below and were chosen because, of the JLE case histories (and indeed, of all the published case histories reviewed by the author), they best met the criteria outlined above.

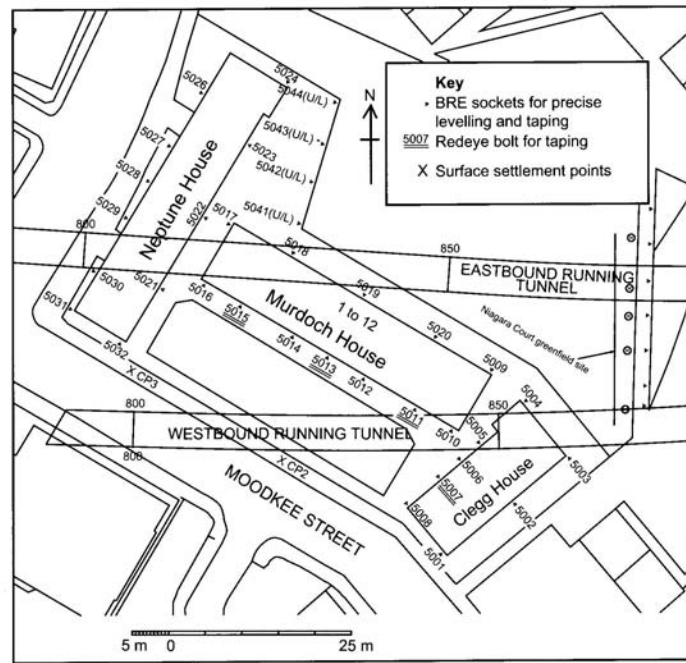


Figure 9.2: Plan of Moodkee Street case study buildings, Rotherhithe (after Mair and Taylor, 2001)

9.3.2 Description of buildings

Murdoch, Clegg and Neptune Houses are located at the corner of Neptune and Moodkee Streets, Rotherhithe, approximately 200m west of Canada Water station. They were constructed in 1931 for the employees of the South Metropolitan Gas Company. The layout of the buildings is shown in figure 9.2 including the location of the JLE tunnels under the buildings. Each building is constructed of solid load bearing brick masonry (as opposed to cavity wall construction) and is three storeys high. Murdoch and Neptune Houses are approximately 40m by 8m in plan, with Clegg House being half as long. Photographs of the buildings taken by the author in December 2004 are shown in figure 9.3.

The footings for the buildings are shown in figure 9.4. These comprise strip footings with stepped brickwork under the wall to a depth of 450mm atop a 910mm wide mass concrete strip. The concrete strip is typically 430mm thick, except for a section of Murdoch House, where part of the footing was thicker (600mm) and was constructed on a number of concrete piers (approximately 3.2m deep) where the building outline crossed the site of an old pond.



(a) Murdoch House from the South



(b) Murdoch (centre), Clegg (left) and Neptune (right) Houses from the East

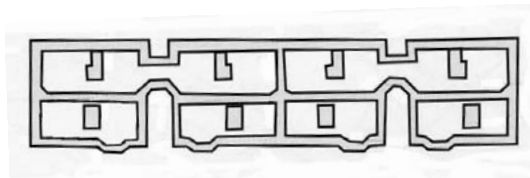


(c) Clegg House from the South East

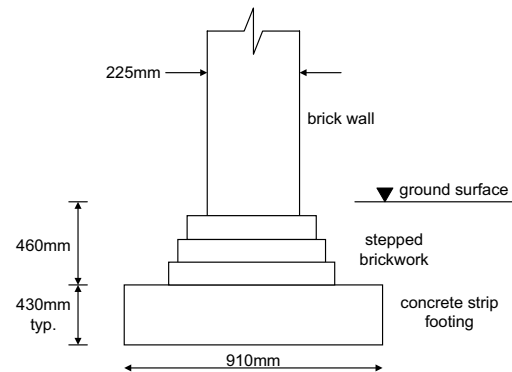


(d) Neptune House, from the North West

Figure 9.3: Moodkee street buildings



(a) Murdoch house footing plan (after Withers, 2001b)



(b) Section through footings for Moodkee Street buildings

Figure 9.4: Moodkee street building footings

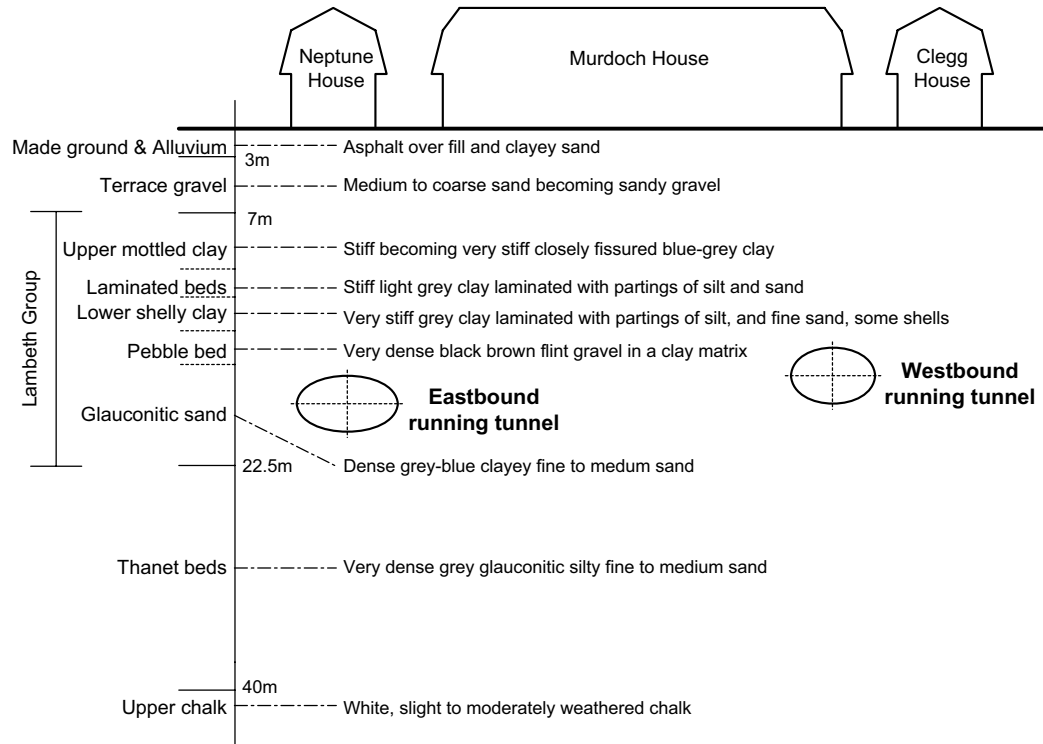


Figure 9.5: Indicative section at Moodkee Street buildings (after Withers, 2001b)

9.3.3 Ground conditions

The ground conditions at the site consist of 2m of made ground overlying 4m of Thames Gravel. The gravel lies on top of the various units of the Lambeth Group, which extends to a depth of 20 to 25m, under which lies a bed of Thanet Sand to the top of the Upper Chalk formation at a depth of approximately 40m. Figure 9.5 shows a typical section near the site, with the various stratigraphic units and the location of the tunnels relative to the buildings. The soil layer information is from Mair and Taylor (2001) and Withers (2001b) with the material descriptions and layer depths taken from borehole 417 located approximately 20m south west of Neptune House. Geotechnical properties for each of the soil layers are given by Withers et al. (2001); the soil model and material parameters used in the numerical model are described in section 9.4.

9.3.4 Settlement and damage predictions

Prior to the observed site settlements being made available, predictions were made by the Geotechnical Consulting Group (GCG) regarding the expected greenfield surface settlements, the settlements under each of the buildings and the expected damage to the structures at the site. These predictions formed a report on the structures prepared at the time of the tunnel construction and are detailed by Mair and Taylor (2001). Each of the buildings was predicted to behave almost rigidly in response to the tunnelling-induced ground movements and incur negligible damage. The actual predictions made are detailed and compared to both the observed site settlements and the settlement results from the finite element analysis of this thesis in section 9.5.

9.3.5 Tunnel construction

The JLE running tunnels were constructed beneath the Moodkee Street site using earth pressure balance shields. The tunnels are located approximately 17m below Neptune House, rising towards Canada Water station in the East, with the shields passing under the site during February (Westbound) and July (Eastbound) 1996. The shields used were 7m long Kawasaki machines of 5m outer diameter. They erected 1.2m long, 0.25m thick precast concrete bolted ring segments at the rear of the shield as a permanent tunnel lining. The finished tunnels have an outer diameter of 4.9m and an inner diameter of 4.4m.

9.3.6 Observed settlements during tunnelling

During tunnelling, on-site measurements were made of surface settlements. Methods used for settlement monitoring included precise levelling on sockets in the building walls and tape extensometer measurements between the levelling sockets (Withers, 2001b). The locations of the various monitoring points are shown in figure 9.2. The observed site settlements are compared to both predictions the finite element results of this thesis in section 9.5.

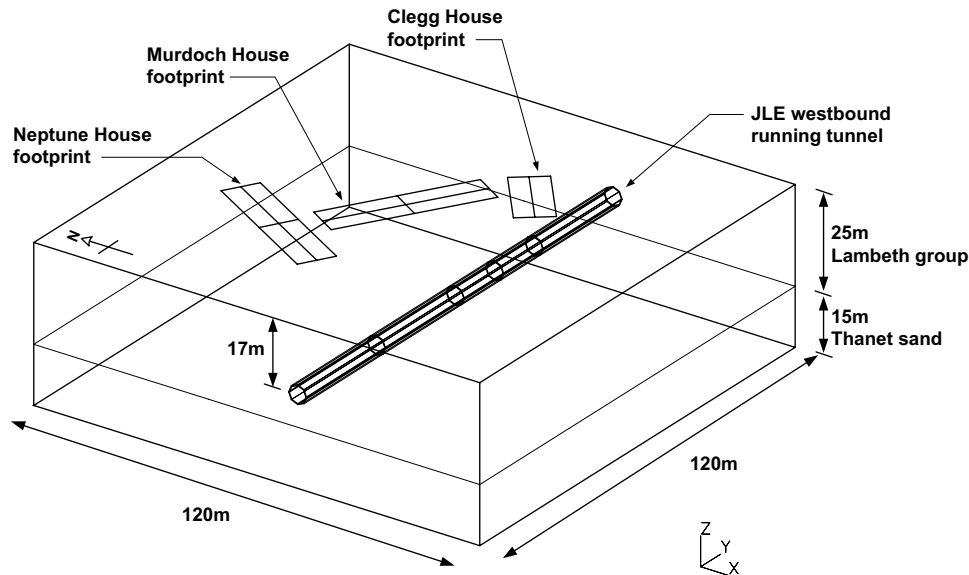


Figure 9.6: Layout of Moodkee Street case study problem

9.4 Composition of numerical model

Model layout and tunnelling

The geometry of the case study problem assumed for the numerical model is shown in figure 9.6. Only the westbound running tunnel is modelled as this was the first tunnel constructed and individual (single tunnel) data are thus available.

Figure 9.7 shows a plan of the site used for the numerical model. The location of the building footprints on the plan is detailed in table 9.1. Tunnel construction is modelled in stages with the stage boundaries representing the location of the tunnel heading during actual construction at the dates shown. The tunnel was modelled at a constant 17m depth (to the centreline) with an inner diameter to the inside of the tunnel lining of 4.4m and a 250mm thick lining. The lining was modelled as a linear elastic concrete material with properties given in table 7.1.

The footing plan assumed is shown in figure 9.8 and is a simplified version of the actual footings shown in figure 9.4.

The mesh used for the case study finite element model is shown in figure 9.9. Boundary conditions applied to the soil mesh are fully fixed displacements (x , y and z displacements

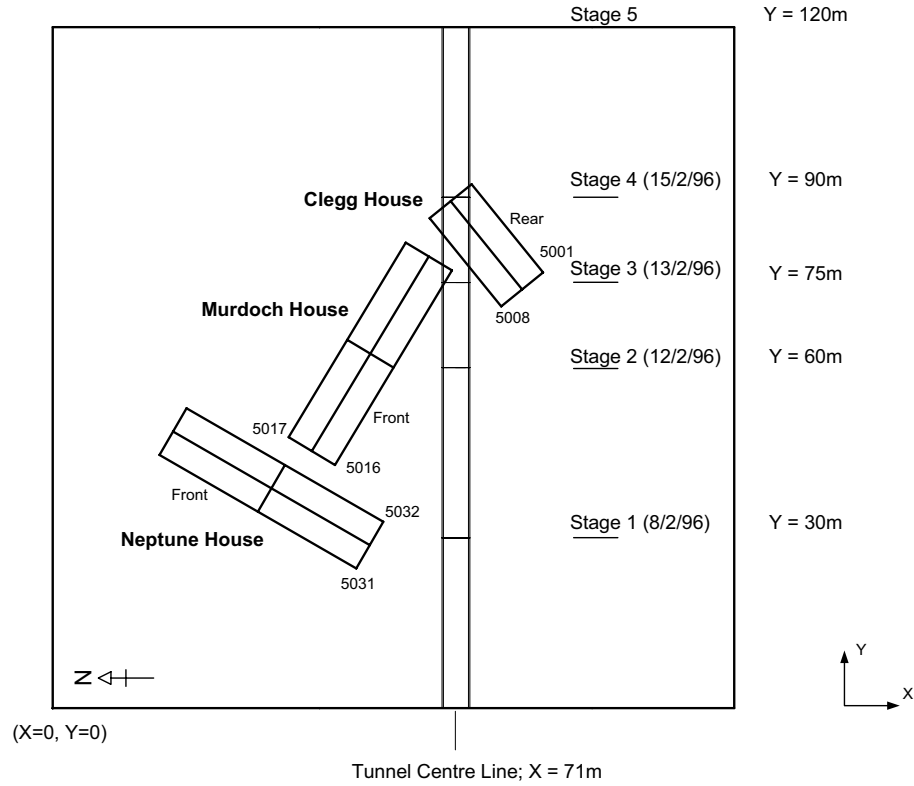
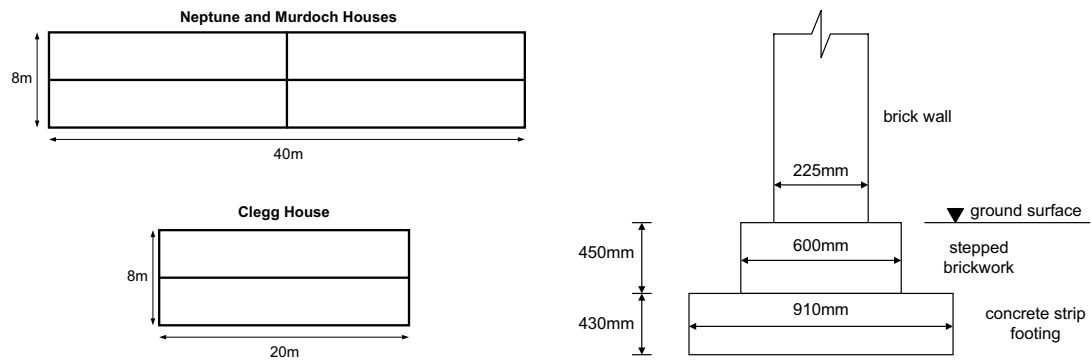


Figure 9.7: Plan of Moodkee Street case study problem



(a) Simplified footing plans assumed for numerical model

(b) Section through footings assumed for all Moodkee Street buildings

Figure 9.8: Simplified footings for model of Moodkee Street buildings

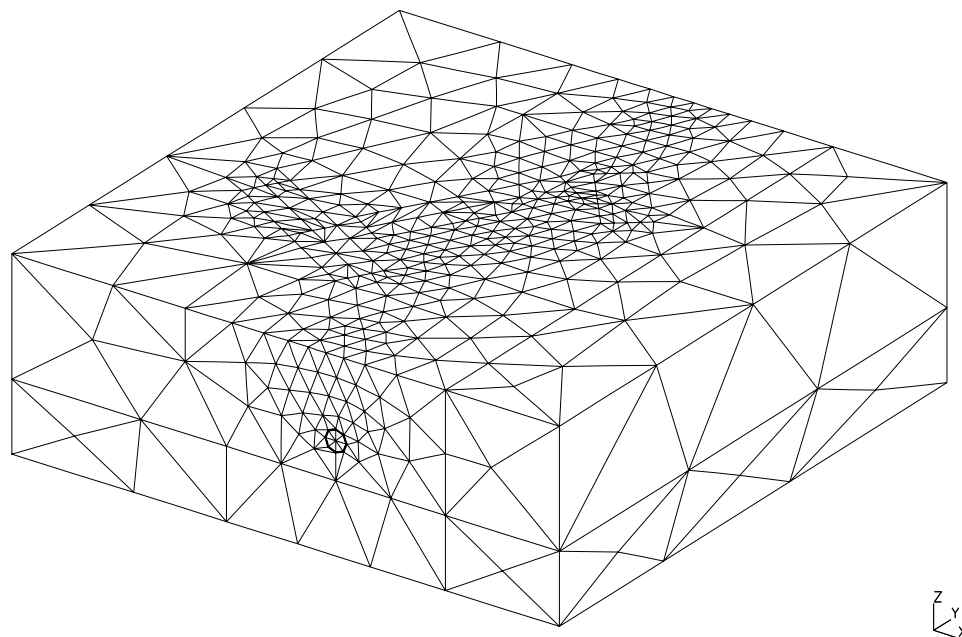


Figure 9.9: **Finite element mesh for Moodkee Street case study problem**

Table 9.1: **Location of Moodkee Street buildings for case study**

Building	Skew angle of long axis to tunnel	Coordinates of corner (X,Y)
Neptune House	120^0	5031:(53.460, 24.147)
Murdoch House	31^0	5017:(41.556, 47.750)
Clegg House	141^0	5001:(86.346, 76.767)

fixed) at the base and horizontal displacements (x or y displacements as appropriate) fixed perpendicular to each vertical soil boundary.

Soil model and material parameters

The soil profile assumed at the site is that shown in figure 9.5 above. This soil profile is approximated for modelling purposes by the profile shown in figure 9.6 comprising 25m of Lambeth Group soil overlying 15m of Thanet Sand with the base of the soil mesh taken at the top of the Upper Chalk formation at a depth of 40m.

The soil model used for both soil layers is the nested yield surface model described in section 7.4. This model was used as it is important to be able to model the non-linearity of the soil response at small strains. This model is the only soil model capable of capturing

Table 9.2: Material properties for Lambeth Group layer

Property	Withers (2001)	Other calc's/comments/ref's	Value
γ	15-21kN/m ³	Made ground: 15-19kN/m ³ , alluvium: 16-20kN/m ³ , terrace gravel: 19-20kN/m ³ , Lambeth Group: 19-21kN/m ³	20kN/m ³
s_{u0}	s_{u0} = 15-70kPa	Range for made ground. Surface value chosen as approx average	40.0kPa
μ	s_{u0} = 100-400kPa	Group range. Increase in s_u with depth chosen appropriate for depth. At $z=20$ m with $\mu=4.0$ kPa/m, $s_{u0}=120$ kPa thus within range	4.0kPa/m
G_0^s	E_u/s_u = 1500	From Figure 5.9 of Withers et al. (2001). $G_0^s/s_u=1500/2(1+\nu)$ or $G_0^s=500s_u$ (rigidity value from Wisser (2002))	2.0×10^4 kPa
ω	Nil	$\omega = G_0^s/10$ (Wisser (2002))	2.0×10^3 kPa/m
ν	Nil	Nil	0.49

Yield surface parameters								
Surface	c'_α	g'_α	Surface	c'_α	g'_α	Surface	c'_α	g'_α
1	0.020	0.900	4	0.100	0.300	7	0.300	0.100
2	0.040	0.750	5	0.150	0.200	8	0.500	0.050
3	0.060	0.500	6	0.200	0.150	9	0.700	0.025

this small strain non-linear response available in the OXFEM program. The use of a model developed for cohesive soils for the non-cohesive layers at this site (the Glauconitic sand and the Thanet sand) was felt to give a more accurate response than a non-cohesive model (such as a linear elastic-frictional model) with no facility for modelling non-linearity at small soil strains. The properties used in the model for the Lambeth Group and Thanet Sand layers are described below.

The Lambeth Group properties are based on those given by Withers et al. (2001) for all components of the Group using the simplifying assumption that the upper 25m of model soil represents only the Lambeth Group and that this layer comprises only one material. The values selected were derived from the literature as outlined in table 9.2. The yield surface parameters are those presented by Houlsby (1999). The ratio of horizontal to vertical stress is chosen as 1.0 for all soil layers.

The properties chosen for the Thanet Sand model layer are derived from information presented by Kovacevic et al. (2001) and Powrie and Batten (2000). The use of a model

Table 9.3: Material properties for Thanet Sand layer

Property	Reference value	Reference/calc's/comments	Value
γ	22kN/m ³	Kovacevic et al. (2001)	22kN/m ³
G_0^s	$E' = 300\text{MN/m}^2$	Powrie and Batten (2000). E_u assumed to be slightly higher in range 300-320 MN/m ² . Then $G_0^s = E_u/2(1+\nu)$	$1.05 \times 10^5\text{kPa}$
ω	Nil	$\omega = G_0^s/10$ (Wisser (2002))	$1.05 \times 10^4\text{kPa/m}$
s_{u0}	G_0^s	$s_{u0} = G_0^s/500$ (Wisser (2002))	210kPa
μ	Nil	$\mu = s_{u0}/10$ (Wisser (2002))	21kPa/m
ν	Nil	Nil	0.49

Yield surface parameters								
Surface	c'_α	g'_α	Surface	c'_α	g'_α	Surface	c'_α	g'_α
1	0.020	0.900	4	0.100	0.300	7	0.300	0.100
2	0.040	0.750	5	0.150	0.200	8	0.500	0.050
3	0.060	0.500	6	0.200	0.150	9	0.700	0.025

developed for cohesive soils for such material is not the most realistic approach, but it is felt that this is worthwhile for the ability to include non-linear behaviour at small strains. The values selected were derived from the literature as outlined in table 9.3

Properties for beams representing buildings

The dimensions used for each of the facades to determine their equivalent beam properties are given in figures 9.10 to 9.13. These dimensions are based on site measurements and photographs (not construction drawings). In addition, it is assumed that the end walls of all buildings have the dimensions and layout shown in figure 9.13. In reality each building has one end wall of approximately 9.5m long and three sets of windows. It is not felt that this simplification significantly affects the results of the numerical model relative to the case study.

The properties for each of the walls modelled are given in table 9.4. The properties are derived using the methods described in Chapters 4 and 5 of this thesis where definitions of the various properties are also given. A smooth base is assumed for the determination of the properties as it is felt that strip footings will allow some movement between soil and footing at foundation level, unlike a piled raft for example. The short interior walls

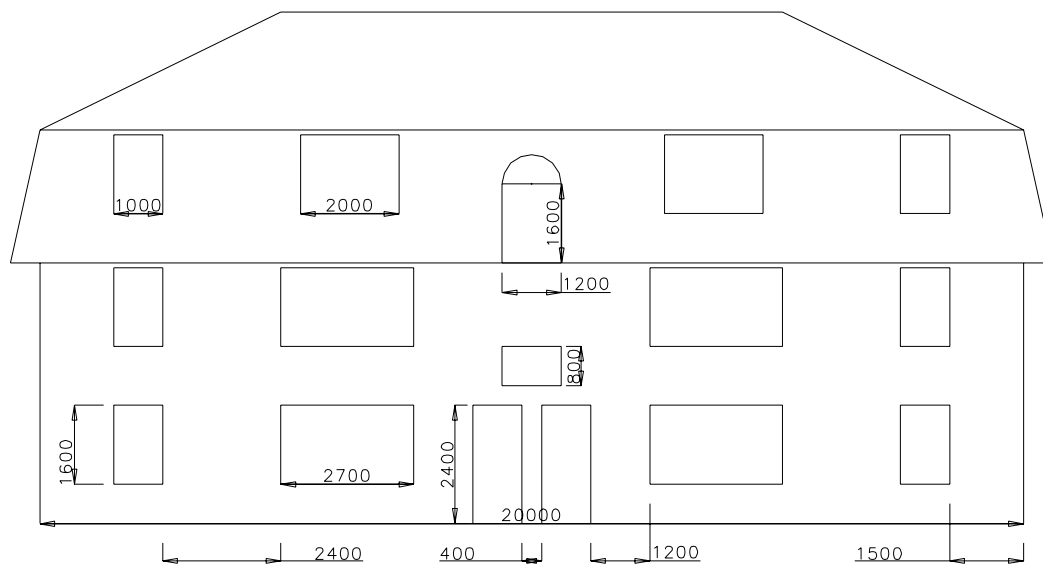


Figure 9.10: Clegg House front dimensions (mm)

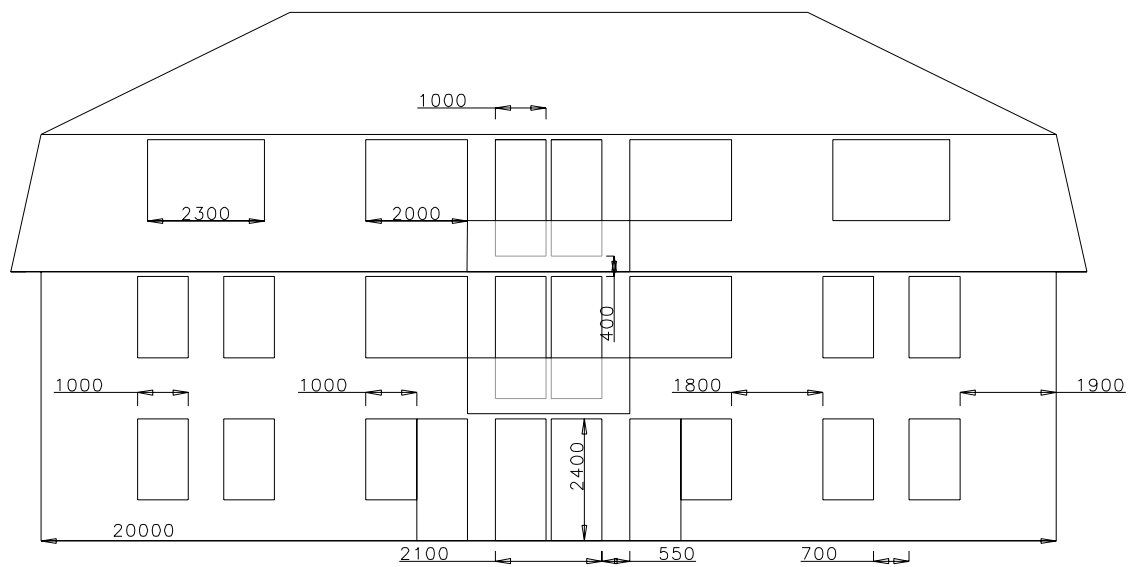
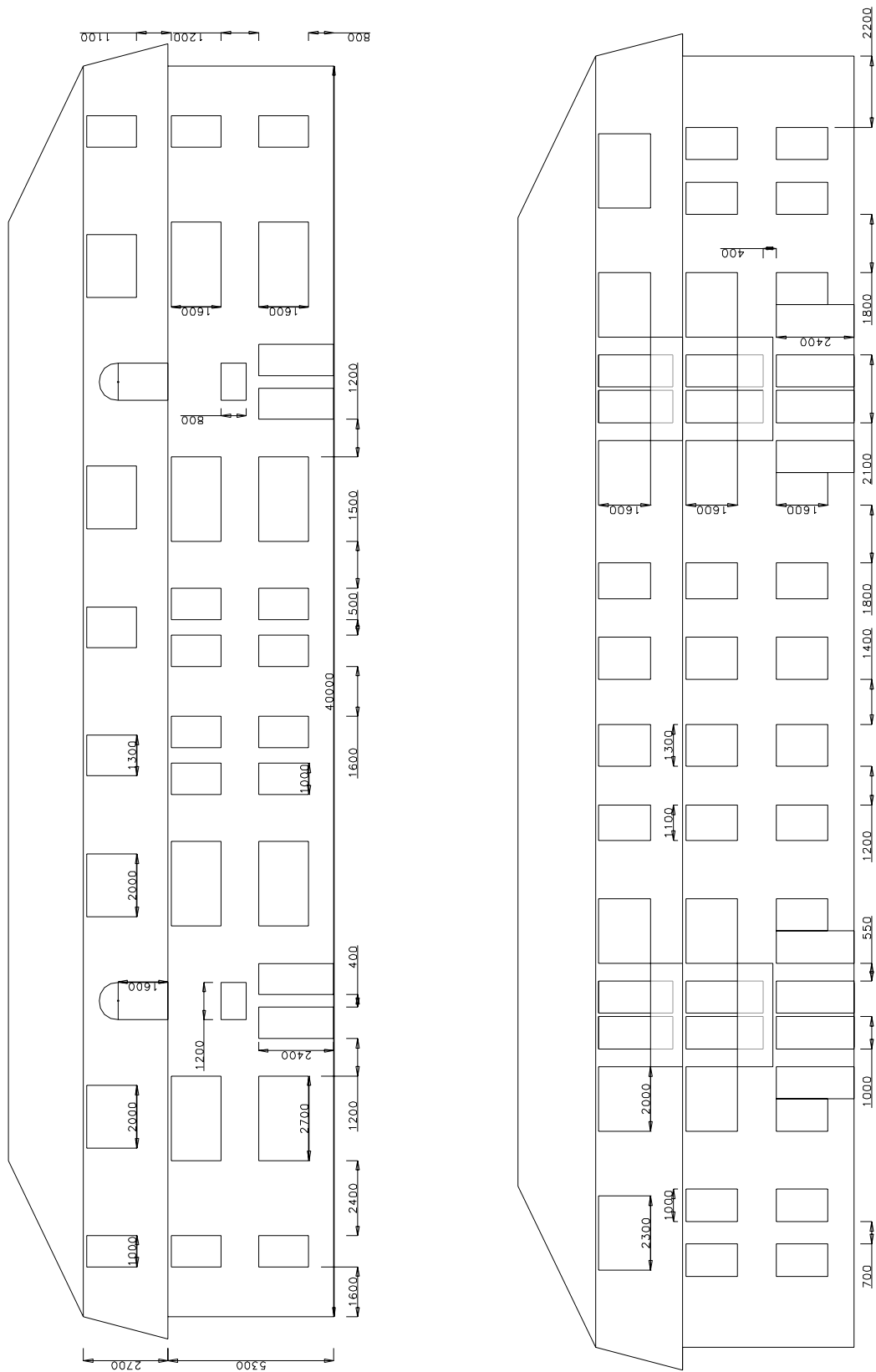


Figure 9.11: Clegg House rear dimensions (mm)



(a) Neptune/Murdoch House front dimensions

(b) Neptune/Murdoch House rear dimensions

Figure 9.12: Neptune/Murdoch House front and rear dimensions (mm)

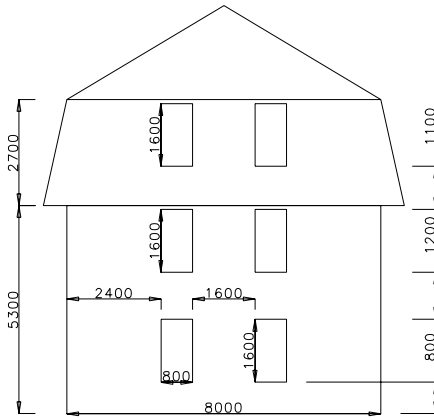


Figure 9.13: All building end dimensions (mm)

are assumed to have the same properties as the end walls and the long interior walls are assumed to have the same properties as the front facade of the respective building. The footings are included in the assessment of the properties of the beams to represent the walls. The κ_{crit} and f_b values for the beams are those deemed to be most appropriate from the discussions in Chapter 8.

The walls and the footings are assumed to have a self weight of $20kN/m^3$ which contributes to the loads on the beams. In addition a dead load for floor and roof loading of $0.5kN/m^2$ is assumed. Self weight and dead loads are calculated using the building dimensions and are applied at the nodal points on the surface beams. The loads differ from node to node due to openings in the facades.

Calculation procedure

The calculation is undertaken in stages in a similar fashion to the procedure described for the example analyses in section 8.2.1. The volume loss used in the excavation is 0.8%, which is the reported observed volume loss for the westbound tunnel (Withers, 2001b). Table 9.5 details the calculation stages for the analysis.

Table 9.4: Material properties for equivalent masonry surface beams

Clegg House front wall and long interior wall				
E	G	A^*	I_{in}^*	I_{out}^*
$1.000 \times 10^7 \text{kPa}$	$4.167 \times 10^6 \text{kPa}$	1.336m^2	13.631m^4	$5.637 \times 10^{-3} \text{m}^4$
J	k	γ	κ_{crit}	f_b
$2.117 \times 10^{-2} \text{m}^4$	0.85	20.0kN/m^3	1.00×10^{-5}	0.01
Clegg House rear wall				
E	G	A^*	I_{in}^*	I_{out}^*
$1.000 \times 10^7 \text{kPa}$	$4.167 \times 10^6 \text{kPa}$	1.189m^2	12.712m^4	$5.016 \times 10^{-3} \text{m}^4$
J	k	γ	κ_{crit}	f_b
$1.884 \times 10^{-2} \text{m}^4$	0.85	20.0kN/m^3	1.00×10^{-5}	0.01
Murdoch/Neptune Houses front wall and long interior wall				
E	G	A^*	I_{in}^*	I_{out}^*
$1.000 \times 10^7 \text{kPa}$	$4.167 \times 10^6 \text{kPa}$	1.725m^2	17.998m^4	$7.277 \times 10^{-3} \text{m}^4$
J	k	γ	κ_{crit}	f_b
$2.733 \times 10^{-2} \text{m}^4$	0.85	20.0kN/m^3	1.00×10^{-5}	0.01
Murdoch/Neptune Houses rear wall				
E	G	A^*	I_{in}^*	I_{out}^*
$1.000 \times 10^7 \text{kPa}$	$4.167 \times 10^6 \text{kPa}$	1.574m^2	16.874m^4	$6.166 \times 10^{-3} \text{m}^4$
J	k	γ	κ_{crit}	f_b
$2.316 \times 10^{-2} \text{m}^4$	0.85	20.0kN/m^3	1.00×10^{-5}	0.01
All Houses end walls and short interior walls				
E	G	A^*	I_{in}^*	I_{out}^*
$1.000 \times 10^7 \text{kPa}$	$4.167 \times 10^6 \text{kPa}$	0.639m^2	5.847m^4	$2.696 \times 10^{-3} \text{m}^4$
J	k	γ	κ_{crit}	f_b
$1.013 \times 10^{-2} \text{m}^4$	0.85	20.0kN/m^3	1.00×10^{-5}	0.01

Table 9.5: Calculation stages for Moodkee Street case study analysis

Stage	No. Steps	Description
1	15	Application of building weight and initial soil stresses
2	10	Reset displacements and strains to zero
3	15 GF / 30 EMB	Tunnel construction stage 1: 00.0 - 30.0m
4	15 GF / 30 EMB	Tunnel construction stage 2: 12.5 - 60.0m
5	15 GF / 30 EMB	Tunnel construction stage 3: 25.0 - 75.0m
6	15 GF / 30 EMB	Tunnel construction stage 4: 30.0 - 90.0m
7	15 GF / 30 EMB	Tunnel construction stage 5: 35.0 - 120.0m

9.5 Numerical modelling results

9.5.1 Introduction

Two finite element analyses are undertaken in this section using the model described above. A greenfield analysis is undertaken and the results compared to observed JLE project greenfield settlements at a number of sites near the Moodkee Street location. A combined analysis with the buildings included is then undertaken, with buildings represented by equivalent masonry beams. Observed building displacements are compared to the finite element results as well as the predicted building responses from the published JLE studies.

9.5.2 Greenfield analysis

The results of the greenfield finite element analysis are shown in figures 9.14 and 9.15 for each tunnelling stage as construction progresses. A transverse section of the settlement profile after the final tunnelling stage is shown in figure 9.16 along with observed settlements at a number of monitored JLE greenfield sites. Table 9.6 contains a description of each of the greenfield sites which were all monitored for settlements due to EPBM tunnelling through Lambeth Group soils and thus provide a good reference. The monitoring and the observed displacements at the sites are fully described by Withers (2001a).

The finite element displacements (obtained using an input volume loss of 0.8% as noted in section 9.4) can be seen to be shallower and wider than a Gaussian curve calculated using the same volume loss and a trough width parameter of 0.45. This is a common trait of finite element models of greenfield sites as discussed in sections 2.3.2 and 2.4. The maximum finite element settlement closely matches the maximum settlements from each of the observed case study greenfield sites, and in the central trough region exhibits a similar shape, in particular to the Old Jamaica Road settlements. The shape of the trough, however, is shallower and wider than the Niagara Court greenfield site. This site is the closest to the Moodkee Street buildings and the observed trough there closely matches the Gaussian curve. This indicates that the results from the combined finite element analyses

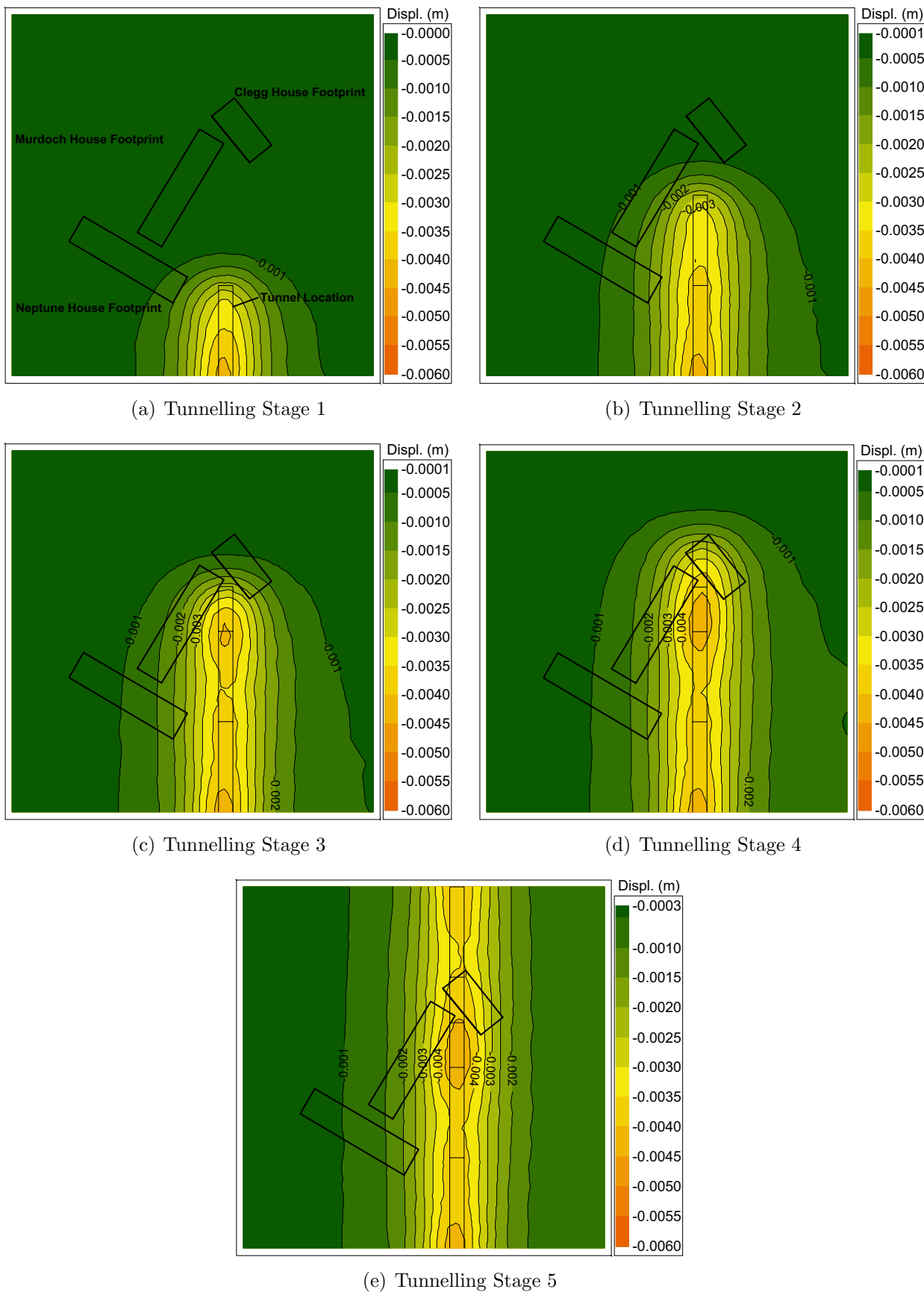


Figure 9.14: Moodkee Street FE Model Greenfield run: Surface displacement contours

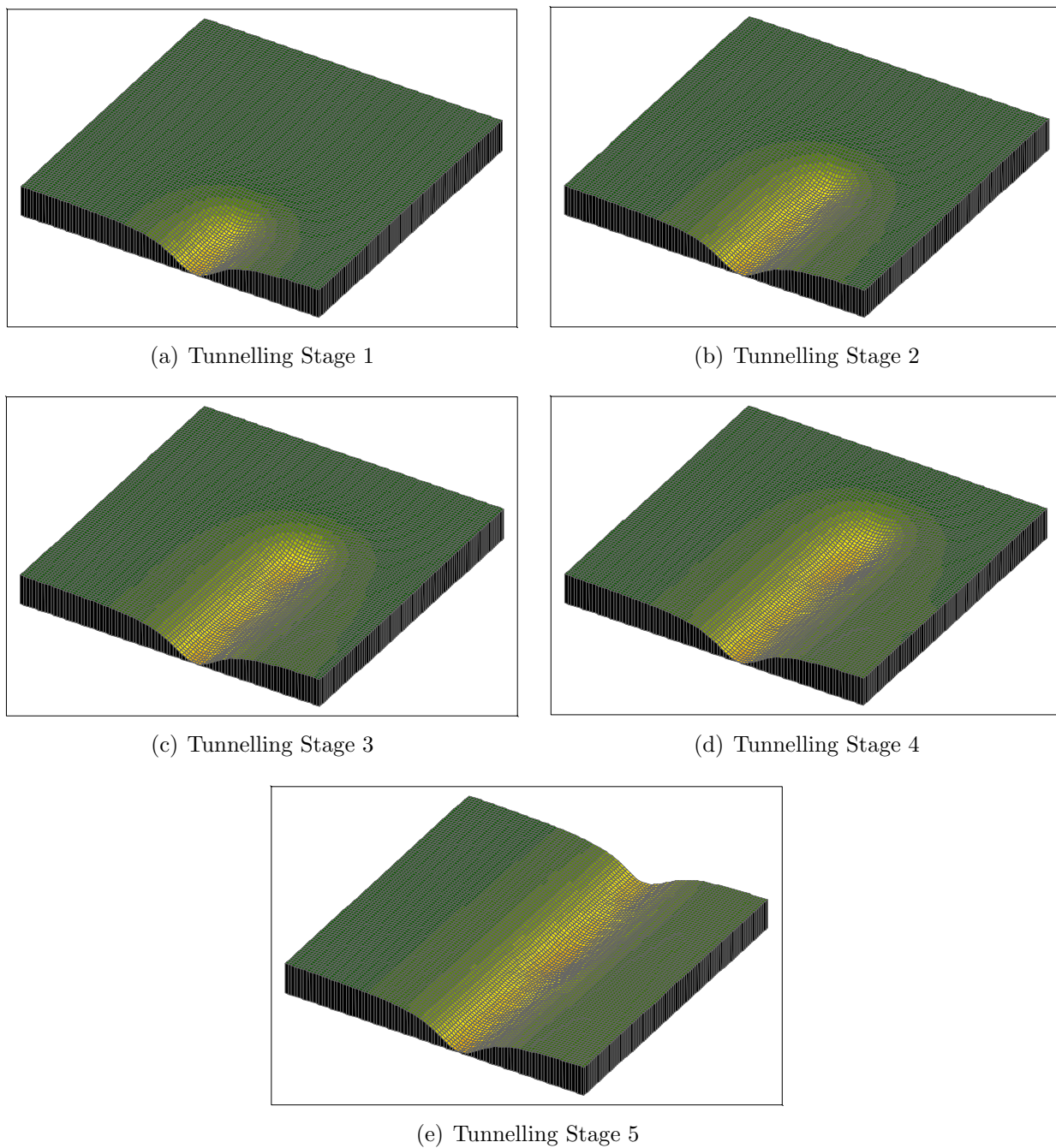


Figure 9.15: Moodkee Street FE Model Greenfield run: 3D Surface profile

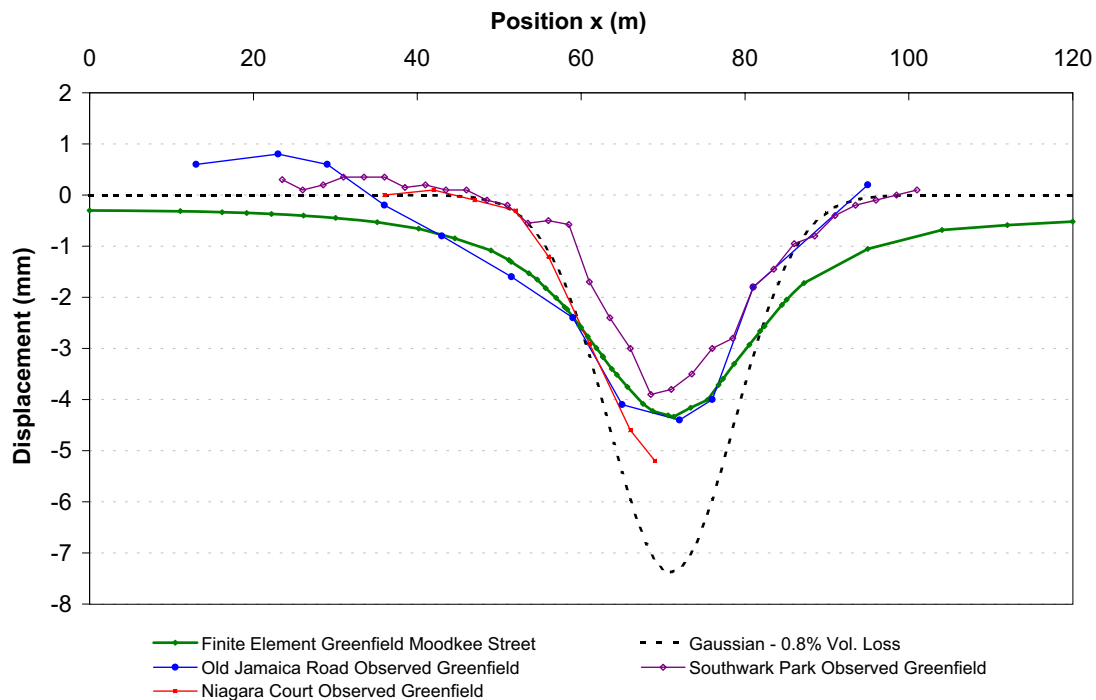


Figure 9.16: JLE observed greenfield displacements and finite element results (transverse slice at Y=60m)

Table 9.6: Observed greenfield sites presented in JLE case studies

Site	Source	Description
Niagara Court	Withers (2001a)	Located 10m east towards Canada Water Station from Clegg House in similar ground conditions. Settlements were monitored at the base of a low (0.8m) brick wall.
Southwark Park	Withers (2001a)	Located 300m west of the Moodkee Street site. Ground conditions are generally similar to the site although the tunnel is slightly deeper in the Glauconitic Sand layer. Regularly spaced survey points located on open grass.
Old Jamaica Road	Withers (2001a)	Located approximately 1km west of the Moodkee Street site. Ground conditions are similar but with the level of the tunnels slightly deeper and in the lower clayey beds of the Lambeth Group. An irregular line of survey points was situated on asphalt and concrete pavement slabs.

with buildings will be based on corresponding greenfield finite element settlements which exhibits less severe curvatures than were observed in the field. It should also be noted, however, that the volume loss calculated for the observed JLE greenfield sites was in the range of 0.4-0.6%; the shapes of the observed troughs thus differ from the finite element trough which exhibits small displacements at locations remote from the tunnel centre line which contribute to its 0.8% volume loss.

It is interesting to compare the finite element results above to those published by Kovacevic et al. (2001) for a finite element model of the Southwark Park greenfield site. The authors present a range of displacements resulting from the use of volume loss values of 0.5, 1.2 and 2.0 per cent. For the westbound tunnel the maximum settlement values are 3.3mm, 9.3mm and 16.5mm respectively. The maximum settlement of 4.3mm for the Moodkee Street site finite element model presented in this thesis compares favourable with these Southwark Park results when considering that a volume loss of 0.8% was used here.

9.5.3 Combined analyses with buildings

The results of the finite element analysis including equivalent masonry beam models of the three buildings are shown in plan and three dimensions in figures 9.17 and 9.18 respectively. Figures 9.19 to 9.24 show comparisons of the finite element displacements along each of the building walls, with the observed as well as the predicted building displacements as reported by Withers (2001b). The predicted greenfield and the finite element greenfield settlements (along each building wall) are also shown for comparison.

The predicted settlements were made by the Geotechnical Consulting Group (GCG) as part of the JLE research project and are detailed by Mair and Taylor (2001). The method of predicting the settlements and building response followed the approach described in section 2.5 including the relative stiffness method (Potts and Addenbrooke, 1997). The predictions are reproduced in this thesis from the comparisons given by Withers (2001b) and the predictions by Mair and Taylor (2001). A summary of the building settlements and results from the finite element model and the observed data is included in table 9.7.

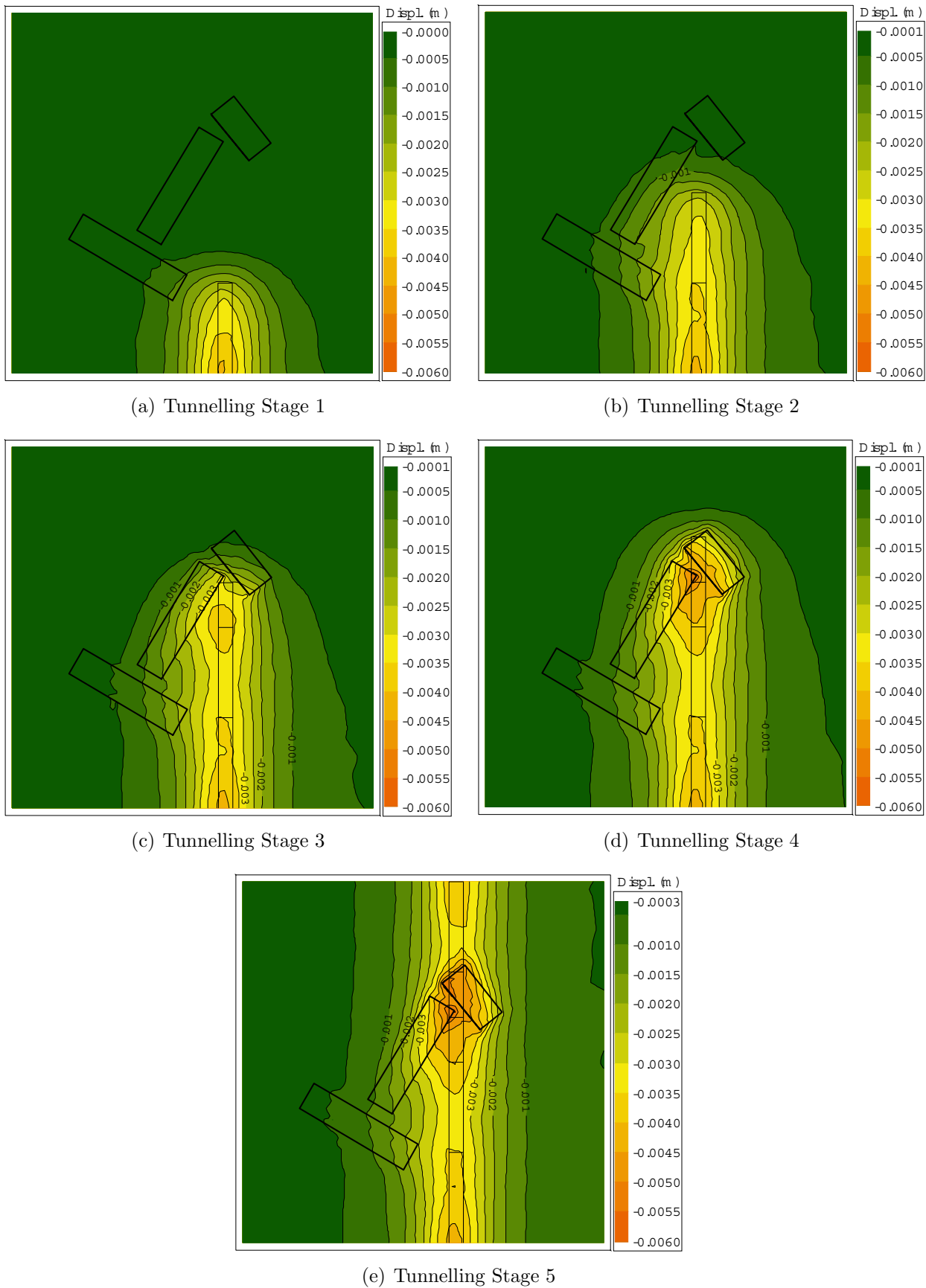
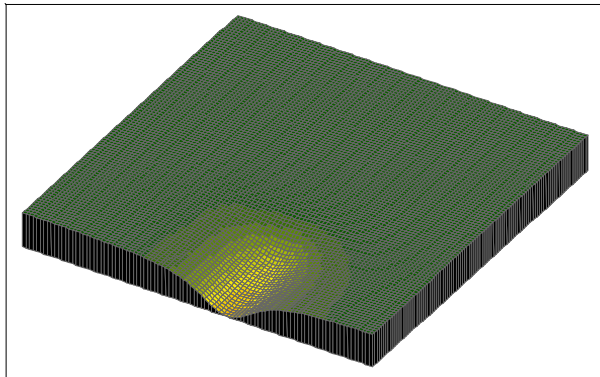
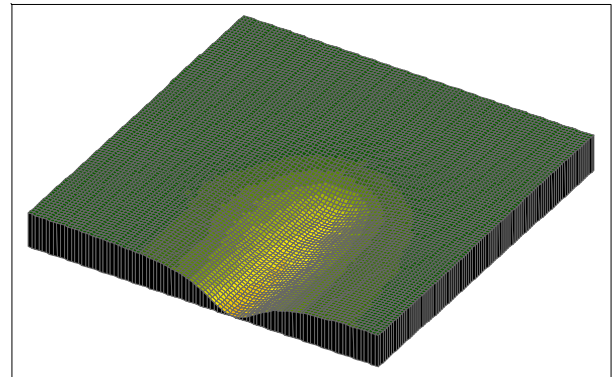


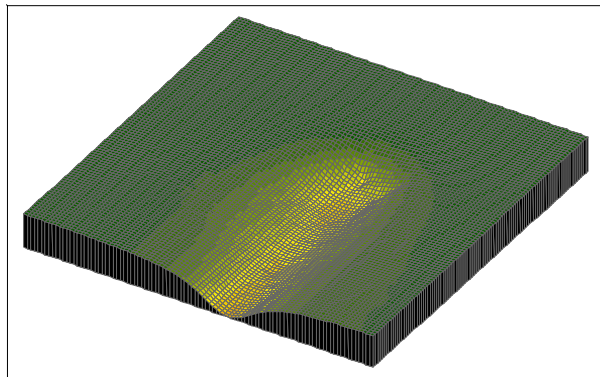
Figure 9.17: Moodkee Street FE Model with Equivalent Masonry Beams: Surface displacement contours



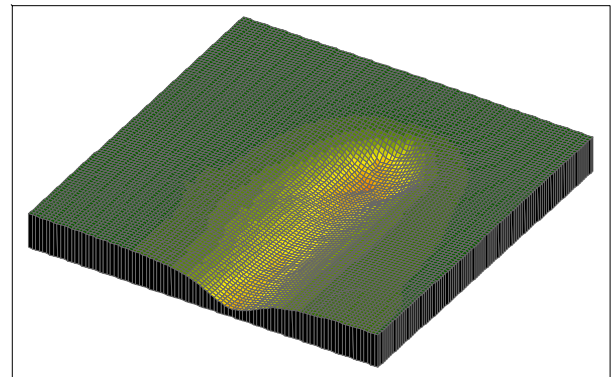
(a) Tunnelling Stage 1



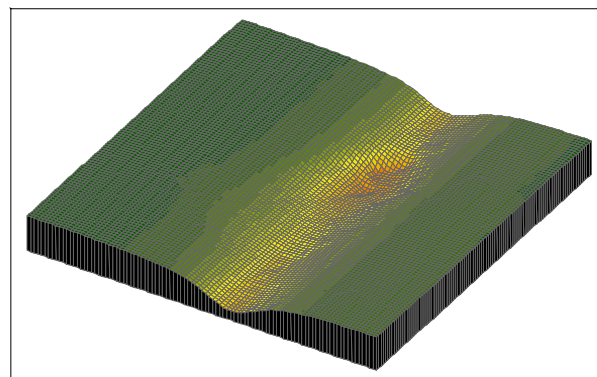
(b) Tunnelling Stage 2



(c) Tunnelling Stage 3



(d) Tunnelling Stage 4



(e) Tunnelling Stage 5

Figure 9.18: Moodkee Street FE Model with Equivalent Masonry Beams: 3D Surface profile

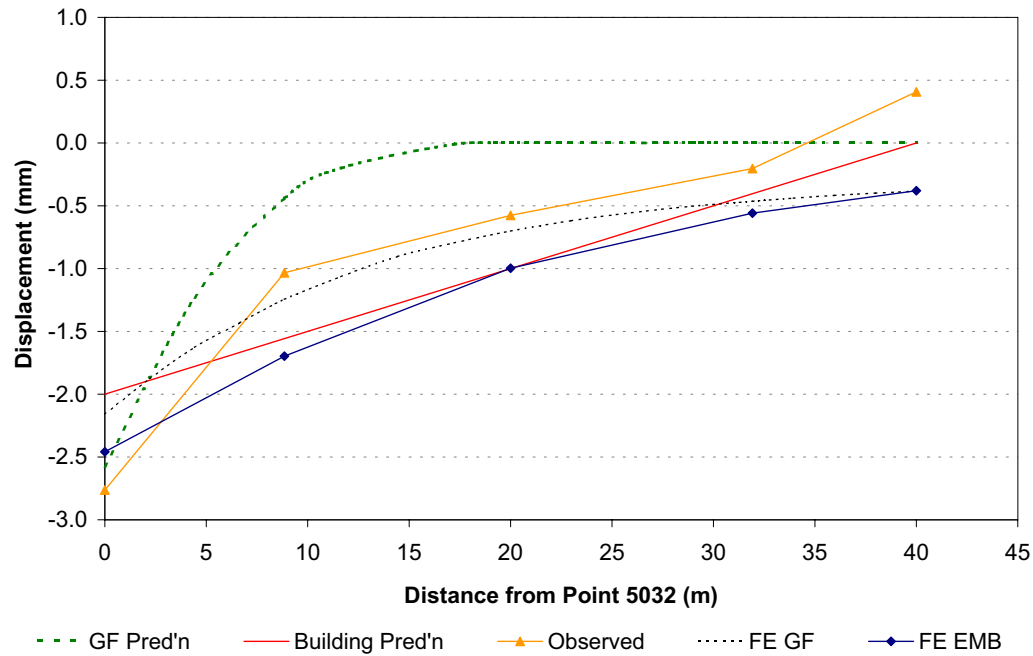


Figure 9.19: Predicted, Observed and Finite Element Settlements for Neptune House (Rear)

Neptune House

Figures 9.19 and 9.20 show the settlements of Neptune House. The building is wholly in hogging mode with a maximum observed settlement of almost 3mm. Neptune House was predicted to behave almost rigidly in response to the tunnelling using this method. The observed settlement magnitudes were very similar to those predicted, but the response of the building was more flexible. Both the front and rear facades acted more flexibly than predicted in hogging with the maximum deflection ratio (Δ/L) value being 0.003%. The shape of the building displacement profile can be seen to follow a broadly similar shape to that of the assumed greenfield profile for the site.

The finite element model results using the equivalent masonry beams show the building acting flexibly in hogging, following the greenfield profile (of the finite element greenfield run). The curvature of the building (Δ/L of 0.001%) is less severe than the observed curvature of the building for both the front and rear facades, but the hogging nature of the response is very similar. The lower curvature in the FE building response is not surprising considering the difference between the finite element and the predicted greenfield

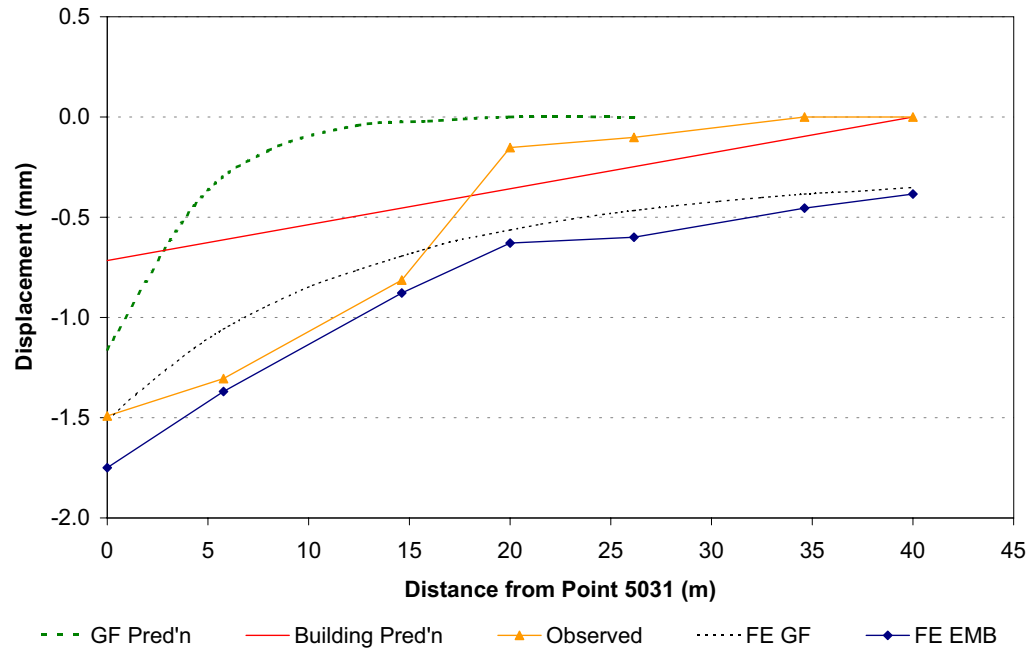


Figure 9.20: **Predicted, Observed and Finite Element Settlements for Neptune House (Front)**

curves; the finite element greenfield response being less severe. This leads to the finite element model of the building also having a lower curvature. The finite element building response relative to the finite element greenfield response compares favourably, however, to the observed building response relative to the predicted greenfield settlement. The finite element model leads to a ‘negligible’ predicted damage category which agrees with the fact that there was no damage recorded as a result of the JLE tunnelling (Withers, 2001b).

Murdoch House

Murdoch House was also predicted to respond in a rigid manner as shown in figures 9.21 and 9.22. The observed building response was, however, more flexible than assumed although the magnitudes of the maximum settlement were as predicted.

The rear facade of the building behaved flexibly in hogging as shown, with a deflection ratio of 0.005%, closely following the assumed greenfield profile for the site. The finite element response for the rear facade was less severe, but still exhibited a hogging curvature (Δ/L of 0.001%), following a similar profile to the finite element greenfield prediction.

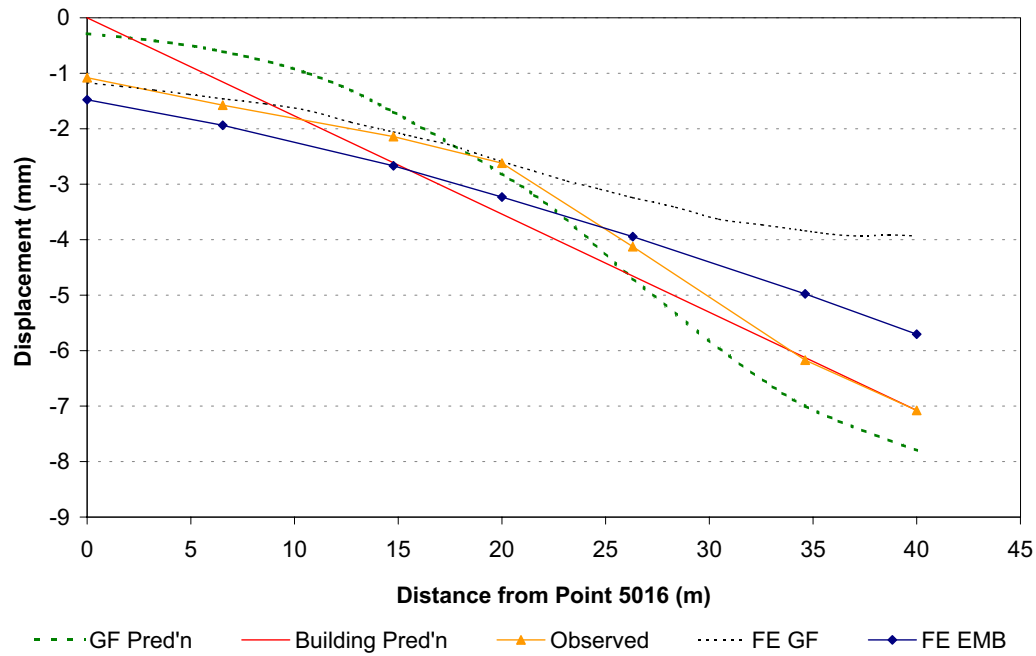


Figure 9.21: Predicted, Observed and Finite Element Settlements for Murdoch House (Front)

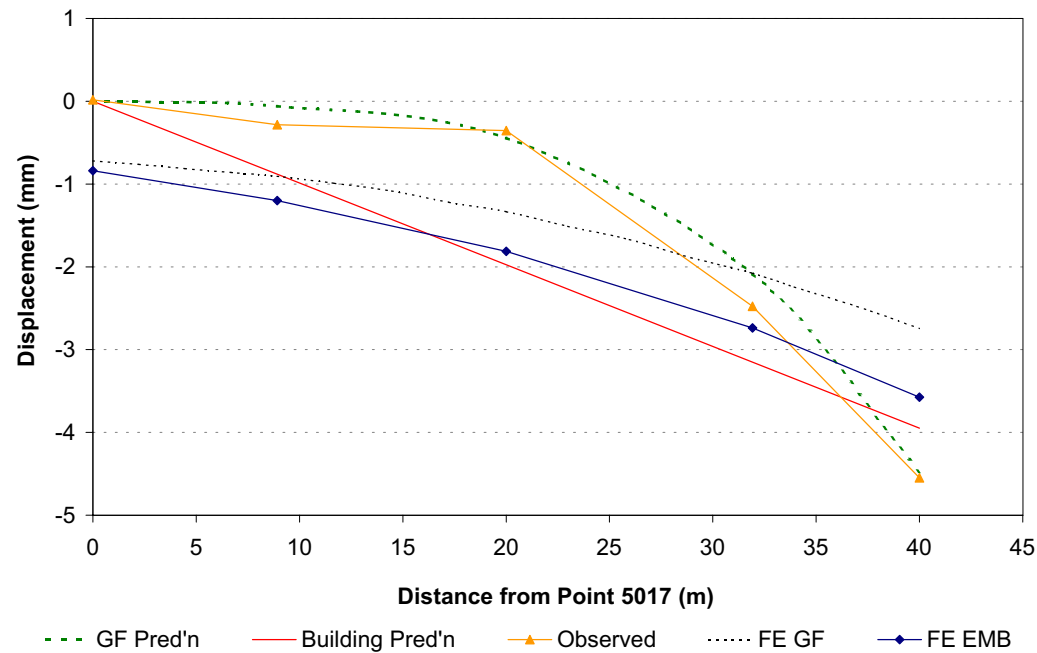


Figure 9.22: Predicted, Observed and Finite Element Settlements for Murdoch House (Rear)

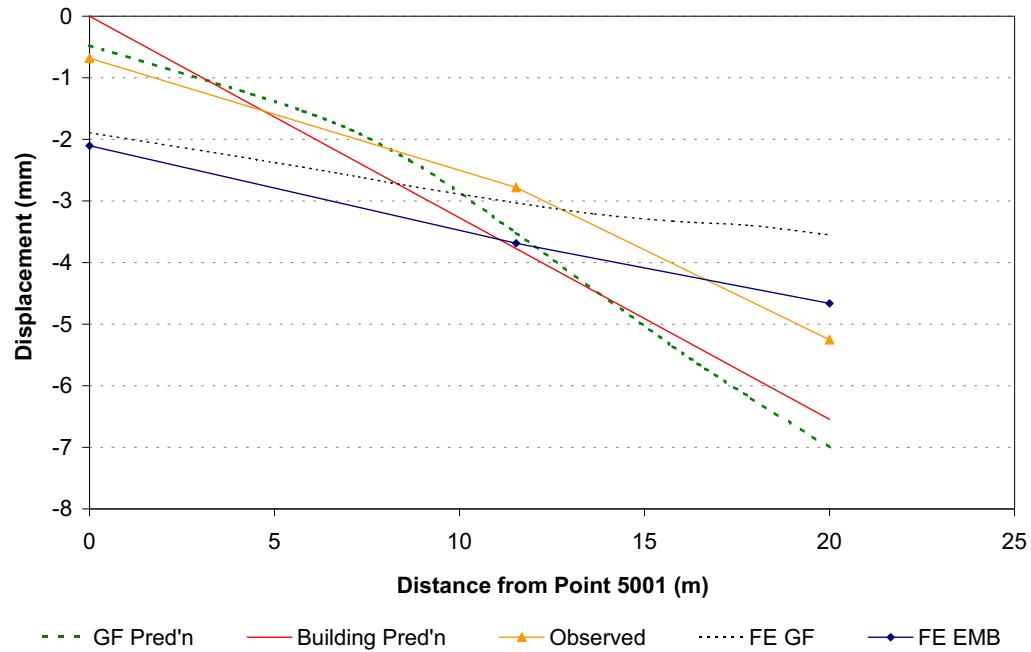


Figure 9.23: **Predicted, Observed and Finite Element Settlements for Clegg House (Rear)**

The front of the building exhibited a slight hogging curvature in the final stage with a deflection ratio of 0.004%. This compares to the finite element response where the deflection ratio was 0.001%. The 0.007% maximum deflection ratio noted in table 9.7 was a temporary curvature during an intermediate tunnelling stage. This situation was not replicated in the finite element model with the maximum intermediate deflection ratio being 0.001%.

For both the front and rear facades, the finite element response can be seen to be less severe than the observed response. The building response does, however, follow a similar shape to the finite element greenfield profile in the same way that the observed building response was more flexible than predicted and followed the predicted greenfield surface profile in hogging. Both the observed and finite element building responses exhibited negligible horizontal tensile strains and damage.

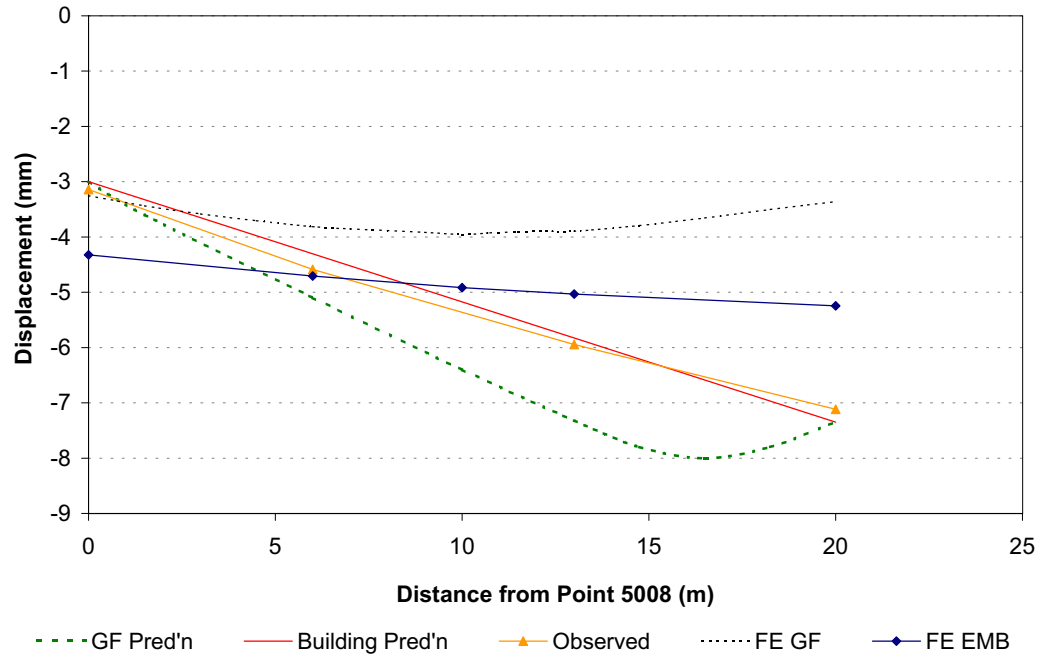


Figure 9.24: Predicted, Observed and Finite Element Settlements for Clegg House (Front)

Clegg House

The predicted and observed responses for Clegg House, along with the finite element model results are shown in figures 9.23 and 9.24. The observed response closely matched the rigid predicted response, with negligible building curvatures developing and with very similar maximum settlements. The finite element response for both front and rear facades was also essentially rigid, with negligible curvatures developing. The shape of the model results, however, differ from the observed profile, with the finite element building response being rigid, but in relation to the finite element greenfield settlement profile which differs from the predicted greenfield profile. Both the observed and finite element building responses exhibited negligible horizontal tensile strains and damage.

9.5.4 Numerical Modelling Summary

The finite element displacements agree reasonably well with the observed displacements. In all situations where the buildings were observed to exhibit hogging curvature in response

Table 9.7: Moodkee Street Case Study: Observed and Finite Element Model Comparison

Data	Murdoch		Neptune		Clegg	
	Front	Rear	Front	Rear	Front	Rear
Obs. Max. Settlement (mm)	7	5	2	3	7	5
FE Max. Settlement (mm)	6	4	2	3	5	5
Obs. Max. Δ/L Hog (%)	0.007	0.005	0.001	0.003	Neg.	Neg.
FE Max. Δ/L Hog (%)	0.001	0.001	0.001	0.001	Neg.	Neg.
Obs. Max. Δ/L Sag (%)	Neg.	Neg.	N/A	N/A	Neg.	Neg.
FE Max. Δ/L Sag (%)	N/A	N/A	N/A	N/A	Neg.	Neg.
Obs. Horiz. Tensile Strain ($\mu\epsilon$)	Neg.	Neg.	Neg.	Neg.	Neg.	Neg.
FE Horiz. Tensile Strain ($\mu\epsilon$)	Neg.	Neg.	Neg.	Neg.	Neg.	Neg.
Obs. Damage Category	Neg.	Neg.	Neg.	Neg.	Neg.	Neg.
FE Damage Category	Neg.	Neg.	Neg.	Neg.	Neg.	Neg.

to the JLE tunnelling, the finite element model (with buildings represented by equivalent masonry beams) also predicted a hogging response. In sagging, the buildings in the finite element model acted in a rigid manner similar to the Moodkee Street buildings.

The severity of the hogging curvatures in the finite element model, however, were generally less than were observed in practice. The fact that the curvatures of the finite element greenfield profile are less severe than the observed greenfield settlements, means that the finite element building predictions could never exactly match the observed building predictions. However, when considering the finite element predictions with respect to the finite element greenfield curves, the equivalent masonry beam model can be considered to have worked very well.

As discussed in section 9.4, the soil model used for these analyses was developed for modelling cohesive soils rather than the non-cohesive soils that exist at the Moodkee Street site. The results presented should thus be considered with this in mind. Numerical analyses of this case study site with a more appropriate soil model could form part of future modelling work.

The case study site chosen could be said to be less useful than other alternatives, as the settlements and damage were not severe enough to test the numerical model fully. This is something of a ‘catch-22’ situation as the site was chosen partly for the fact that the

buildings were not protected by compensation grouting, but the reason protection was not required was that the predicted settlements and damage were considered to be negligible. Further case study verification of the equivalent masonry beams is required using cases with more severe surface settlements.

9.6 Conclusions

This chapter described the use of numerical modelling techniques to model the impact of tunnel construction on surface settlements and building damage in three dimensions. The numerical modelling techniques used are those developed as part of this thesis and described in previous chapters.

The results of the numerical models have been compared to the observed surface settlements from on-site monitoring of Neptune, Murdoch and Clegg Houses during the construction of the Jubilee Line Extension tunnels in 1996. This comparison shows that the building response predicted by the numerical model, agrees well with the observed response. In particular, the model accurately predicts a flexible response when the buildings are in a hogging mode of deformation and a stiff response when they are in sagging, as was observed during tunnelling on-site. The ability of the 3D model to capture the response of multiple buildings with differing asymmetrical orientations to the tunnel during a staged construction sequence which allows investigation of transitory effects was demonstrated. The importance of modelling of both the three-dimensional aspects of building response to tunnelling and the different nature of the response of masonry buildings in hogging and sagging when making predictions regarding building response is thus reinforced.

The wide range of different case study sites in the published JLE material and the quality of detailed observations of building response to tunnel construction should encourage more researchers to make use of the data available in the future development of 3D numerical modelling of building response to tunnelling.