Tunnelling in Warsaw- Technical Abstract

Catherine Tyldsley (ME)

Introduction

A second Metro line spanning from East to West is under construction in Warsaw and a third line is being planned and will connect to line 2. A geological report found that the ground conditions varied along the line and was a mixture of sands and clays. In the South Eastern region of the site there was a large layer of sand and the tunnel face was in mixed ground conditions. A tunnel boring machine with an Earth pressure balance shield was used to minimise settlements and arch grouting was used under buildings in the South Eastern region. The objectives of the investigation were to analyse the ground movements induced by tunnelling and compare these to past technical experiences to find out whether the complicated geological conditions affected the results. To do this the transverse and longitudinal settlements in grouted and non-grouted areas will be compared alongside the level of face pressure used during tunnelling will be determined. To assist with the design of line 3 the effect of tunnelling on buildings within the connection area of line 3 will be investigated.

Technical Background Information

A strategy devised by Peck, 1969 suggested that a Gaussian curve distribution could be used to model the transverse settlement trough once the TBM had passed a sufficient distance to allow the settlement trough to develop. Once the Gaussian curve is known, the volume loss can be expressed as a percentage of volume of soil excavated by the tunnel. O'Reilly and New (1982) also found that past UK tunnelling data suggested that the point of inflexion is a linear function of the tunnel depth and trough width parameter, K. In clays K is within the range of 0.4-0.6 and between 0.25-0.45 in sands. It has also been proposed that the longitudinal settlement can be portrayed using a cumulative probability curve, Attewell and Woodman (1982), where the settlement at the tunnel face passes is approximately $0.5S_{MAX}$ in clays however when an EPBM is used this may be reduced. The presence of buildings may reduce the deflection due to their stiffness, the effect of this can be assessed using the deflection ratio of the green field and building settlements to obtain the modification factor in both the hogging and sagging regions. Superposition of the effect of separate tunnels can be used to find the settlement due to both tunnels, O'Reilly and New (1982).

Investigation

Observing the transverse settlements found that the mixed face ground conditions greatly affected the volume loss and trough width parameter. Boring of the first tunnel caused less volume loss in the grouted area (0.36%) than the non-grouted area (0.69%) which would not be expected. However when the second tunnel was constructed they were more consistent, 0.50% and 0.58% respectively. It was found that the trough width parameter values were higher than expected in sands and lower than expected in the presence of clays. However the values obtained from tunnel 2 were more consistent than tunnel 1 suggesting that the standard of workmanship had improved between 1 and 2 tunnels being constructed. Grouting appeared to increase the trough width parameter which was the desired effect.

Longitudinal settlements suggested that a high face pressure was initially used during construction giving 10% of maximum settlement at the cutter face however this decreased in the predominantly clay section where a settlement of 50% was found. The settlement then greatly increased to 80% in the grouted areas. The reason for the variation could be due to the mixed face conditions or the standard of workmanship.

It was found that in the area which grouting was conducted that there was an additional settlement which was not caused by the construction of the tunnels. The analysis showed that at 2 building sections the building deformation was greater than the green field which was not expected but was supported by the Gaussian curves which were larger than the green field curves. However a third building section showed rigid behaviour after both tunnels were constructed. Although the investigation was conducted assuming that the buildings were one, they were in fact multiple buildings with little or no spaces between them and this may have affected the results obtained.

Conclusion

In conclusion it was found that the mixed ground induced a lot of unpredictability in the behaviour and produced range of volume losses. It also increased the trough width parameter to greater values than would be expected. The face pressure was initially high but decreased over the tunnelling length. The buildings appeared to behave flexibly however this needs to be investigated further by considering multiple buildings instead of a single length.

Tunnelling in Warsaw

Catherine Tyldsley (ME)

Fourth-year undergraduate project

In Group D, 2013/2014

I hereby declare that, except where specifically indicated, the work submitted herein is my own original work

Contents

Table of Figures

Table of Tables

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1. Introduction

1.1 Warsaw

Warsaw is the largest city in Poland and is located next to the Vistula River. It has an inner city population of approximately 1.717 million and a suburbs population of 0.422 million. The inner city area is 517.24 square kilometres with a further 721.38 square kilometres of suburbs. No Ring Road system for vehicle traffic exists in Warsaw resulting in the underground Metro being a popular mode of transport accommodating 139.17 million riders per year (Metro Warszawskie Company, 2012).

The current Metro line, line 1, opened in 1995 and runs from the North to the South of the city, it is 22.7km long and includes 21 stations. Line 2 is under construction and will link the West and East suburbs to the inner city. Along line 2, there will be 27 stations in total, 7 of which are in the central section. A transfer to the existing line 1 will be at Świetokrzyka Station. Planning is in progress for a third line which will provide connections from the South eastern suburbs to the centre. It will include 8 stations and a transfer to line 2 at Stadion Station. Figure 1.1 shows the interaction of the three lines.

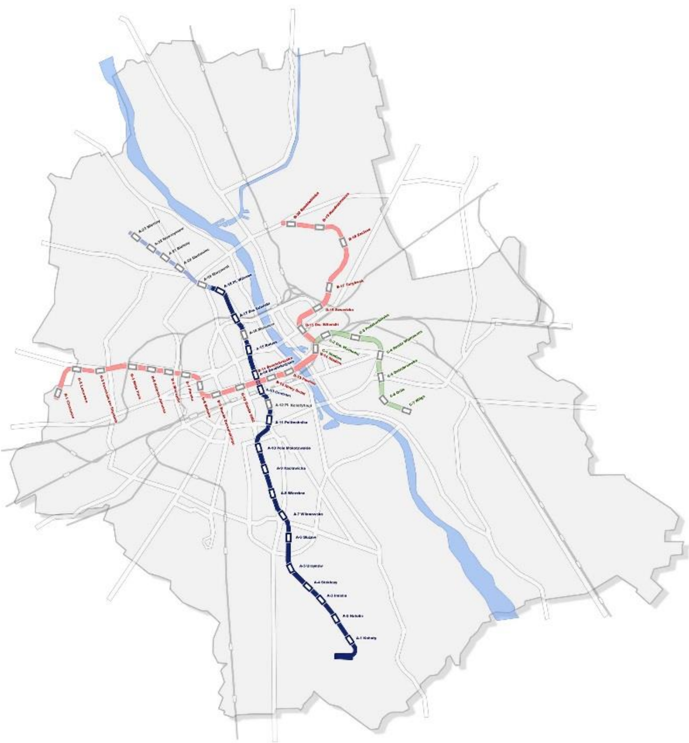


Figure 1.1 Interaction of the Metro lines. Blue being line 1, red being line 2 and green being the proposed line 3.

1.2 Motivation

The construction of line 1 was prolonged due to a number of factors. A lack of funding, poor planning and input from government officials all contributed to the construction taking longer than anticipated. Construction began in 1984 and 11 years later the Metro was opened. This is in direct contrast to lines 2 and 3 which will be constructed using further advances in technology which are better suited to the hydro-geological conditions found in Warsaw. A Tunnelling Boring Machine (TBM) with an Earth Pressure Balance shield (EPB) is being used to handle the soft ground conditions and minimise disturbance to the surrounding ground. This is essential as the majority of the Metro is to run under a dense urban development including a number of pre-war buildings with historical monument status (UITP CEEC, 2013).

An Italian construction company, Astaldi, was awarded the contract for line 2 and as part of the work they installed numerous pieces of measuring instruments in order to monitor the performance of the tunnelling. These monitoring instruments are extremely useful when one is evaluating the success of using the advanced tunnelling methods in these ground conditions.

An area of particular interest is where the proposed line 3 will connect to line 2 as in this area there are a number of tall buildings and the ground has a large layer of sand above the proposed tunnel axis. Arch grouting around the tunnel was used to protect the buildings from any damaging settlement and the performance of this can be analysed using the data from the monitoring instrument. This can then be used to assist any decision making when further designing the section of line 3 in this area.

This report will therefore focus on this region of line 2 in anticipation of line 3 being constructed in similar ground.

1.3 Site Location

The transfer station to line 3 is proposed to be at Stadion Station, C14 which is located on the lower over-flood terrace on the right bank of river Vistula in the Praga Pólnoc District. As shown in Figure 1.2 tunnel, D15, exits the station and turns North to run along the centre of Targowa Street. Along Targowa Street it runs under the tram line which separates the vehicle traffic lanes and reaches Dworzec Wileński Station, C15.

When exiting station C14, tunnel D15 runs under several buildings and is also in close vicinity (10-20m distance) of tall buildings along Targowa Street.

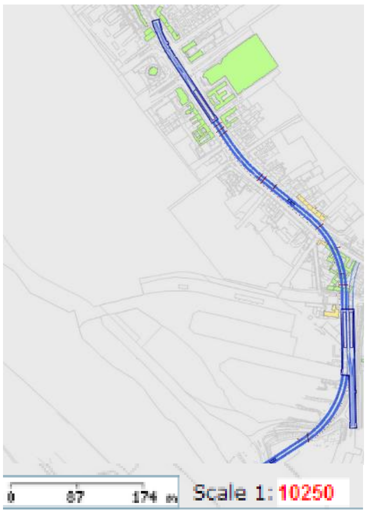


Figure 1.2 Tunnel leaving C14, turning north and then running under Targowa Street

1.4 Project Objectives

1. Understand past technical experience of tunnelling and to what degree this can be used to analyse the data that is available for the site in Warsaw.
2. Recognise the possible effect of the geological conditions on the tunnelling.
3. Extracting and manipulating suitable data in order to analyse the ground movements induced by tunnelling.
4. Create a method to perform the analysis which is of a suitable accuracy.
5. Perform an analysis on the transverse settlements in grouted and non-grouted areas.
6. Determine the likely influence of face pressure on the longitudinal settlements.
7. Compare the settlements of buildings to the green field case and use this to determine an approximate building stiffness that can then be used when designing line 3 which is to pass within the same building area.

2. Technical Background Information

2.1 Transverse Settlements

2.1.1 Green field Settlement

Firstly, the green field[1] conditions are considered for a single tunnel. It was following the strategy suggested by Peck, 1969, that a Gaussian curve distribution could be used to potray the transverse settlement trough immediately after construction as shown in Figure 2.1. Equation 2.1 is used to describe the green field settlement for a single tunnel.

$$S_v = S_{max} exp\left[\frac{-y^2}{2i^2}\right] \quad \textbf{Equation 2.1}$$

where S_v is settlement

S_{max} is the maximum settlement on the tunnel centre line

y is the horizontal distance from the centre line

i is the horizontal distance from the tunnel centre line to the point of inflexion on the settlement trough

However sufficient time must be allowed for the TBM to pass the point of interest in order to allow for the settlement trough to be develop. A distance of $2z_0$ is considered to be sufficient for the TBM to have fully passed where z_0 is the tunnel axis depth.

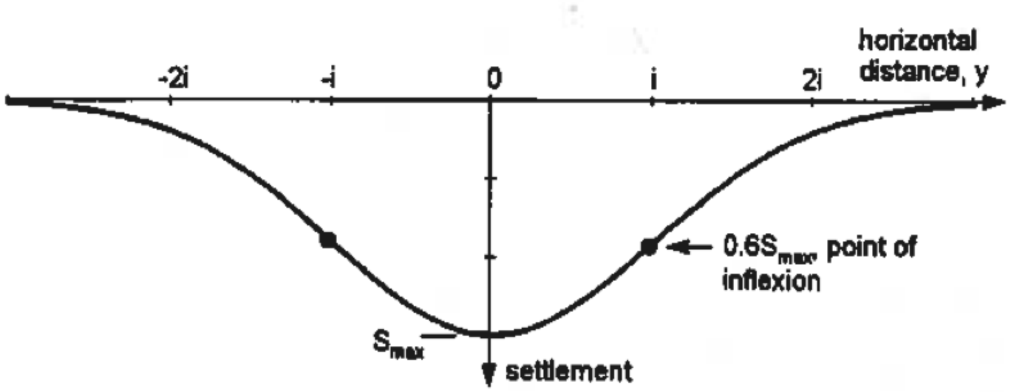


Figure 2.1 Gaussian approximation of the green field transverse settlement trough

2.1.2 Volume loss

The volume of the surface settlement trough per metre length, V_s can be found by integrating Equation 2.1, see Equation 2.2. The volume of ground lost in the area close to the tunnel is known as volume or ground loss and is denoted, V_t . In clays under drained conditions it can be assumed that V_s=V_t whereas in dense sands undrained conditions V_s is less than V_t due to dilation, Cording and Hansmire (1975). However a

[1] Green field conditions are experienced when the land is undisturbed and in its natural state

more effective way of representing the volume loss is as a fraction of the excavated tunnel area. Assuming that the tunnel is circular the volume loss is shown in Equation 2.3.

$$V_s = \sqrt{2\pi} i S_{max} \quad \textbf{Equation 2.2}$$

$$V_s = V_l \frac{\pi D^2}{4} \quad \textbf{Equation 2.3}$$

2.1.3 Trough width parameter

Examining past UK tunnelling data O'Reilly and New (1982) suggested that the point of inflexion i, is a linear function of the tunnel depth z_0 and the trough width parameter, K, Equation 2.4. It was also shown that i was independent of the tunnelling method and diameter and K is dependent on the material that the cutter face is in. Typically K is equal to 0.5 for tunnels in clay and 0.25 in sands. Although there is some variation in the trough width parameter as shown in Figure 2.2 for clays and Figure 2.3 , in sands (Mair, 1997). However the mean line of both plots support the values given by O'Reilly and New (1982).

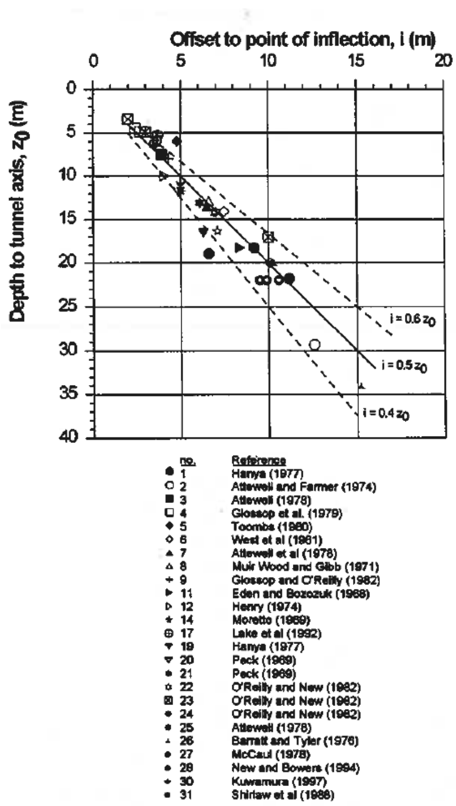


Figure 2.2 Variation of trough width parameter in clays

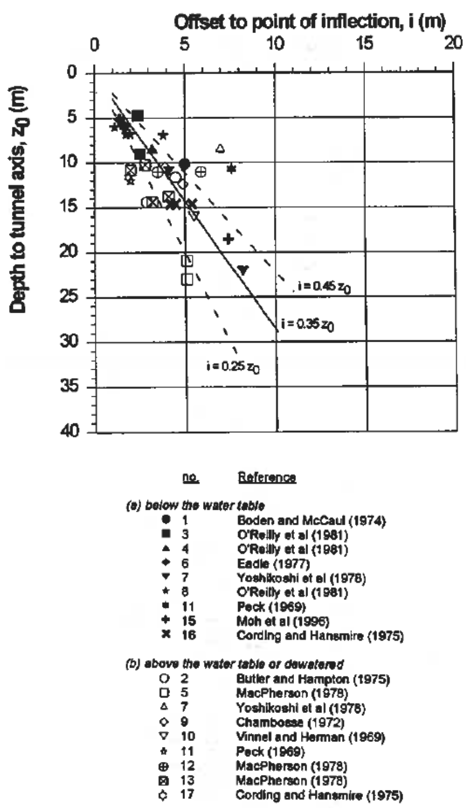


Figure 2.3 Variation of trough width parameter in sands

$$i = Kz_0 \quad \textbf{Equation 2.4}$$

2.2 Failure Mechanisms

The failure mechanisms demonstrated by the soil above the tunnel is dependent on the type of soil it is. In clays the mechanism travels upwards and outwards resulting in the trough being larger than the diameter of the tunnel. Conversely, in sands the failure mechanisms propagates upwards causing a narrower trough width to be formed. See Figure 2.4 (Mair, 1979).

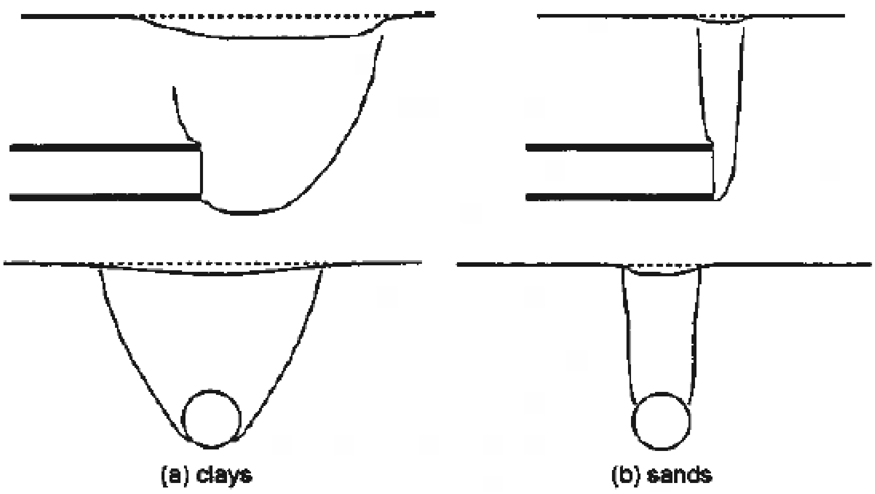


Figure 2.4 Failure mechanisms in clay and sand

2.3 Longitudinal Settlements

Following on from the transverse settlement trough being portrayed by a Gaussian distribution curve, the longitudinal settlement can then be profiled as a cumulative probability curve as shown by Attewell and Woodman (1982). Collectively they found that in clays the surface settlement at the cutting face is approximately $0.5S_{max}$ when tunnelling without any face support. This is then reduced if an EPB shield is used and results in the cumulative probability curve being translated so that movement is further away from the cutting face as shown in Figure 2.5 (Mair and Taylor, 1997). However this approximation has not been validated in sands.

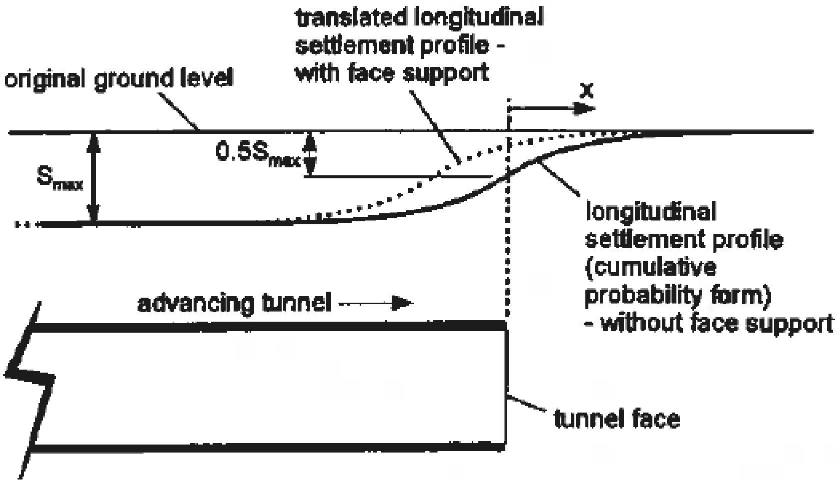


Figure 2.5 Longitudinal surface settlement trough (Mair and Taylor, 1997)

2.4 Building Deformation

2.4.1 Green field Deformation and Deflection Ratio

It is common to assume that the building will deform with the green field settlement. This is a conservative approach because the inherent stiffness of the building will reduce the deflection ratio, horizontal strains and therefore potential damage to the building. In order to calculate the deflection ratio, the building is split into separate hogging and sagging zones, using the point of inflexion as the separation point, as shown in Figure 2.6. The deflection ratio is defined as the ratio of the difference in settlement between the green field and building lines and the “effective building length”, see Equation 2.5.

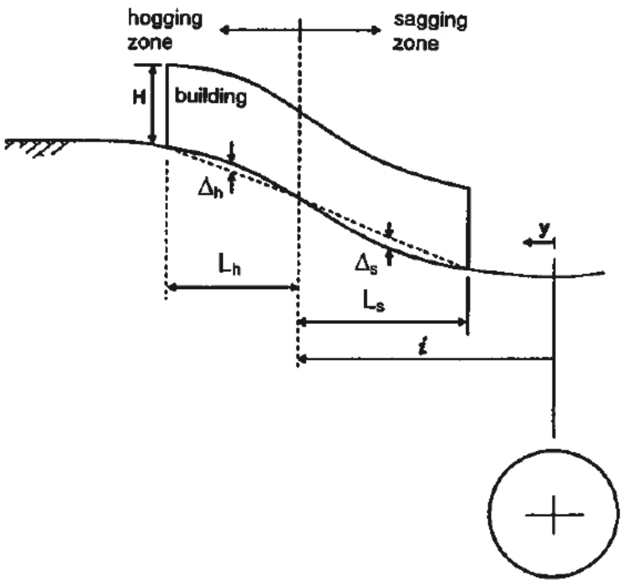


Figure 2.6 Deformations of the building above a tunnel (Mair et al, 1996)

$$Deflection\ Ratio = \frac{\Delta_h}{L_h}\ or\ \frac{\Delta_s}{L_s}$$ **Equation 2.5**

2.4.2 Modification Factor

A modification factor was suggested by Potts and Addenbrooke (1997) to account for building stiffness, see Equations 2.6 and 2.7. If $M^{DR,hog} = 0$ the building is fully rigid and no deflection is experienced by the building however if $M^{DRhog} = 1$ the building is fully flexible and follows the green field curve. Figure 2.7 can then be used to determine the range of building stiffness that the building demonstrated.

$$M^{DR,hog} = \frac{DR_{hog}}{DR^{gf}_{hog}} \quad \textbf{Equation 2.6} \qquad M^{DR,sag} = \frac{DR_{sag}}{DR^{gf}_{sag}} \quad \textbf{Equation 2.7}$$

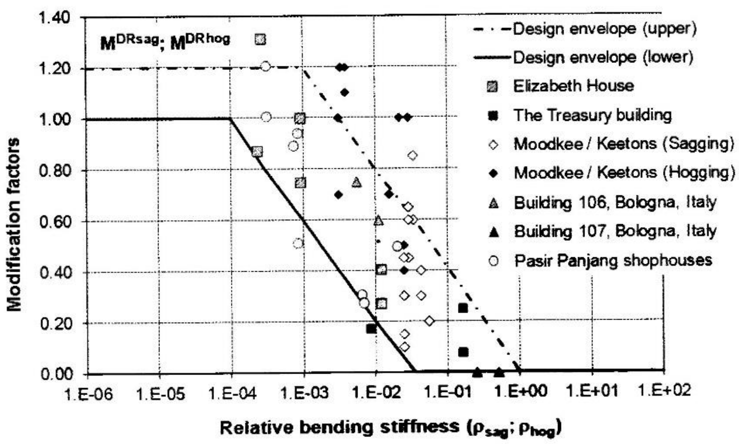


Figure 2.7 Field response to tunnelling using the modification factor and relative building stiffness (Goh, 2010; Farrell, 2010)

2.5 Parallel Tunnels

In the situation where there are two tunnels being constructed it is important to differentiate between the effect of the first, second and both tunnels collectively. O'Reilly and New (1982) derived the solution for the transverse settlements due to parallel tunnels using superposition. The effect of superposition is shown in Figure 2.8 and the corresponding equation in Equation 2.8.

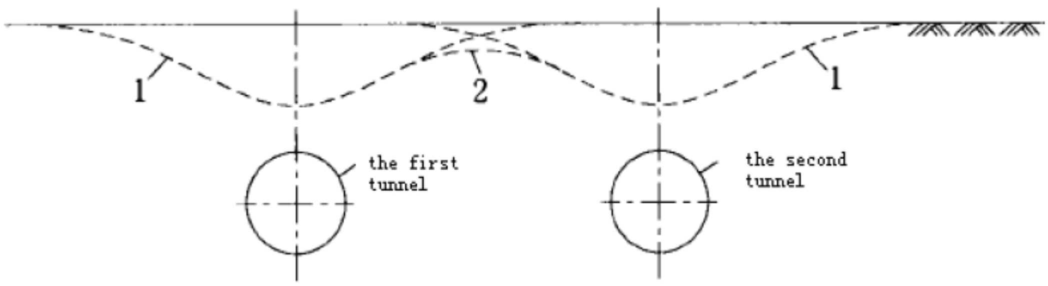


Figure 2.8 Surface settlement due to parallel tunnels

$$S(y,z) = \frac{V_s}{i\sqrt{2\pi}}\left[exp\left(-\frac{y^2}{2i^2}\right) + exp\left(-\frac{(y-D)^2}{2i^2}\right)\right] \quad \textbf{Equation 2.8}$$

3. Geological Report Findings

3.1 Morphological and Geological Conditions of the Site

The ground surrounding C14, C15 and D15 is made up of approximately horizontal ground strata. At ground level there is 1-2m of man-made fill (3m maximum) which is underlain by sandy deposits (7-18m) and is greatest at the South Eastern side of the site. There are fluvial deposits within the sand and these consist of medium and coarse sand with gravel and medium-coarse sands. Underneath the sandy deposits there are Pliocene high plasticity clays which range from a depth of 9m to 22m. However in the South East region there are areas of boulder clays between the sandy deposits and high plasticity clays. Boulder clays comprise of sandy clays, clayey sands, clays and silty clays and are at a maximum depth of 4m. Also within the South Eastern region there are silty sands and silts imbedded within the high plasticity clays.

3.2 Hydrogeological Conditions of the Site

Saturated fluvial sands and gravels overlay the high plasticity clays and have a high permeability coefficient, $k=3x10^{-4}m/s$. The Pliocene high plasticity clays were at a depth of 9-15m however at C14 Pliocene soils with alluvial deposits were found at a depth of 22m. Within the Southern region of the site old Quaternary deposits were found in localised areas but were almost totally eroded.

The natural variation of ground water level is ±0.5m and 1-1.5m in the northern and southern parts of the site respectively. The tunnel was constructed in the high plasticity clays and fluvial sand with gravel zones. It is expected that no deformation of ground water flow will occur due to the interference of the tunnels and stations.

3.3 Geotechnical Layers

3.3.1 Division of Geotechnical layers

Firstly, the soils were separated into groups of similar strains, strengths and permeability. These properties are initially governed by grain size distribution, origin and conditions experienced during consolidation. This criteria was used along with the geological engineering documentations made by GEOTEKO in 2007 and Consortium GEOTEKO-SGGW-GEOPROJEKT in 2003-2004, 2009 and 2010 in order to classify the soil into 16 geotechnical layers.

3.3.2 Identified Soil Layers

3.3.2.1 Tertiary Soils

Within the tertiary soils the layers identified were of Pliocene origin and were categorised as:

- Ia: High plasticity clays, silty high plasticity clays, more plastic clays
- Ib: Other cohesive soils, mainly silty clays and silts
- Ic: Sands of different grain-size distribution, mainly silty and fine sands

3.3.2.2 Quaternary Soil

Quaternary soils created between early Pleistocene and Holocene periods. Grain-size distribution was used to classify non-cohesive layers II, III and IV, cohesive layers V and VI and layers of inorganic matter, layer VIII. The quaternary soils layers identified were:

- II: Fine and silty sands
- III: Medium and coarse sands
- IV: Dense sand-gravel mixtures and gravels
- V: Clays
- VI: Clayey sands and silts
- VII: Organic muds

4. Construction Process

4.1 Method of Tunnelling

Two tunnels were constructed using a tunnel boring machine with an Earth pressure Balance Shield, EPBM. An EPBM is suitable for the soft ground conditions in the Warsaw region. The EPBM mixes the excavated soil into a slurry which is then used to balance the earth and water pressures and stabilises the machine-soil interface preventing face collapse[2]. Within the EPBM machine there is a method of excavating the soil, a system to mix the soil, a screw and belt conveyor to remove the excavated soil and a control system to main a pressure balance. A typical EPBM configuration can be seen in Figure 4.1. The geometry of the lining was the same for the North and South tunnel and is summarised in Table 4.1. A data management tool was used to continuously monitor the

[2] Collapse of the ground at the cutting face

TBM tunnelling positions allowing for any adjustment to the face pressure and cutter positions to be made during execution.

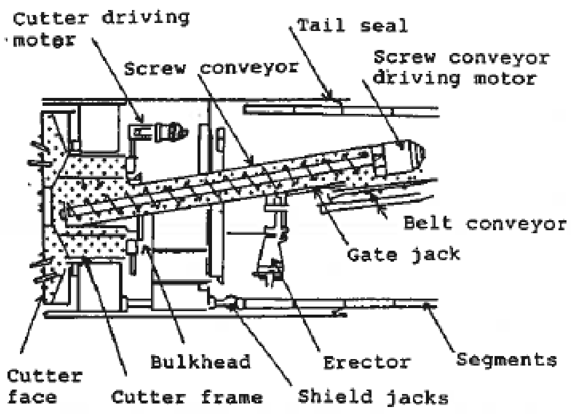


Figure 4.1 Earth Pressure Balance Machine Concept (Fujita, 1989)

Ring Inner Diameter (m)	5.4
Ring Outer Diameter (m)	6
Shield Outer Diameter (m)	6.27
Face Area (m^2)	30.88
Thickness ring/shell (m)	0.0
Ring Distance (m)	1.5
Shield Length (m)	6.55

Table 4.1 Geometry of the lining

4.2 Grouting

4.1.1 Grouting Configuration

A cement based and chemical grouting mix was used to stiffen the structure of the soil, reducing surface settlements and widening the settlement trough. Horizontal directional drilling HDD from station C14, was used due to existing utilities. Grouting extends for 250m and 24 pipes per tunnel were needed to consolidate the soil to the correct volume and were placed in a quincunx configuration as shown in Figure 4.2. The minimum vertical distance between the pipes and building foundation is 4m which gives an area of protection of approximately 4m larger than the building footprint. HDD was conducted blind because the section is not affected by consolidation onset by hole drilling. Figure 4.3 shows the area of grouting.

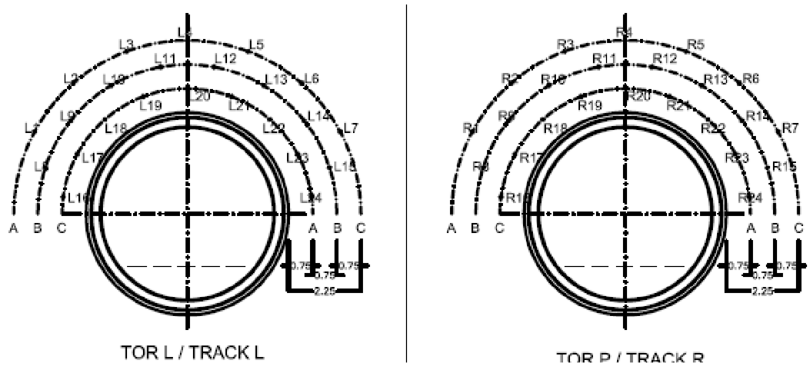


Figure 4.2 Grouting hole configuration

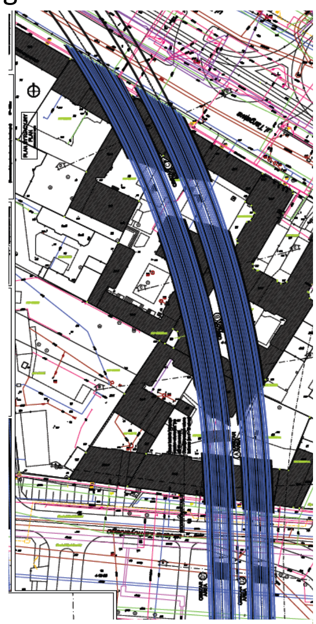


Figure 4.3 Blue area indicates the area of grouting conducted

4.1.2 Grouting Procedure

In order to guide the drilling of the holes a magnetic field was created and was detected topographically. This was then used to find the position of the drilling rods and was continuously monitored throughout the drilling process.

The HDD was conducted from inside station C14 and due to the small work area only one set of equipment was used. Equipment was used to drill holes for the first tunnel and then moved to drill the second set of holes. Whilst the second holes were being drilled it was possible to start the grouting process in the first set of holes.

Once drilling was complete, the rods were removed from and valve pipes inserted into the casing. Sheath grouting was then conducted and the casing was removed as the lining was made. Grouting is complete once the drilling fluid is fully replaced by the grout.

To conduct the grouting a hydraulic double packer was inserted to the furthest location and gradually removed. Grouting was continued at each sleeve until the design volume or refusal pressure was reached. For the first pass of grout the design volume was 12% of the volume of soil to be treated and the refusal pressure was 8 bar. Whereas the second pass of chemical grout had a design volume of 22% and refusal pressure of 12 bar. An average absorption level into the soil of 8% and 18% was expected in the first and second pass respectively.

5. Monitoring Data

5.1 Instruments

Three types of monitoring instruments were used to analyse the settlement caused by tunnelling and these were levelling pins (SL), mini prisms (MP) and ground pins (CS). Each were installed and regularly monitored during the construction period. The typical installation of each type of equipment can be seen in Figure 5.1.

Each set of equipment was given an “alarm value” and if the displacement became greater than this value then the engineer would be notified in order for appropriate action to be taken.

A list of the instruments used in this report can be found in the Appendix.

Figure 5.1 Installation of levelling pin, mini prism and ground pin

5.2 Data Representation & Interpretation

An issue found during the analysis was that the instruments were not all installed before tunnel construction was commenced and the frequency of monitoring was not constant. Therefore the instruments used for the investigation were carefully selected to provide the most accurate analysis. Also, as mentioned previously, it is essential that the effect of separate tunnels and both tunnels is known. To do this the effect of the first tunnel must be removed when analysing the second tunnel. Superposition can then be used to obtain the effect due to two tunnels.

It was decided that a C++ programme would be used to present the data as this allowed for the most accurate manual approximations to be made.

6. Investigation

6.1 Transverse Settlements

6.1.1 Method

Due to the steady variation in strata along the length of D15 it was not necessary to perform a transverse analysis at every set of instruments. Cross-sections were chosen at areas which may be of interest, such as at the centre of intersecting roads, before and after buildings and approximately 50m between these points.

In order to evaluate the effect on the ground due to the first tunnel, the settlements of the instruments were taken after the tunnel has passed the point of interest by a distance 2z, as shown in Figure 6.1. Grouting may cause heave due to the addition of material into the soil, in order to account for this the difference between the value after grouting and the value after the first tunnel was taken. To find the settlement due to the second tunnel a similar method was used, in turn the settlement after the first tunnel must be subtracted from the final value. These calculations are summarised in Equations 6.1 to 6.4.

To plot the points a co-ordinate system was made which was perpendicular to the direction of tunnelling. The settlements were then plotted and assessed by eye as to whether a Gaussian curve of form would be suitable for the settlement trough. The maximum settlement and point of inflexion were varied until the most applicable Gaussian curve was found. From the Gaussian curve, the trough width parameter and volume loss could then be found using equation 2.4. The direction of tunnelling is shown in Figure 6.2.

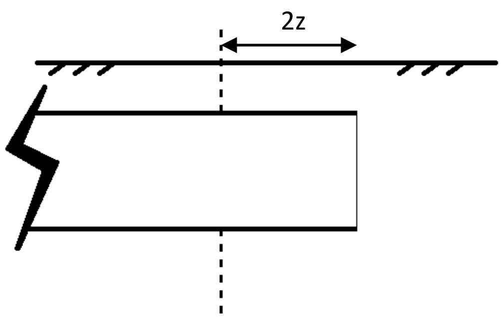


Figure 6.1 Position of tunnel when settlements are taken

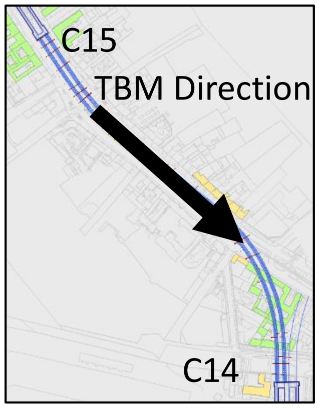


Figure 6.2 TBM direction

Equation 6.1

$$True\ First\ settlement = Grouting\ Settlement - Recorded\ First\ Settlement$$

Equation 6.2

$$True\ Second\ Settlement = Recorded\ Second\ Settlement - Recorded\ First\ Settlement$$

6.1.2 Results

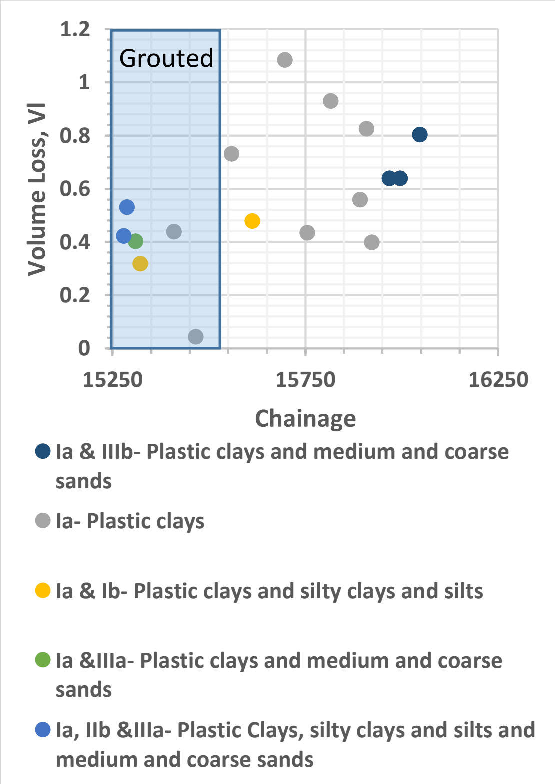


Figure 6.3 Volume loss against chainage- first tunnel

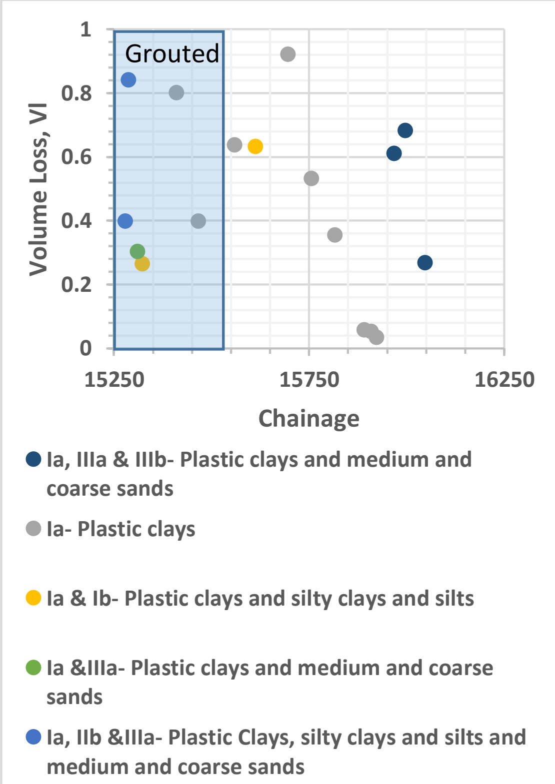


Figure 6.4 Volume loss against chainage- second tunnel

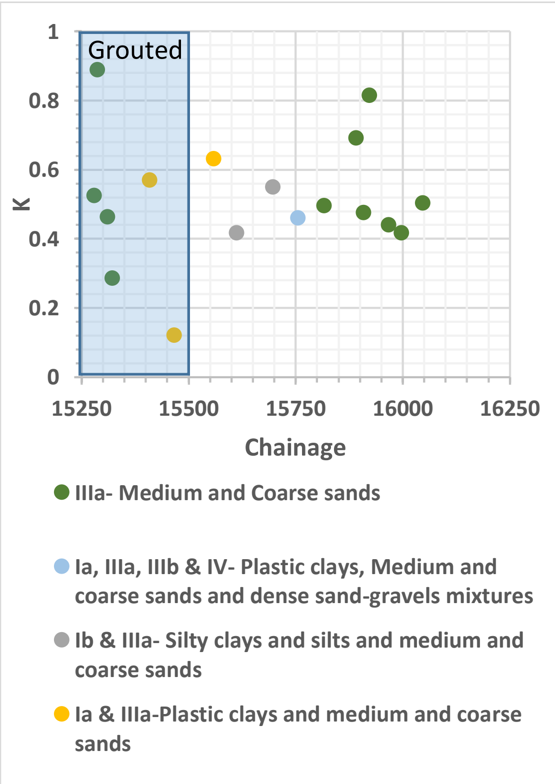


Figure 6.5 Trough width parameter against chainage- first tunnel

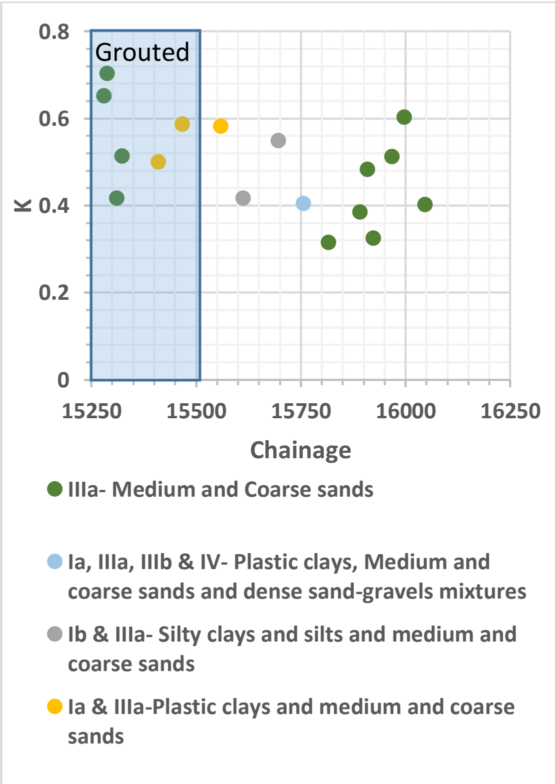


Figure 6.6 Trough width parameter against chainage- second tunnel

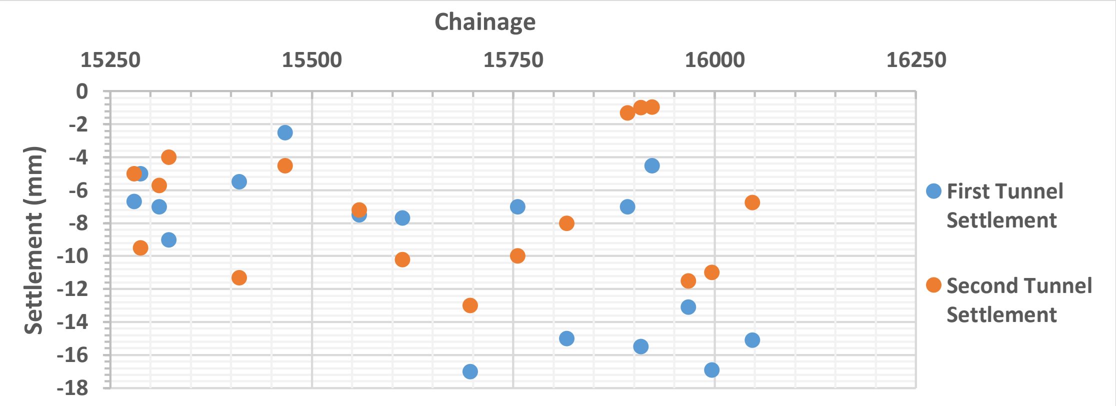


Figure 6.7 Settlement against chainage

6.1.3 Discussion

6.1.3.1 Volume Loss

From a review of the results for the first tunnel it can be seen that the mean volume loss in non-grouted areas is 0.69% whereas in the grouted areas it is significantly lower at 0.36%. It would be expected that the volume loss would remain constant and this is not the case however this could also be due to the change in strata between the non-grouted and grouted areas.

Figure 6.3 and Figure 6.4 shows the volume loss in different strata along the project chainage, it should be noted that tunnelling began at the higher chainage- approximately 16050m. It can be seen from Figure 6.3 that the volume loss at the start of tunnelling (highest chainage) was approximately 0.80% and then decreased suddenly to 0.60%.In this area there were mixed face conditions. This could be due to the workforce adjusting to the strata as tunnelling progresses. Figure 6.3 also shows that the volume loss is lower than expected in areas where the tunnel face is fully in plastic clay. A cause of this may be the material above the tunnel being entirely sand which would cause the trough to be narrow hence reducing the volume loss, see Figure 6.8.

Within the grouted area, the first set of instruments encountered, D1536, experience an extremely low volume loss in comparison to the other values within the grouted area. The volume loss experienced was equal to 0.04%. Investigating this point further it was found that 5 days before the TBM had reached the instrument, there was a sudden increase in settlement; the maximum being -2.7mm. Figure 6.9 shows the settlement over time and it shows that there was then a smaller increase in settlement as the TBM passed. This suggests that once the sudden increase in settlement was detected by the operators, the face pressure was increased in order to prevent damage to the buildings. Therefore, resulting in a lower settlement when the TBM passed the instrument and therefore a smaller volume loss.

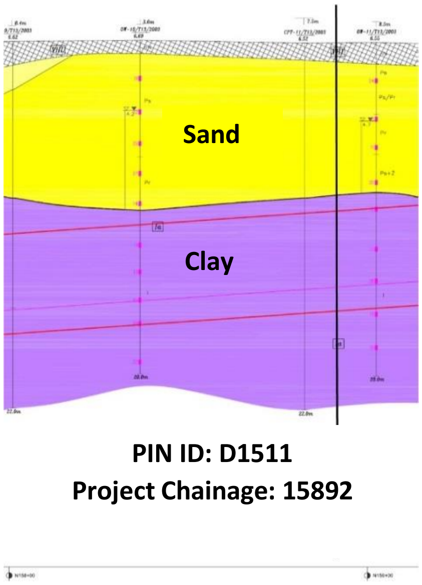


Figure 6.8 Cross section showing layered sand and clay strata

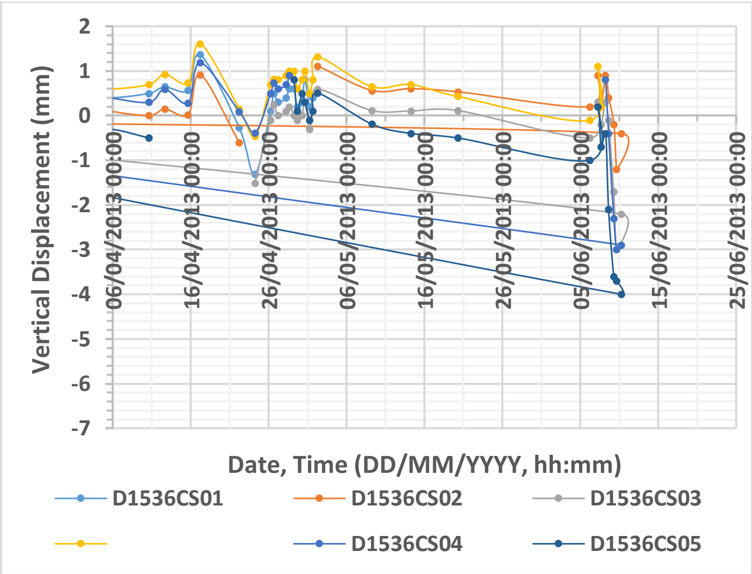


Figure 6.9 Settlement against time for D1536- the first set of instruments within the grouting area

Similarly in tunnel 2, there were a large range of volume loss values however the volume loss in the non-grouted and grouted areas were more consistent with means of 0.581% and 0.502% respectively (excluding anomalies). Three points in Figure 6.4 are significantly lower than the other results. Further analysis found that the readings within these points increased when the tunnel passed. This suggests that there was not an increase in settlement which does not follow previous trends. The type of instruments being measured at these cross-sections are ground pins therefore a cause of this could be human error when using an automatic level machine.

Volume loss within the grouted area of tunnel was more varied than tunnel 1 however there were 4 points that showed similar volume losses. There was a large volume loss in the penultimate instrument within the grouting zone, this was also the first set of instruments after exiting the building. The presence of buildings can provide additional stiffness which may decrease the settlement. Therefore when exiting the building zone the sudden increase in settlement may be due to the abrupt loss of stiffness. This would in turn result in the large increase in volume loss shown in Figure 6.4.

6.1.4.2 Trough width parameter
In sand the range of expected trough width parameter values is between 0.25 and 0.45, Figure 6.4 shows that the values of K in the medium and coarse sand are above or towards the upper limit of this boundary.

In the grouted area, the trough width parameter is initially low which would result in more damage on the buildings. It immediately increases which suggests the operatives adjusted the face pressure in order to lower the potential damage to the buildings.

Overall tunnel 2 is more stable than tunnel 1 which suggests that the operatives may have revised their technique in reflection of the outcome of tunnel 1, Figure 6.5. Again, the trough width parameter in sand is towards the upper boundary. The data point which is the greatest shows a large settlement after both tunnels, it is also within 100m of the starting point of tunnelling and may be within the learning period of the operatives.

There is less variation within the grouted area and the K values are generally greater than that in the non-grouted area which is expected because this would cause less damage to the buildings above.

6.1.3.3 Settlement
At the beginning of tunnelling it would be expected that the settlement would initially be large and then it would decrease suddenly as the tunnel progressed because the operatives would adjust to the strata, Mair (1996). However from Figure 6.6 it can be seen that there is not a sudden decrease in settlement and the settlement steadily reduces along the tunnel length. This may be because of the mixed face ground conditions which was experienced during the tunnelling. Mixed face tunnelling increases the difficulty of maintaining a constant maximum settlement and volume loss.

6.2 Longitudinal Settlements

6.2.1 Method
In order to plot the longitudinal settlement, the settlement of a pin above the tunnel axis was plotted as the tunnel approached. The first and second tunnel were modelled separately and compared between strata types.

6.2.2 Results

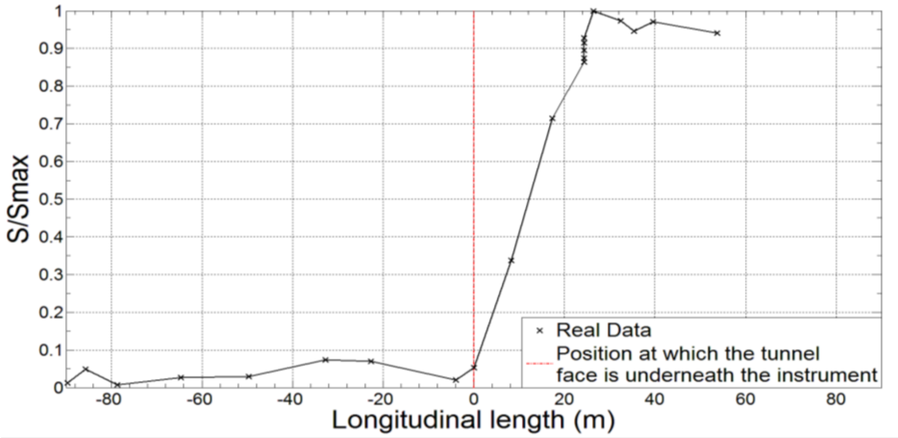


Figure 6.10 Longitudinal settlement in plastic clays and medium and coarse sands- first tunnel

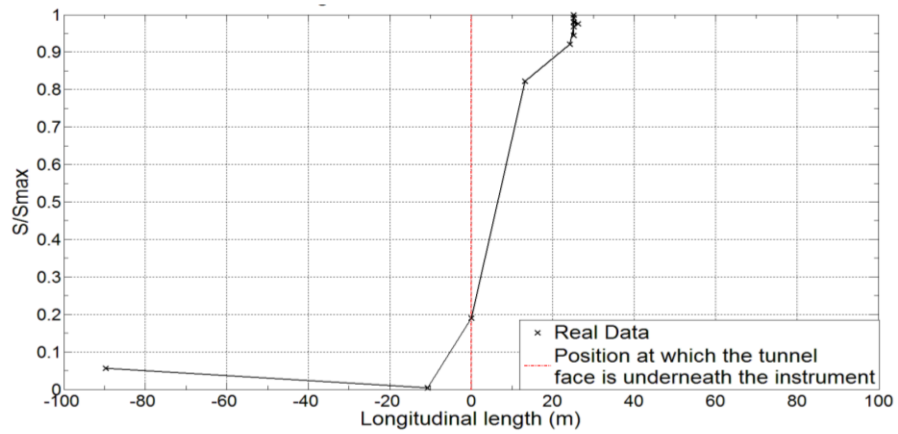


Figure 6.11 Longitudinal settlement in plastic clays and medium and coarse sands- second tunnel

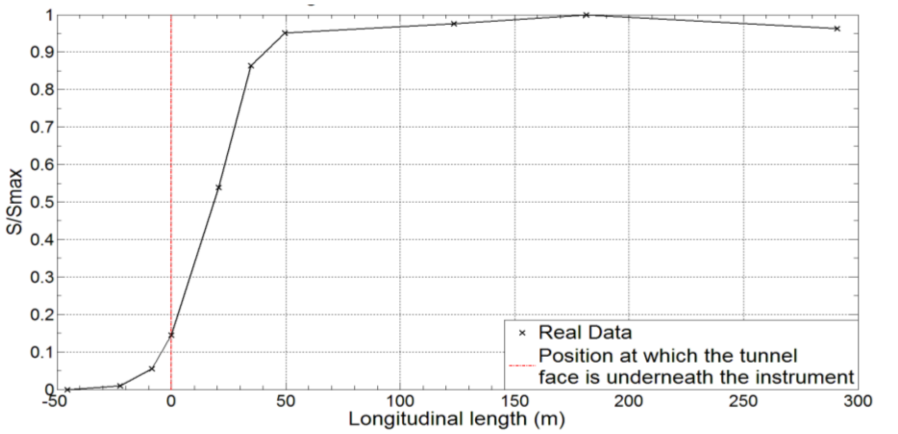


Figure 6.12 Longitudinal settlement in plastic clays- first tunnel

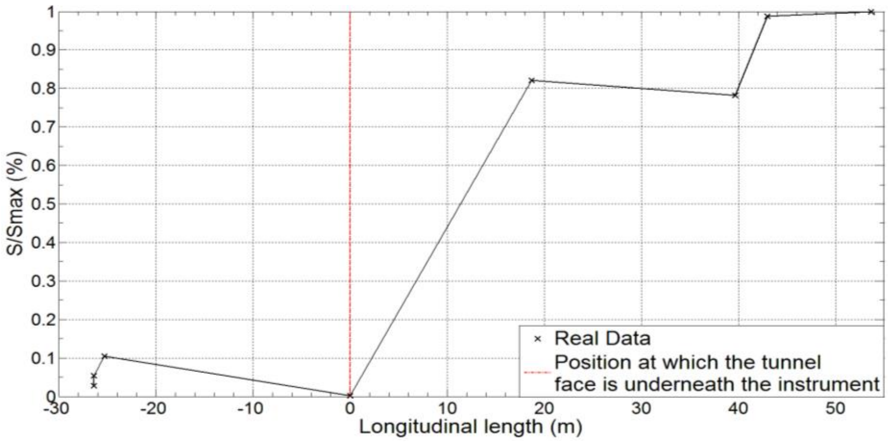


Figure 6.13 Longitudinal settlement in plastic clays- second tunnel

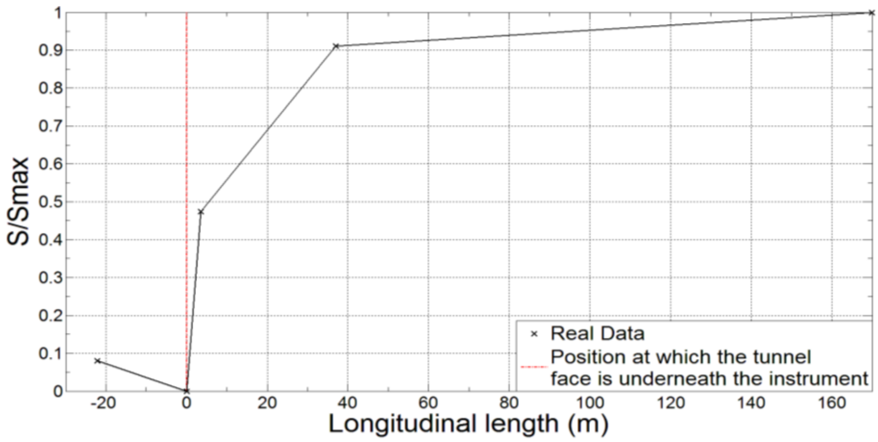


Figure 6.14 Longitudinal settlement in high plasticity clays- first tunnel

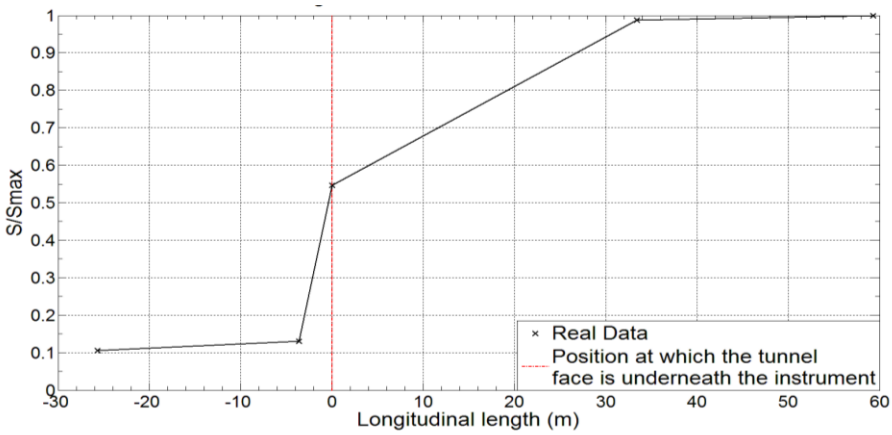


Figure 6.15 Longitudinal settlement in high plasticity clays- second tunnel

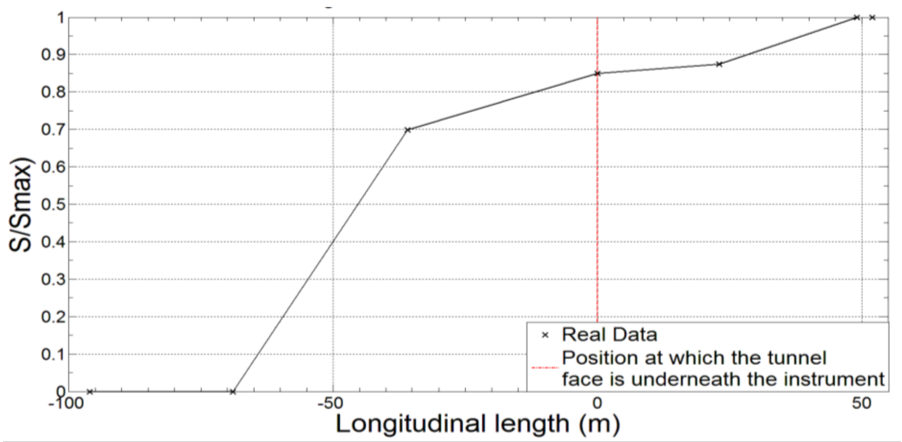


Figure 6.16 Longitudinal settlement in plastic clays, silty clays and silts and medium and coarse sands-first tunnel

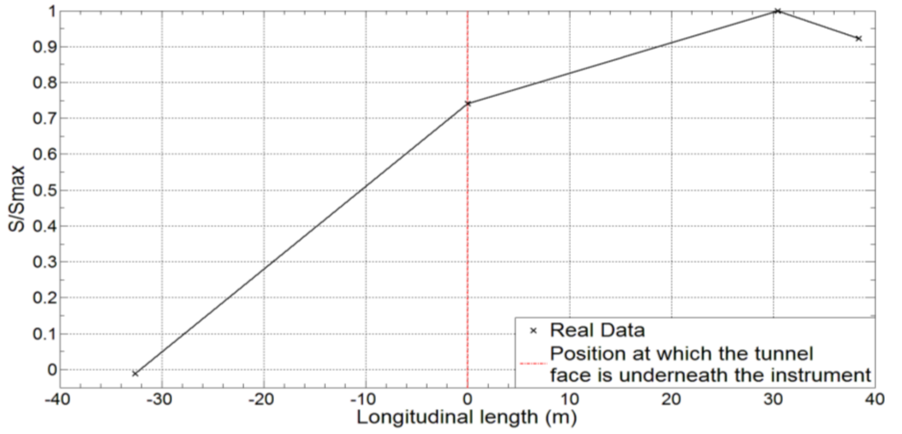


Figure 6.17 Longitudinal settlement in plastic clays, silty clays and silts and medium and coarse sands-second tunnel

6.2.2 Discussion

The longitudinal settlement was found at various locations along the TBM tunnelling path, in different strata and in non-grouted and grouted areas. The first location

Figure 6.10 and Figure 6.11, is at a TBM chainage 96m. It can be seen that the settlement at the point when the tunnel passes for the first and second tunnel is 5% and 20% of the maximum settlement respectively. This is significantly lower than was predicted in section 2.2 however that prediction had only been proven in clays and the strata in this area is a mixture of sand and clays. It was also stated that the use of an EPBM would decrease the settlement at the instrument point. Although a settlement of this size would suggest that a high face pressure had been used in order to prevent any settlement at the cutter face.

The second location, TBM chainage 232m, shows a similar behaviour to the first location- Figure 6.12 and Figure 6.13. Little settlement is seen when the tunnel face is in line with the instrument and when the second tunnel passed there was no settlement. In this area the tunnel was being constructed in plastic clays, therefore a settlement of 50% would be assumed when the tunnel was passing (this may be smaller as an EPBM shield was used). However it should be noted that the number of readings taken when the tunnel was underneath the instruments was sparse, meaning the reliability of the recorded settlement should be questioned.

The third location, TBM chainage 489m, the tunnel was constructed in high plasticity clays. This area showed the most similar behaviour to that predicted in section 2.2. It also suggests that a lower face pressure was used which may be because the operatives had gained more confidence or that they were more familiar tunnelling in the ground conditions. When the first tunnel passed, Figure 6.14 shows that the settlement was 0 but then it quickly decreased to 50%. There was a small time delay between the reading of the instrument and the tunnel passing, therefore the true settlement when the tunnel passed the instrument may be thought to be between 0% and 50%. It appears that when the second tunnel was being constructed there was an additional settlement of approximately 10% of the maximum settlement before the tunnel had approached the point, see Figure 6.15. This could be due to consolidation after the first tunnel or due to ground movement not associated with the Metro construction.

The final longitudinal settlement plot, Figure 6.16 and Figure 6.17, was analysed for a point within the grouted area. It can be seen from the longitudinal plots that the settlement was significantly higher than the other location and was approximately 80% of the maximum settlement. The cause of this could be the mixed face conditions as this would cause the settlement to be harder to control. However it should again be noted that the number of readings taken within this time period were low and the reliability of the data could be unreliable.

6.3 Building Deformation

6.2.3 Method

There are pictures of the buildings that are being investigated in Figure 6.18 to Figure 6.22. In order to calculate the effect on the buildings due to tunnelling each building edge needed to be compared with a green-field section close to the building. Ideally the lower edge of the building foundation would be used but information about the buildings were not provided. The strata within the building area is extremely varied and there are mixed face conditions, see Figure 6.23. For this reason, grouting was performed prior to the construction of the tunnels and any heave caused by this needed to be accounted for during the analysis. To do this, a plot of settlement against time was made to determine when the effects of grouting had subsided and this settlement was then taken into consideration.

The settlement points were then plotted for the construction of the first and second tunnels using the data from green-field and building instruments. It was then determined whether a Gaussian curve would be suitable for the data and if it was, superposition was used to find the effect of both tunnels. From these plots the hogging and sagging deflection ratio was calculated for the green-field and building and was then used to find the modification factor and range of building stiffness's within Figure 2.7. In the case of single tunnels the point of inflexion was used to determine areas of hogging and sagging. However in the case of both tunnels, the second derivative was calculated to determine the points of maximum gradient and these were then used to assess the hogging and sagging region. When performing the building deformation analysis it was assumed that the multiple buildings acted as one. This would not be the case however it simplified the analysis in order to provide an initial prediction of the building behaviour. In reality the buildings are a small distance apart at either side of the building, however these distances are not equal at either side of them, complicating the analysis further.

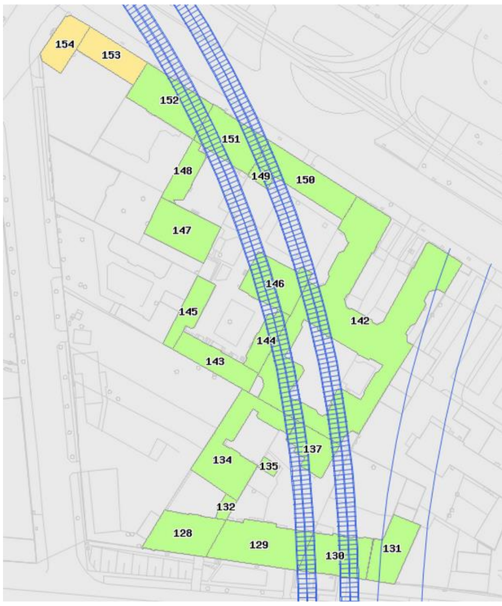


Figure 6.18 Building numbers in grouting area

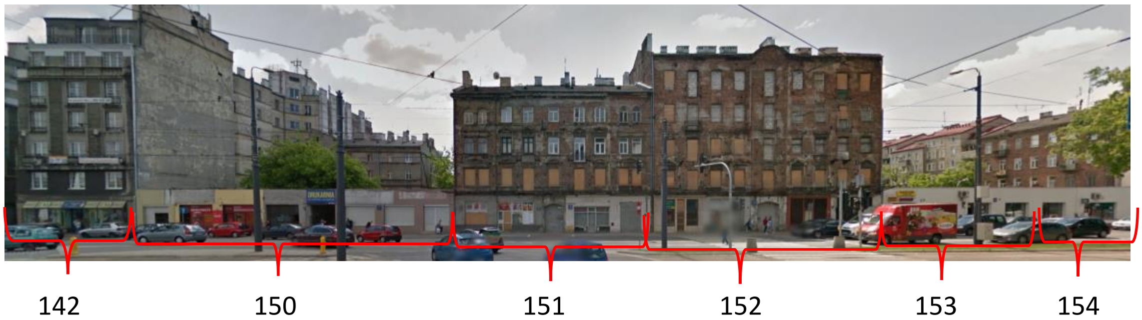


Figure 6.19 Elevation of buildings 142, 150-154

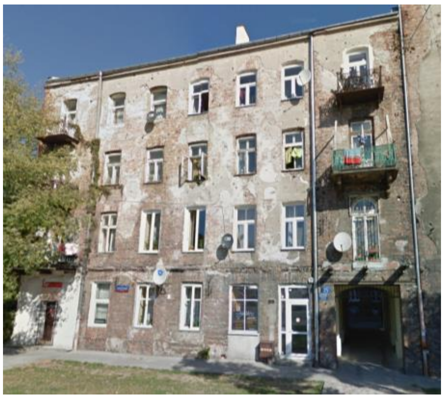
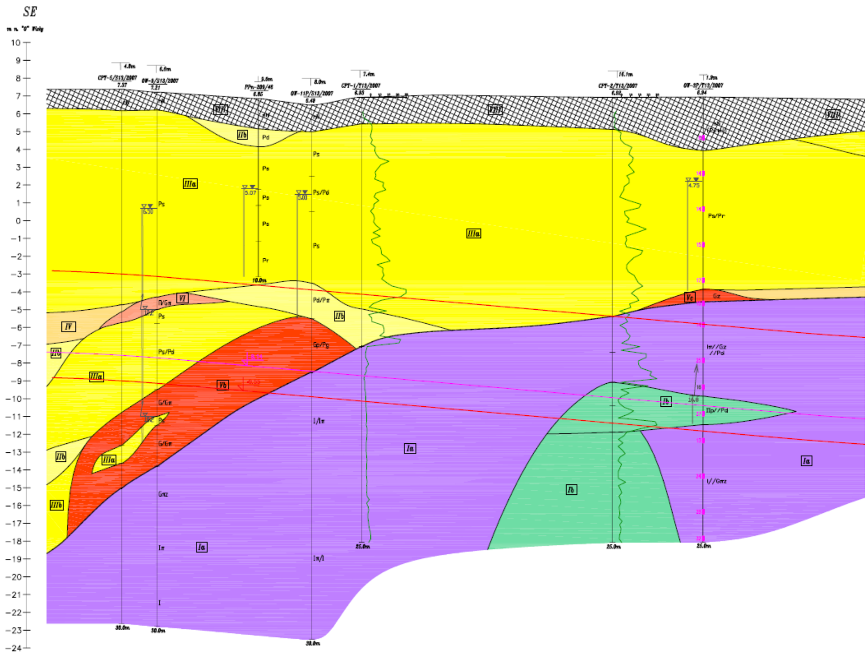


Figure 6.20 Building 128 elevation

Figure 6.21 Building 129 elevation

Figure 6.22 Building 130 and 131 elevations



	Nr warstwy Layer No.	Wiodący rodzaj gruntu Main type of soil	I_D/Dr	I_L/LI	I_p
Trzeciorzęd Tertiary	Ia	I, Iπ, Cz	–	<0.15	>0.4
	Ib	G, Gπ, Π	–	<0.15	<0.4
	Ic	Pπ, Pd, Ps	>0.7	–	–
Czwartorzęd Quaternary	IIb	Pπ, Pd	0.4–0.7	–	–
	IIc		>0.7	–	–
	IIIa	Ps, Pr	<0.7	–	–
	IIIb		>0.7	–	–
	IV	Po, Ż	>0.5	–	–
	Vb	G, Gp	–	<0.25	<0.2
	Vc	Gπz, Gπ	–	<0.2	>0.2
	VI	Pg, Π	–	>0.1	<0.2
	VIIa	Nmp	>0.3	–	–
	VIIb	Nmg	–	>0.4	–
	VIII	nN	–	–	–

Figure 6.23 Cross section of the ground within the grouted area

6.3.2 Results

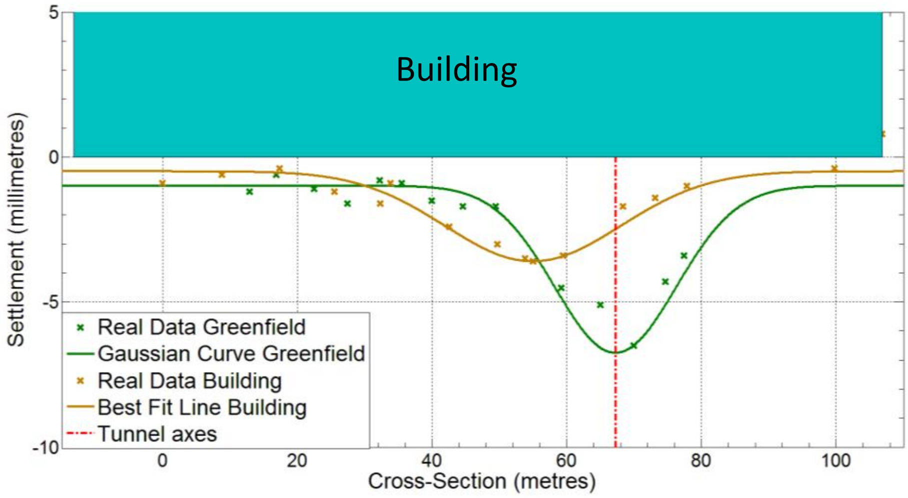


Figure 6.24 Comparison of the green field and building settlements after the first tunnel- Building 1

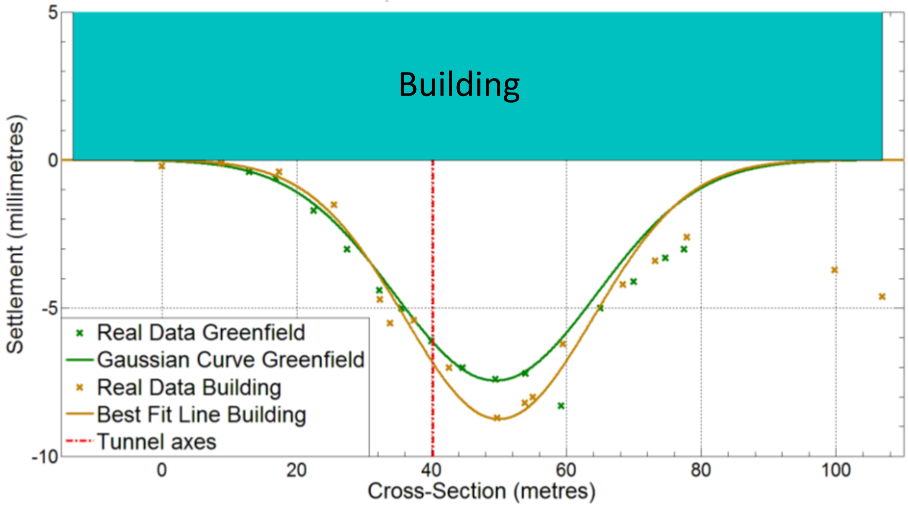


Figure 6.25 Comparison of the green field and building settlements after the second tunnel- Building 1

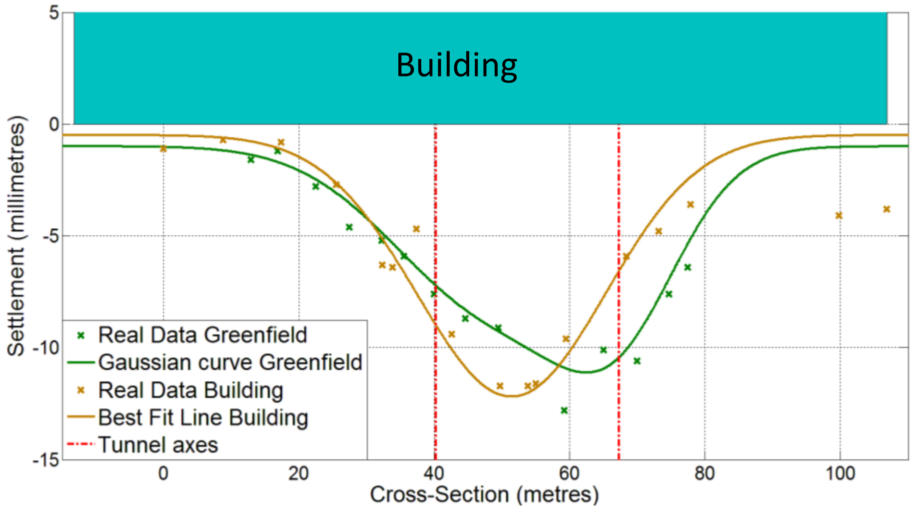


Figure 6.26 Comparison of the green field and building settlements after both tunnels- Building 1

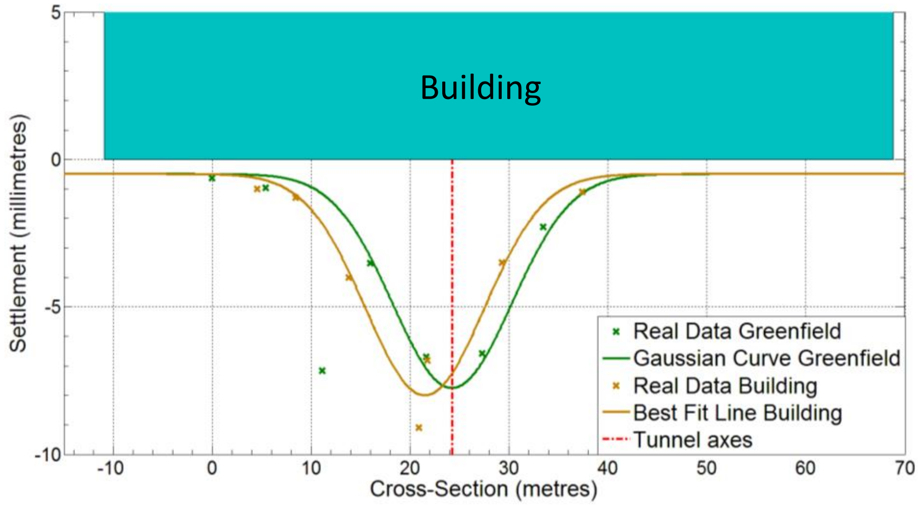


Figure 6.27 Comparison of the green field and building settlement after the first tunnel- Building 2

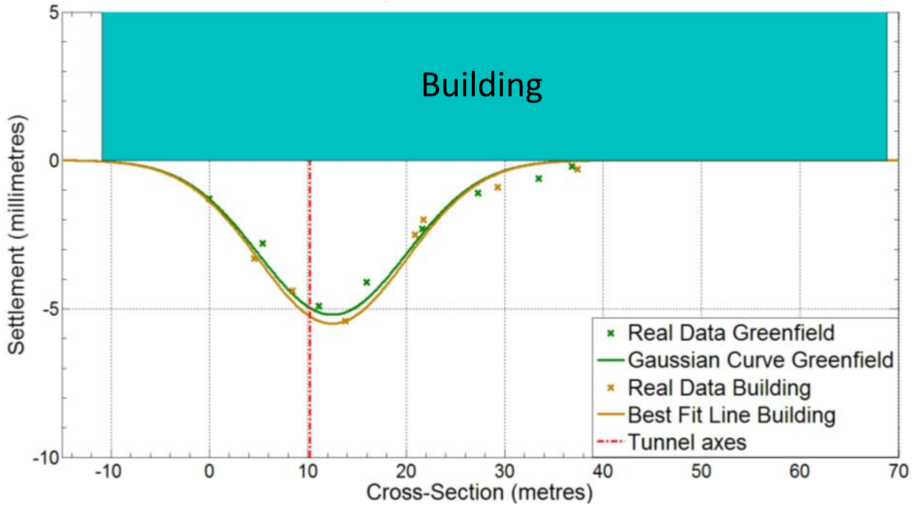


Figure 6.28 Comparison of the green field and building settlement after the second tunnel- Building 2

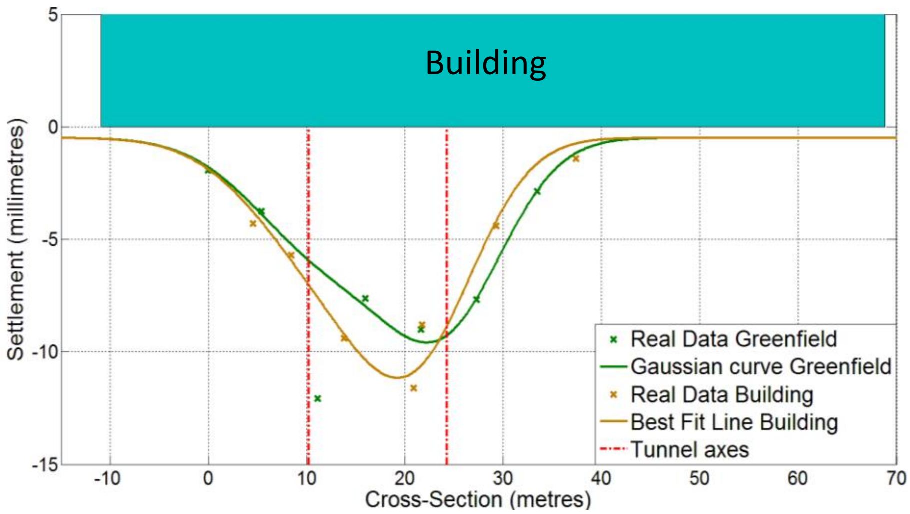


Figure 6.29 Comparison of the green field and building settlement after both tunnels- Building 2

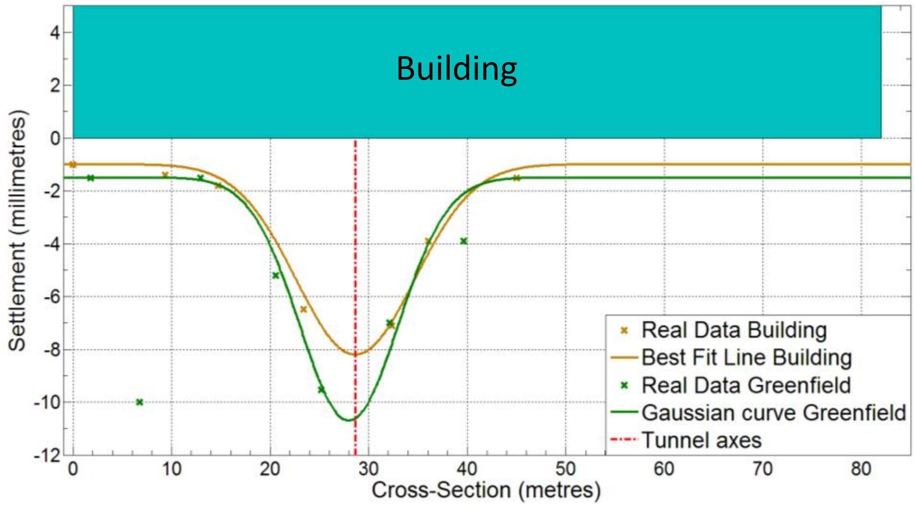


Figure 6.30 Comparison of the green field and building settlement after the first tunnel- Building 3

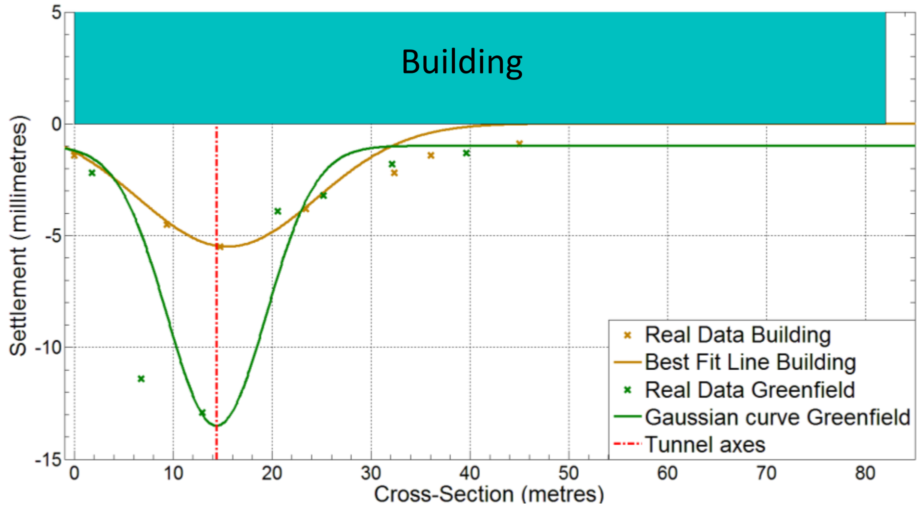


Figure 6.31 Comparison of the green field and building settlement after the second tunnel- Building 3

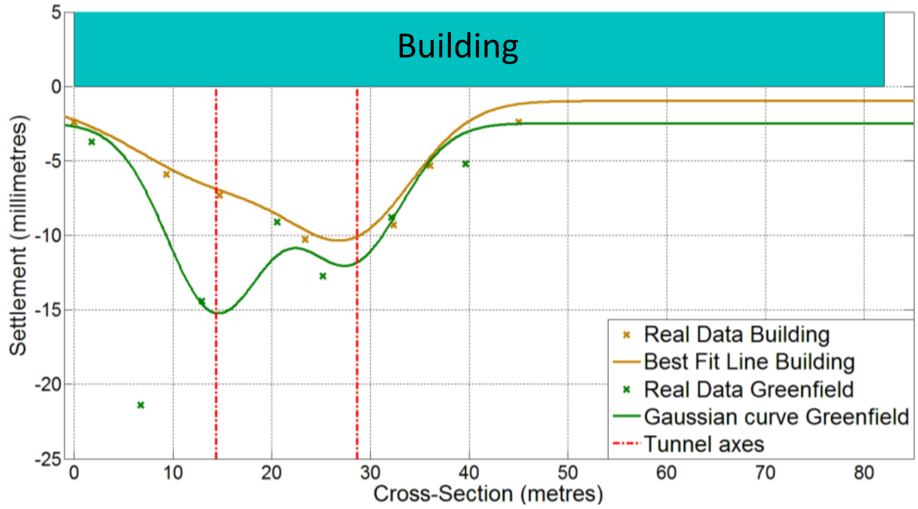


Figure 6.32 Comparison of the green field and building settlement after both tunnels- Building 3

Table 6.1 Volume loss and trough width parameter for the building area

Building	Volume Loss (%)				Trough Width Parameter ,K			
	Tunnel 1		Tunnel 2		Tunnel 1		Tunnel 2	
	Green field	Building	Green field	Building	Green field	Building	Green field	Building
1	0.38	0.30	0.99	1.09	0.55	0.79	0.91	0.85
2	0.36	0.37	0.35	0.37	0.36	0.36	0.45	0.45
3	0.34	0.32	0.51	0.44	0.36	0.30	0.30	0.30

Table 6.2 Modification factor for the building area

Building	**Tunnel**	**Modification Factor**		
		Hogging	**Sagging**	**Hogging**
1	**1**	0.37	0.29	0.36
	2	1.27	1.22	1.27
	Both	1.77	0.99	1.03
2	**1**	1.25	1.08	1.21
	2	1.10	1.04	1.07
	Both	1.99	1.30	1.59
3	**1**	0.63	0.71	0.63
	2	0.08	0.26	0.23
	Both	0.20	0.64	0.64

6.2.4 Discussion

6.3.2.1 Volume loss and trough width parameter

A mixture of levelling pins, mini prisms and ground pins were used to find the settlement of the ground and building and the position of these can be seen in Figure 6.33 and

Figure 6.36. It was important that the green field pins were not too far away from the building in order to perform an accurate comparison.

Firstly, considering the Gaussian curves presented in Figure 6.24 to Figure 6.32 it can be seen that there was an additional settlement which was not caused by the construction of the tunnel. This additional settlement may have been caused by pedestrian and traffic movement nearby and consolidation of the clay. It can also be seen that the tunnels caused greater final building settlement than green field settlement at building 1 and 2, whereas at building 3 there was a greater settlement in the green field instruments. The behaviour of building 3 is what would be expected in the presence of tunnels whereas it is uncommon to see the behaviour portrayed by buildings 1 and 2.

However this behaviour was not observed when analysing the settlement due to individual tunnels. When the first tunnel passed building 1 (Figure 6.24), the maximum settlement was less and the trough width parameter was greater than that of the green field comparison despite the volume loss being similar. This is the expected and preferred behaviour as it is lowering the potential damage to the building by spreading the deflection over a larger length. It can also be seen that the centre of the Gaussian curve of the building did not lie directly on the centre of the tunnel axis. A reason for this could be due to the tunnel approaching the building at an angle however this was taken into consideration when creating the co-ordinate system of the instruments. The co-ordinate system was created on the same angle as the building at a line between the building and green field, the co-ordinates were then taken as the length along this line. In contrast to the first tunnel, the second tunnel showed a greater settlement in the building than the green field instruments. The building had a similar trough width parameter to the first tunnel whereas the trough width parameter for the green field greatly increased. The volume loss of the second tunnel was also over twice that of the first tunnel which would be expected and was seen by Cording and Hansmire (1985) and was also reported by Cooper and Chapman (1998). This is due to the softening of the ground by the first tunnel, therefore if the same standard of workmanship and tunnelling technique was used a greater second volume loss would be expected. In Figure 6.25 there was a larger settlement experienced by one of the instruments however when observing the behaviour of instruments nearby the readings did not support this instrument, therefore it was considered an anomaly due to human error.

Unlike building 1 similar settlements and volume losses were seen in tunnel 1 and tunnel 2 but the trough width parameter was greater for the second tunnel. As stated before, it would be expected that a greater volume loss would be experienced by the second tunnel. This could be due to a change in workmanship standard or tunnelling technique such as a higher face pressure. Considering the behaviour of building 1 and building 2 it may be predicted that building 3 will experience a volume loss of approximately 0.35%

when the first tunnel passes however the volume loss caused by the second tunnel it is not certain due to the large difference between building 1 and building 2.

Building 3 behaves in a very similar manner to building 1 and 2 in terms of volume loss and trough width parameter. However it is clear from Figure 6.30 that the maximum settlement of the building was less than that of the green field. This behaviour is also reflected when the second tunnel passes and Figure 6.31 shows that the settlement is significantly less than the greenfield. The volume loss also increases from tunnel 1 to tunnel 2 as in building 1. The overall behaviour of building 3 after the tunnel has passed suggests that it behaves rigidly which is what would be expected as the walls and floors of the building resist the deformation of the ground.

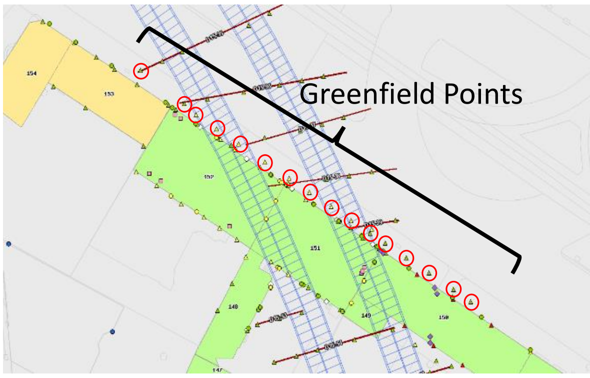


Figure 6.33 Location of green field instruments building 1

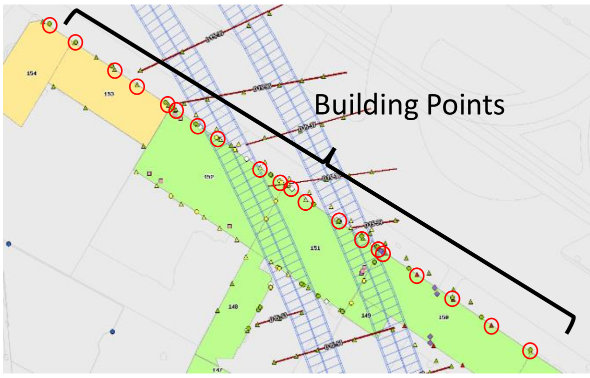


Figure 6.34 Location of building instruments building 1

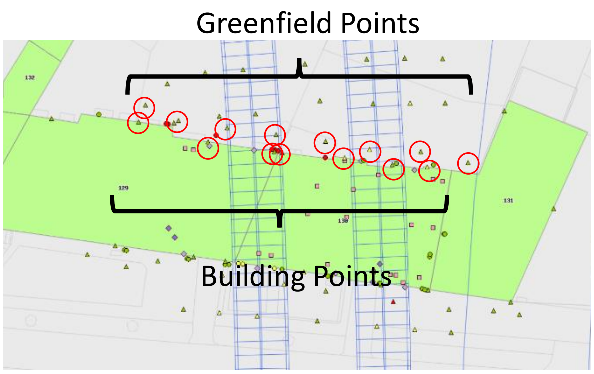


Figure 6.35 Location of instruments building 2

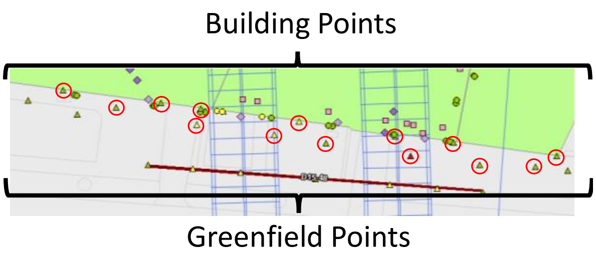


Figure 6.36 Location of instruments building 3

6.3.2.2 Building Deformation

From Table 6.1, the highlighted values are not within the upper and lower design envelopes presented in Figure 6.37 to Figure 6.39. However not all previous experiences have been within these envelopes as demonstrated in Figure 2.6. The values that are within graph limits have been plotted and the relative stiffness's required to obtain these modification factors have been calculated.

The range of building stiffness required for building 1 is from $1.1x10^{-4}$ to $1.3x10^{-3}$, the dominating modification factor for this building is in the hogging zone of the building. However one of the hogging modification factors is not within the design envelope which suggests that the building is acting a lot my flexibly than expected or that there may be some error in the data taken. Building 2 behaved in a similarly manner to building 1 with the hogging zones being the dominating modification factor, however all three modification factors were out of the design envelope. The behaviour of these two buildings support the behaviour suggested by the Gaussian curves found earlier, in that the building is more flexible than the green field assumption.

In contrast, building 3 has modification factors that are within the design envelope and one of the hogging and the sagging have a similar factor therefore giving similar relative building stiffness. However the second modification stiffness is significantly lower resulting in the relative bending stiffness being between $1.65x10^{-4}$ to $1.3x10^{-3}$. This is the same building as that denoted building 2, therefore it suggests that one or both of the settlements are not correct or that the building is demonstrating different behaviour as the tunnel approaches or departs from the building area. This would not be as expected but may be due to the assumption made that the multiple buildings acted as one. They may not act as one building due to the small spaces between either sides of the building, therefore effecting analysis.

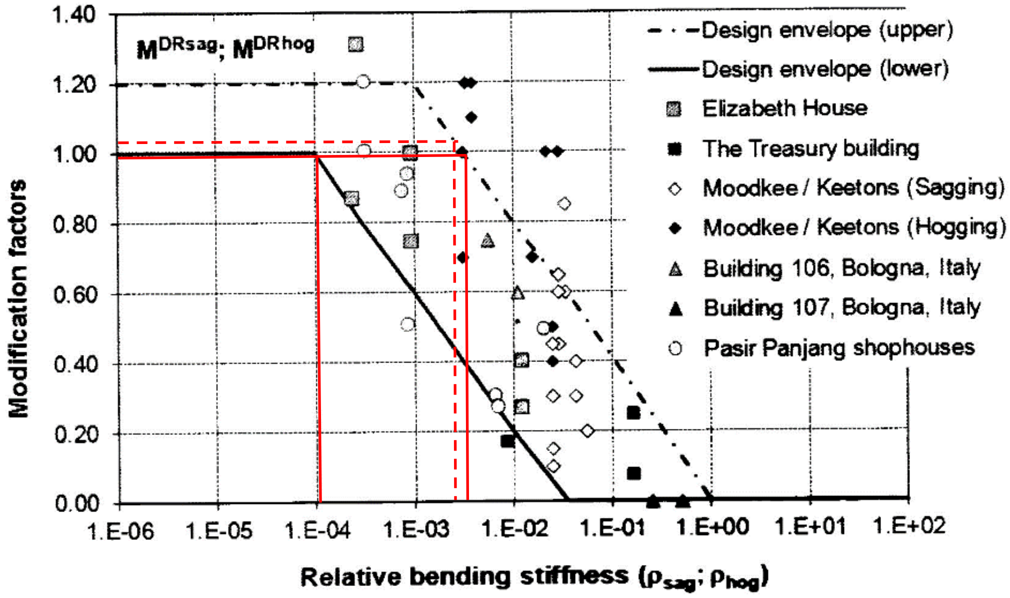


Figure 6.37 Modification factor for building 1- Solid red lines are the modification factor in sagging, dashed red lines are the modification factor in hogging

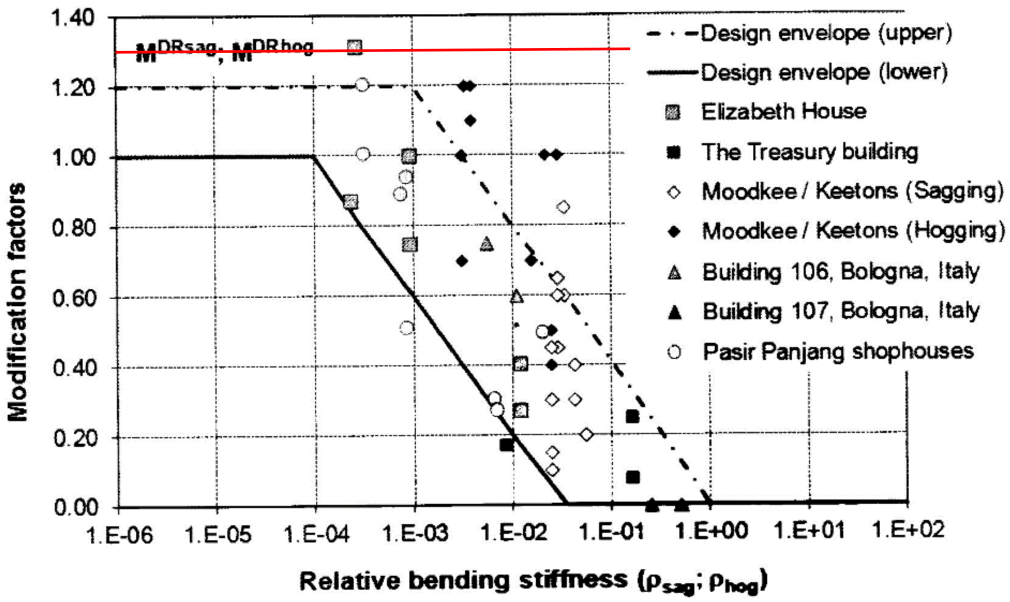


Figure 6.38 Modification factor for building 2- Solid red lines are the modification factor in sagging, dashed red lines are the modification factor in hogging

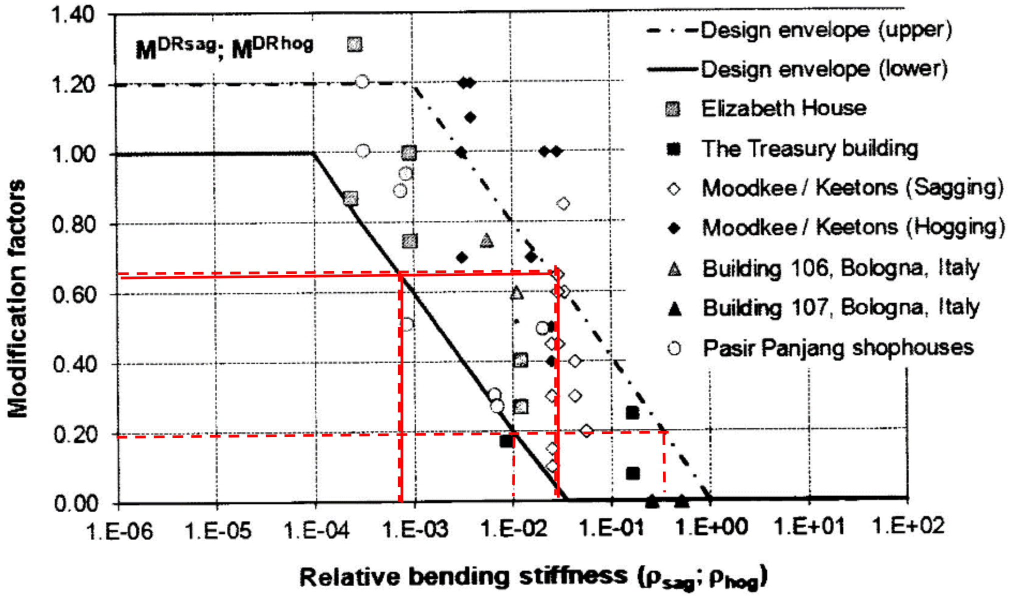


Figure 6.39 Modification factor for building 3- Solid red lines are the modification factor in sagging, dashed red lines are the modification factor in hogging

7. Conclusion

The following conclusions can be made from the investigation regarding the tunnelling of Warsaw within this region of line 2:

- The mixed ground conditions induce a lot of unpredictability in the behaviour of the ground meaning that no trend could be seen clearly from the results gathered. Although it does suggest that in these ground conditions the standard of workmanship influences the ground movement significantly.
- It appeared that grouting did increase the trough width parameter when the first tunnel passed but not the first again suggesting that a high standard of workmanship is needed in mixed ground conditions.
- From the longitudinal settlement it appeared that high face pressures were used in order to maintain a small amount of settlement at the moment of the tunnel passing.
- Analysis indicated that the buildings acted in a generally flexible manner however this may not be the case if they were considered as individual buildings as this would change the deflection ratios.

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Appendix A- Risk Assessment

Reviewing the Risk Assessment which was completed before the project began, I can conclude that the only experienced risk was computer use. However in order to prevent any injury from this a laptop stand and a wireless keyboard and mouse were purchased to ensure that there was no repetitive strain to the body.

Appendix B - Instruments

PIN ID					
D1501CS01	D1510CS07	D1531CS05	D1548CS06	00152SL06	00142MP17A
D1501CS02	D1511CS01	D1531CS06	D1548CS07	00152SL05	00137SL02
D1501CS03	D1511CS02	D1531CS07	D1552CS01	00152SL03	00137SL01
D1501CS04	D1511CS03	D1536CS01	D1552CS02	00152SL02	00134SL05
D1501CS05	D1511CS04	D1536CS02	D1552CS03	00152SL01	D1545CS01
D1501CS06	D1511CS05	D1536CS03	D1552CS04	00153SL05	D1545CS02
D1501CS07	D1511CS06	D1536CS04	D1552CS05	00153SL03	D1545CS03
D1504CS01	D1511CS07	D1536CS05	D1552CS06	00153MP01A	D1545CS04
D1504CS02	D1516CS01	D1536CS06	D1552CS07	00154MP01A	D1545CS05
D1504CS03	D1516CS02	D1540CS01	D1552CS08	00142MP27A	D1545CS06
D1504CS04	D1516CS03	D1540CS02	D1552CS09	00142MP26A	D1545CS07
D1504CS05	D1516CS04	D1540CS03	D1552CS10	00142SL03	00131CS04
D1504CS06	D1516CS05	D1540CS04	D1552CS11	00142SL17	00130CS05
D1504CS07	D1516CS06	D1540CS05	D1552CS12	00142SL16	00130CS06
D1506CS01	D1516CS07	D1540CS06	00150CS04	00142MP30A	00130CS07
D1506CS02	D1520CS01	D1540CS07	00150CS03	00142SL13	00129CS05
D1506CS03	D1520CS02	D1543CS01	00150CS02	00142MP11A	00129CS06
D1506CS04	D1520CS03	D1543CS02	00150CS01	00144MP12A	00129CS07
D1506CS05	D1520CS04	D1543CS03	00151CS05	00144MP06A	00129CS08
D1506CS06	D1520CS05	D1543CS04	00151CS04	00144SL03	00130SL03
D1507CS01	D1520CS06	D1543CS05	00151CS03	00144SL04	00130SL07
D1507CS02	D1520CS07	D1545CS01	00151CS02	00142MP23A	00130SL08
D1507CS03	D1524CS01	D1545CS02	00151CS01	00142SL06	00130SL04
D1507CS04	D1524CS02	D1545CS03	00152CS05	00142MP22A	00129SL07
D1507CS05	D1524CS03	D1545CS04	00152CS04	00142SL07	00129SL03
D1507CS06	D1524CS04	D1545CS05	00152CS03	00142MP21A	00129SL08
D1507CS07	D1524CS05	D1545CS06	00152CS02	00142SL08	00131SL01
D1509CS01	D1524CS06	D1545CS07	00152CS01	00142MP20A	00130SL06
D1509CS02	D1524CS07	D1547CS01	D1536CS06	00142MP20B	00130SL02
D1509CS03	D1528CS01	D1547CS02	00153SL04	00142SL09	00130SL05
D1509CS04	D1528CS02	D1547CS03	00150SL05	00142MP33B	00129SL06
D1509CS05	D1528CS03	D1547CS04	00150SL04	00142MP17A	00129SL02
D1509CS06	D1528CS04	D1547CS05	00150SL03	00142SL10	00129SL05
D1509CS07	D1528CS05	D1547CS06	00150SL02	00142MP16A	00131CS02
D1510CS01	D1528CS06	D1547CS07	00150SL01	00142MP15A	00131CS01
D1510CS02	D1528CS07	D1548CS01	00151SL05	00142MP14A	00130CS04
D1510CS03	D1531CS01	D1548CS02	00151SL04	00142MP02A	00130CS02
D1510CS04	D1531CS02	D1548CS03	00151SL03	00142MP01B	00130CS01
D1510CS05	D1531CS03	D1548CS04	00151SL02	00142MP19A	00129CS04
D1510CS06	D1531CS04	D1548CS05	00151SL01	00142MP18A	00129CS03

Table B.1 Pins used in investigation

Appendix C- Investigation Results

Grouted area is highlighted in red in all tables.

PIN ID		CO-ORDINATE	FIRST SETTLEMENT	FINAL SETTLEMENT	SECOND SETTLEMENT
D1504	CS01	0.000	-	-	
	CS02	3.898	-0.690	-3.720	-3.030
	CS03	8.263	-5.760	-12.490	-6.730
	CS04	15.245	-5.170	-6.930	-1.760
	CS05	22.027	-15.050	-16.980	-1.930
	CS06	26.166	-11.570	-12.600	-1.030
	CS07	30.162	-5.745	-6.495	-0.750
D1506	CS01	0.000	-1.285	-5.700	-4.415
	CS02	4.159	-3.955	-12.440	-8.485
	CS03	8.472	-0.985	-7.800	-6.815
	CS04	15.245	-14.085	-20.600	-6.515
	CS05	21.922	-16.895	-19.500	-2.605
	CS06	26.152	-10.345	-10.200	0.145
D1507	CS01	0.000	-5.925	-6.500	-0.575
	CS02	4.176	-1.215	-9.700	-8.485
	CS03	8.460	-2.405	-13.900	-11.495
	CS04	15.260	-5.470	-10.100	-4.630
	CS05	21.902	-13.030	-14.700	-1.670
	CS06	26.050	-9.950	-10.300	-0.350
	CS07	30.297	-4.990	-4.700	0.290
D1509	CS01	0.000	-1.640	0.600	2.240
	CS02	4.166	-1.550	1.600	3.150
	CS03	8.262	-1.445	1.600	3.045
	CS04	14.788	-2.100	-	-
	CS05	22.644	-4.450	-9.300	-4.850
	CS06	26.118	-4.170	-6.700	-2.530
	CS07	34.857	-12.070	-12.140	-0.070
D1510	CS01	0.000	-3.495	-2.700	0.795
	CS02	4.179	-1.665	-2.300	-0.635
	CS03	8.476	-2.255	-3.200	-0.945
	CS04	15.061	-6.535	-5.800	0.735
	CS05	21.790	-15.515	-15.700	-0.185
	CS06	24.892	-9.760	-9.100	0.660
	CS07	29.089	-10.190	-9.300	0.890
D1511	CS01	0.000	-0.975	0.700	1.675
	CS02	4.108	-1.185	-0.100	1.085
	CS03	8.351	-2.135	-1.700	0.435
	CS04	15.283	1.420	-2.500	-3.920
	CS05	22.157	1.290	-	
	CS06	26.321	-5.625	-3.800	1.825

	CS07	30.379	-1.235	0.300	1.535
D1516	**CS01**	0.000	-0.515	-1.600	-1.085
	CS02	4.230	-0.645	-4.400	-3.755
	CS03	8.362	-5.995	-13.830	-7.835
	CS04	14.046	-7.165	-12.600	-5.435
	CS05	21.723	-14.855	-18.600	-3.745
	CS06	25.745	-12.105	-13.000	-0.895
	CS07	30.361	-6.245	-6.200	0.045
D1520	**CS01**	0.000	-0.755	-3.300	-2.545
	CS02	4.167	-0.225	-6.400	-6.175
	CS03	8.470	-0.455	-10.400	-9.945
	CS04	15.628	-4.385	-10.900	-6.515
	CS05	22.183	-6.915	-8.900	-1.985
	CS06	26.454	-5.365	-6.300	-0.935
	CS07	30.548	-3.545	-3.300	0.245
D1524	**CS01**	0.000	0.100	-5.100	-5.200
	CS02	5.120	-1.200	-10.300	-9.100
	CS03	9.229	-3.000	-15.700	-12.700
	CS04	16.245	-11.200	-22.700	-11.500
	CS05	23.105	-16.900	-21.530	-4.630
	CS06	27.220	-13.800	-17.400	-3.600
	CS07	31.534	-8.600	-12.000	-3.400
D1528	**CS01**	0.000	-4.474	-9.669	-5.195
	CS02	4.289	0.115	-7.900	-8.015
	CS03	9.423	-0.345	-10.200	-9.855
	CS04	14.448	1.100	-9.700	-10.800
	CS05	21.810	-7.600	-10.500	-2.900
	CS06	26.167	-		
	CS07	30.467	-		
D1531	**CS01**	0.000	-5.900	-9.200	-3.300
	CS02	4.798	-3.490	-9.400	-5.910
	CS03	8.908	-2.980	-10.100	-7.120
	CS04	16.182	-8.610	-14.000	-5.390
	CS05	23.088	-7.530	-10.000	-2.470
	CS06	27.456	-7.330	-8.700	-1.370
	CS07	31.595	-13.600	-13.600	0.000
D1536	**CS01**	0.000	-0.265	-1.770	-1.505
	CS02	4.794	-0.325	-3.110	-2.785
	CS03	8.412	-0.295	-4.510	-4.215
	CS04	15.312	-0.685	-5.190	-4.505
	CS05	22.116	-1.615	-5.140	-3.525
	CS06	26.494	-0.045	-2.970	-2.925
D1540	**CS01**	0.000	-0.335	-0.890	-0.555
	CS02	4.186	-1.975	-4.700	-2.725

	CS03	8.128	-3.655	-12.000	-8.345
	CS04	15.473	-4.625	-15.900	-11.275
	CS05	22.666	-4.975	-6.800	-1.825
	CS06	26.783	-4.070	-10.430	-6.360
	CS07	29.849	-1.320	-3.400	-2.080
D1545	**CS01**	0.000	6.9398	-0.200	-7.14
	CS02	4.394	2.5423	-3.600	-10.54
	CS03	7.416	-0.4792	-3.400	-10.34
	CS04	15.882	-8.949	-6.600	-13.54
	CS05	23.208	-16.273	-5.100	-12.04
	CS06	27.249	-20.3127	-3.300	-10.24
	CS07	31.822	-24.8825	-0.700	-7.64
D1547	**CS01**	0.000	-0.850	-0.600	0.250
	CS02	3.960	-0.200	-3.900	-3.700
	CS03	8.131	-0.800	-6.285	-5.485
	CS04	13.052	-2.500	-7.400	-4.900
	CS05	21.409	-6.750	-7.100	-0.350
	CS06	25.507	-4.430	-5.035	-0.605
	CS07	30.669	-2.080	-2.500	-0.420
D1548	**CS01**	0.000	-0.320	-5.775	-5.455
	CS02	4.126	-0.620	-10.520	-9.900
	CS03	8.381	-1.300	-9.300	-8.000
	CS04	15.065	-0.580	-12.915	-12.335
	CS05	21.841	-2.560	-10.680	-8.120
	CS06	26.115	-4.900	-9.600	-4.700
	CS07	30.140	-1.900	-7.400	-5.500
D1552	**CS01**	0.000	0.400	-1.600	-2.000
	CS02	4.105	0.000	-3.000	-3.000
	CS03	7.673	-2.310	-8.110	-5.800
	CS04	11.129	-1.100	-6.000	-4.900
	CS05	14.432	-1.800	-6.600	-4.800
	CS06	17.853	-3.600	-8.300	-4.700
	CS07	21.414	-5.200	-8.700	-3.500
	CS08	25.638	-6.700	-9.000	-2.300
	CS09	30.675	-5.500	-6.500	-1.000
	CS10	36.158	-2.600	3.200	5.800
	CS11	41.136	-0.700	-1.400	-0.700
	CS12	46.512	0.800	0.200	-0.600

Table C.1 Transverse settlements raw data

PIN ID	First Tunnel				Second Tunnel			
	Smax	i	Vs	Vl	Smax	i	Vs	Vl
D1504	-15.100	6.000	-0.227	0.803	-6.750	4.500	-0.076	0.269
D1506	-16.900	5.000	-0.212	0.749	-11.000	7.000	-0.193	0.683
D1507	-13.100	5.500	-0.181	0.639	-11.500	6.000	-0.173	0.612
D1509	-3.000	5.000	-0.038	0.133	-0.950	4.000	-0.010	0.034
D1510	-15.500	6.000	-0.233	0.825	-1.000	6.000	-0.015	0.053
D1511	-7.000	9.000	-0.158	0.559	-1.300	5.000	-0.016	0.058
D1516	-15.000	7.000	-0.263	0.931	-8.000	5.000	-0.100	0.355
D1520	-7.000	7.000	-0.123	0.434	-10.000	6.000	-0.150	0.532
D1524	-17.000	7.200	-0.307	1.085	-13.000	8.000	-0.261	0.922
D1528	-7.700	7.000	-0.135	0.478	-10.200	7.000	-0.179	0.633
D1531	-7.500	11.000	-0.207	0.732	-7.200	10.000	-0.181	0.638
D1536	-2.500	2.000	-0.013	0.044	-4.500	10.000	-0.113	0.399
D1540	-5.500	9.000	-0.124	0.439	-11.300	8.000	-0.227	0.802
D1545	-9.000	4.000	-0.090	0.319	-4.000	7.500	-0.075	0.266
D1547	-7.000	6.500	-0.114	0.403	-5.700	6.000	-0.086	0.303
D1548	-5.000	12.000	-0.150	0.532	-12.500	10.000	-0.313	1.108
D1552	-6.700	7.100	-0.119	0.422	-5.000	9.000	-0.113	0.399

Table C.2 Settlement and volume loss for transverse investigation

PIN ID	K First Tunnel	K Second Tunnel
D1504	0.504	0.402
D1506	0.417	0.603
D1507	0.440	0.513
D1509	0.408	0.325
D1510	0.476	0.484
D1511	0.692	0.385
D1516	0.496	0.316
D1520	0.461	0.405
D1524	0.550	0.503
D1528	0.417	0.405
D1531	0.632	0.583
D1536	0.122	0.588
D1540	0.570	0.500
D1545	0.286	0.514
D1547	0.464	0.417
D1548	0.889	0.704
D1552	0.526	0.652

Table C.3 Trough width parameter for transverse investigation

Building No.	PIN ID	SETTLEMENT AFTER GROUTING	SETTLEMENT AFTER FIRST TUNNEL		SETTLEMENT AFTER SECOND TUNNEL		
			ORIGINAL DATA	TRUE DATA	ORIGINAL DATA	ACCOUNTING FOR GROUTING	ACCOUNTING FOR FIRST TUNNEL
1 GF	00150CS04	0.6	-0.6	-1.2	-1	-1.6	-0.4
	00150CS03	-0.7	-1.3	-0.6	-1.9	-1.2	-0.6
	00150CS02	-0.2	-1.3	-1.1	-3	-2.8	-1.7
	00150CS01	0.4	-1.2	-1.6	-4.2	-4.6	-3
	00151CS05	-2.3	-3.1	-0.8	-7.5	-5.2	-4.4
	00151CS04	-1.4	-2.3	-0.9	-7.3	-5.9	-5
	00151CS03	-0.2	-1.7	-1.5	-7.8	-7.6	-6.1
	00151CS02	-1.6	-3.3	-1.7	-10.3	-8.7	-7
	00151CS01	-1	-2.7	-1.7	-10.1	-9.1	-7.4
	00152CS05	5.1	-5.3	-10.4	-12.5	-17.6	-7.2
	00152CS04	-0.4	-4.9	-4.5	-13.2	-12.8	-8.3
	00152CS03	1.2	-3.9	-5.1	-8.9	-10.1	-5
	00152CS02	-0.8	-7.3	-6.5	-11.4	-10.6	-4.1
	00152CS01	-1	-5.3	-4.3	-8.6	-7.6	-3.3
	D1536CS06	0.7	-2.7	-3.4	-5.7	-6.4	-3
	00153SL04	-	-	-	-	-	-
1 B	00150SL05	1	0.1	-0.9	-0.1	-1.1	-0.2
	00150SL04	1.4	0.8	-0.6	0.7	-0.7	-0.1
	00150SL03	4.2	3.8	-0.4	3.4	-0.8	-0.4
	00150SL02	11	9.8	-1.2	8.3	-2.7	-1.5
	00150SL01	6.3	4.7	-1.6	0	-6.3	-4.7
	00151SL05	4.7	3.8	-0.9	-1.7	-6.4	-5.5
	00151SL04	-0.4	0.3	0.7	-5.1	-4.7	-5.4
	00151SL03	-1.2	-3.6	-2.4	-10.6	-9.4	-7
	00151SL02	-5.8	-8.8	-3	-17.5	-11.7	-8.7
	00151SL01	-9.8	-13.3	-3.5	-21.5	-11.7	-8.2
	00152SL06	-10	-13.6	-3.6	-21.6	-11.6	-8
	00152SL05	-9.3	-12.7	-3.4	-18.9	-9.6	-6.2
	00152SL03	-7.4	-9.1	-1.7	-13.3	-5.9	-4.2
	00152SL02	-5.6	-7	-1.4	-10.4	-4.8	-3.4
	00152SL01	-4	-5	-1	-7.6	-3.6	-2.6
	00153SL05	-	-		-	-	-
	00153SL04	-	-		-	-	-
	00153SL03	-	-		-	-	-
	00153MP01A	0	-0.4	-0.4	-4.1	-4.1	-3.7
	00154MP01A	0	0.8	0.8	-3.8	-3.8	-4.6
2 GF	00131CS04	0.925	0.3	-0.625	-1	-1.925	-1.3
	00130CS05	0.405	-0.55	-0.955	-3.35	-3.755	-2.8
	00130CS06	0.865	-6.3	-7.165	-11.2	-12.065	-4.9
	00130CS07	0.405	-3.115	-3.52	-7.215	-7.62	-4.1
	00129CS05	1.3	-5.4	-6.7	-7.7	-9	-2.3
	00129CS06	1.68	-4.9	-6.58	-6	-7.68	-1.1

	00129CS07	-1.12	-3.4	-2.28	-4	-2.88	-0.6
	00129CS08	-7.76	-5.3	2.46	-5.5	2.26	-0.2
2 B	**00130SL03**	-1.8	-2.8	-1	-6.1	-4.3	-3.3
	00130SL07	0.355	-0.945	-1.3	-5.345	-5.7	-4.4
	00130SL08	-0.125	-4.125	-4	-9.525	-9.4	-5.4
	00130SL04	-0.4	-9.5	-9.1	-12	-11.6	-2.5
	00129SL07	0.35	-6.455	-6.805	-8.455	-8.805	-2
	00129SL03	1.2	-2.3	-3.5	-3.2	-4.4	-0.9
	00129SL08	0.745	-0.355	-1.1	-0.655	-1.4	-0.3
3 B	**00131SL01**	0.5	-0.5	-1	-1.9	-2.4	-1.4
	00130SL06	0.665	-0.735	-1.4	-5.235	-5.9	-4.5
	00130SL02	1	-0.8	-1.8	-6.3	-7.3	-5.5
	00130SL05	0.605	-5.885	-6.49	-9.685	-10.29	-3.8
	00129SL06	0.605	-6.495	-7.1	-8.695	-9.3	-2.2
	00129SL02	1.5	-2.4	-3.9	-3.8	-5.3	-1.4
	00129SL05	0.47	-1.03	-1.5	-1.93	-2.4	-0.9
3 GF	**00131CS02**	-1.4	-2.9	-1.5	-5.1	-3.7	-2.2
	00131CS01	1.4	-8.61	-10.01	-20.01	-21.41	-11.4
	00130CS04	-2.1	-3.61	-1.51	-16.51	-14.41	-12.9
	00130CS02	0.5	-4.7	-5.2	-8.6	-9.1	-3.9
	00130CS01	-0.8	-10.325	-9.525	-13.525	-12.725	-3.2
	00129CS04	-1	-8	-7	-9.8	-8.8	-1.8
	00129CS03	0.6	-3.3	-3.9	-4.6	-5.2	-1.3

Table C.4 Raw data for building investigation

Building No.	First Tunnel					Second Tunnel				
	Smax	i	Intersection	K	Vs (%)	Smax	i	Intersection	K	Vs (%)
1 GF	-5.75	9	1	0.55	0.38	-7.45	15	0	0.91	0.99
1 B	-3.1	13	-0.5	0.79	0.30	-8.75	14	0	0.85	1.09
2 GF	-7.25	6	-0.5	0.36	0.36	-5.2	7.5	0	0.45	0.35
2 B	-8.5	6	-0.5	0.36	0.37	-5.5	7.5	0	0.45	0.37
3 B	7.2	6	-1	0.36	0.34	-5.5	9	0	0.30	0.51
3 GF	-9.2	5	-1.5	0.3	0.32	-12.5	5	-1	0.30	0.44

Table C.5 Settlement, volume loss and trough width parameter for building investigation

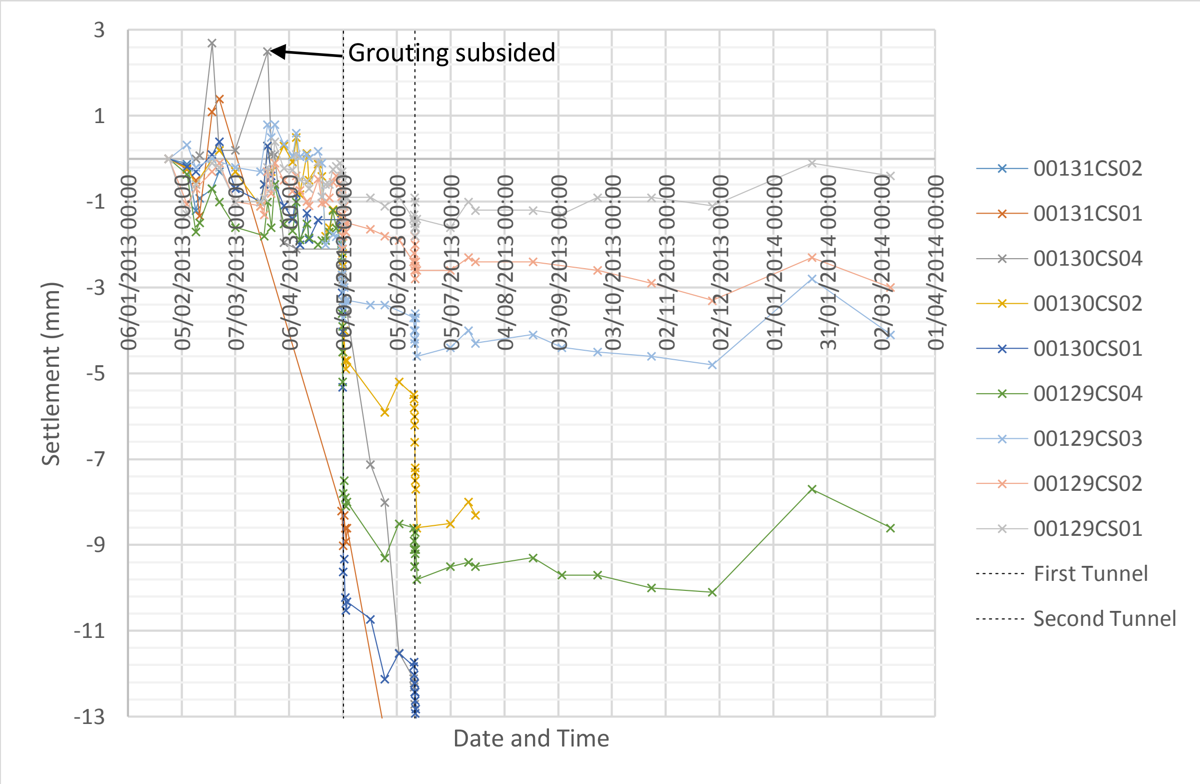

Figure C.1 Settlement against time- Green field, Building 1

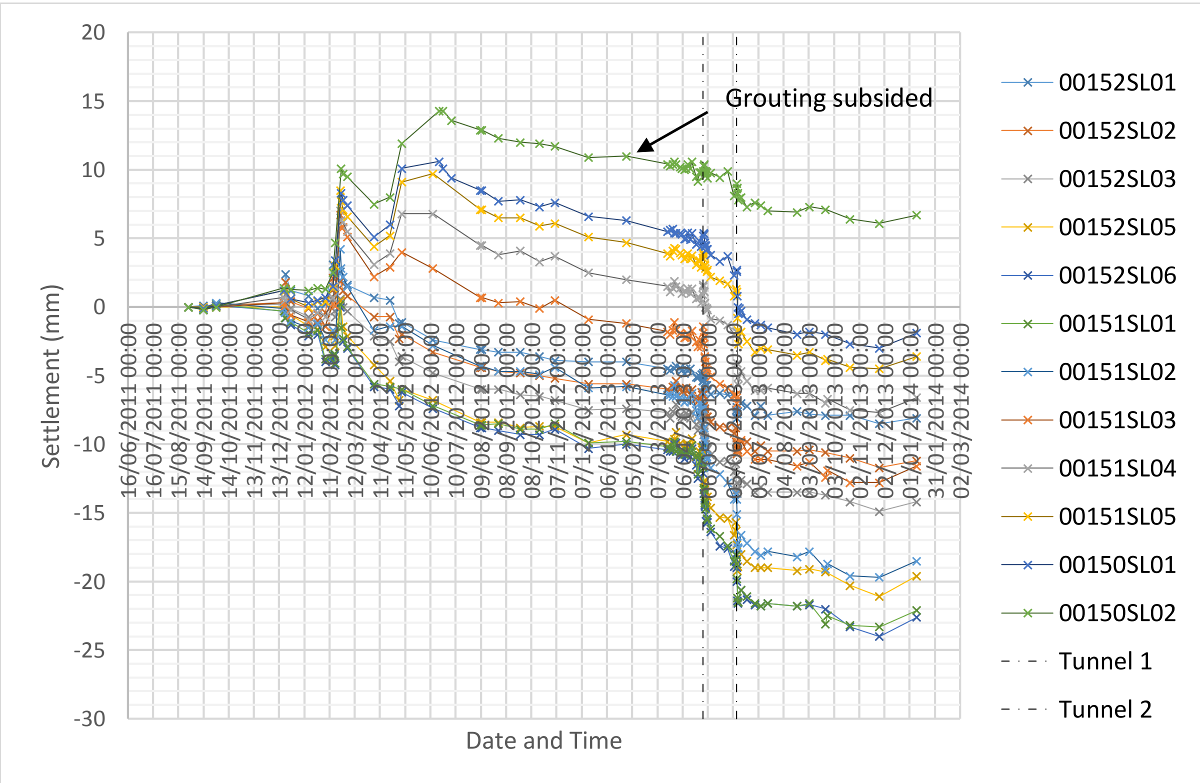

Figure C.2 Settlement against time- Building, Building 1

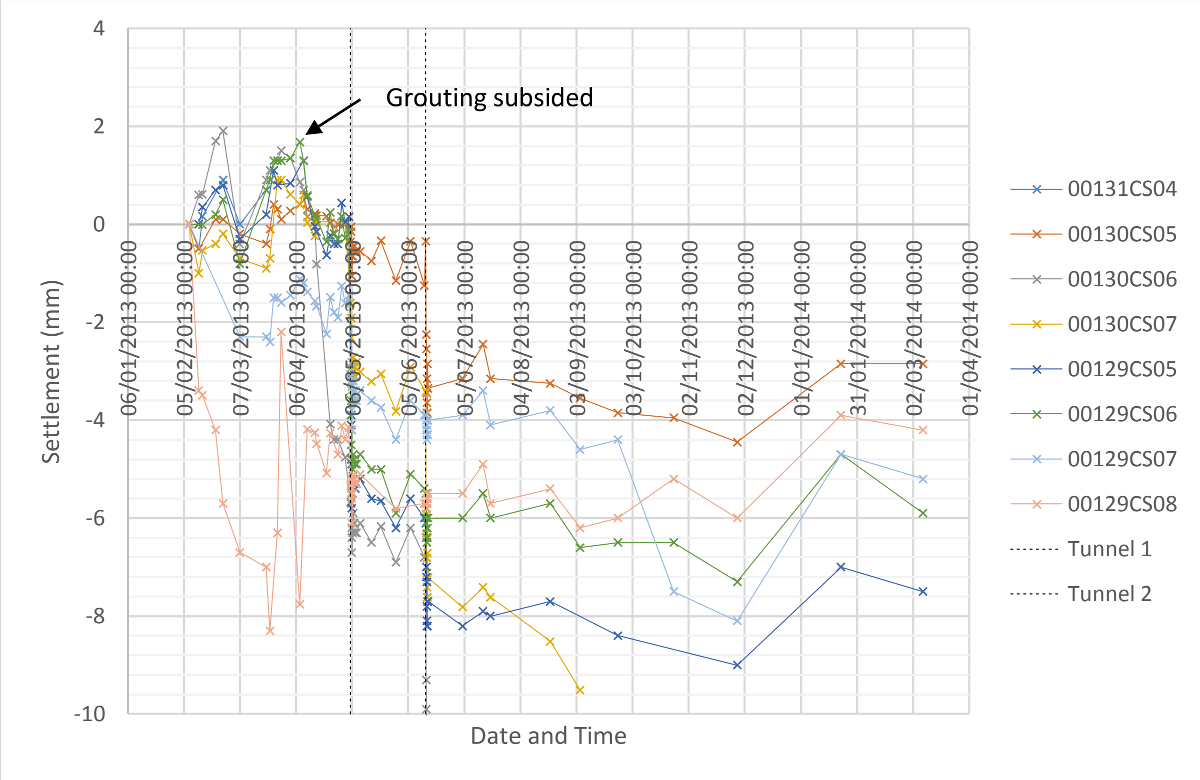

Figure C.3 Settlement against time-Green field, Building 2

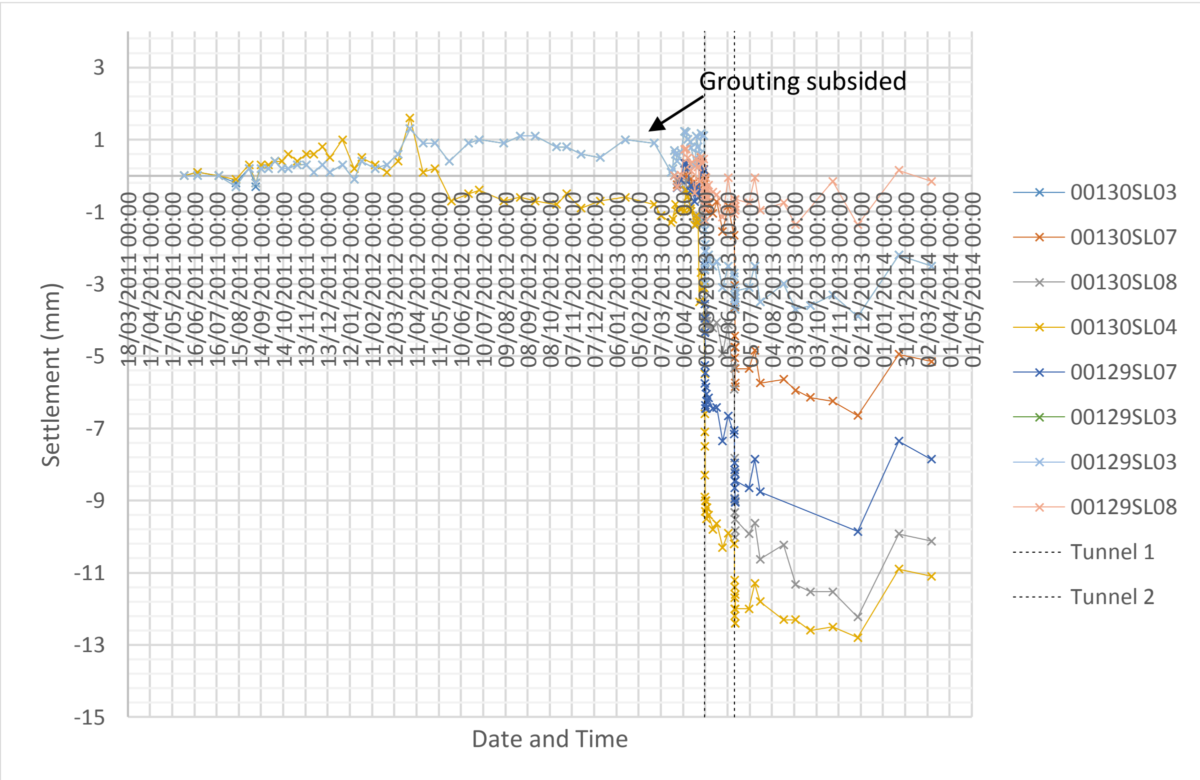

Figure C.4 Settlement against time- Building, Building 2

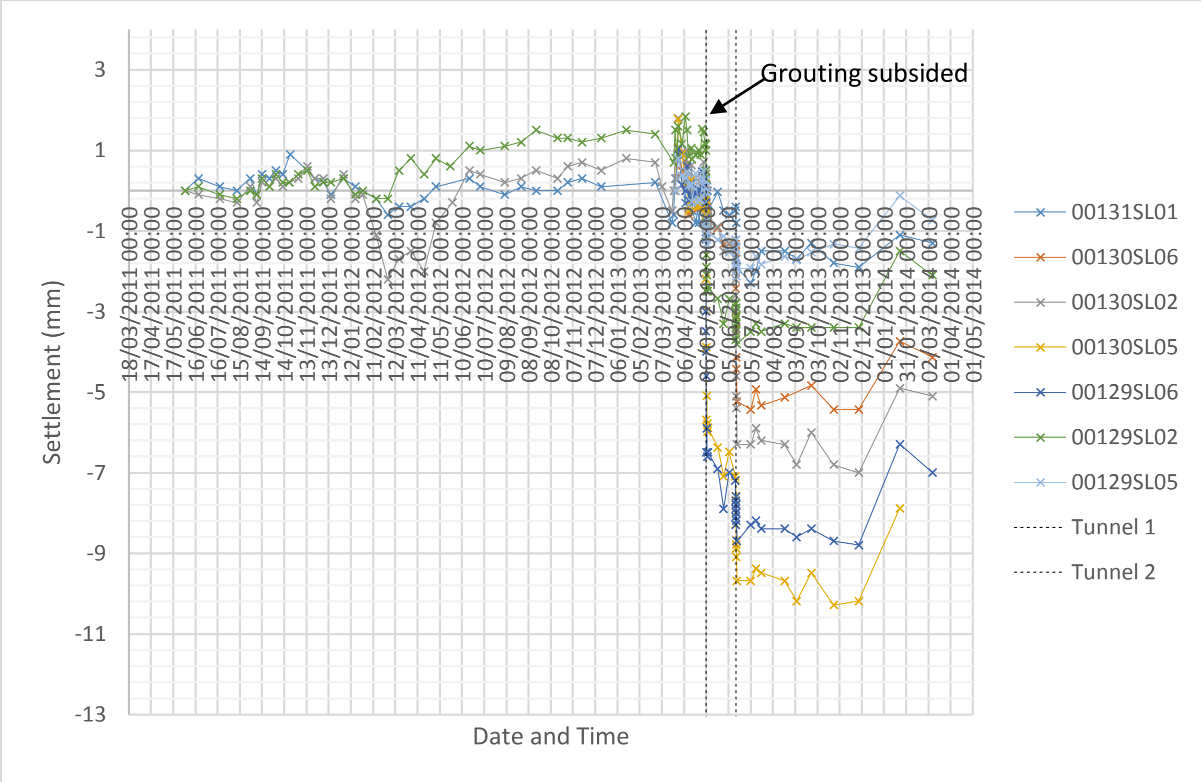

Figure C.5 Settlement against time- Building, Building 3

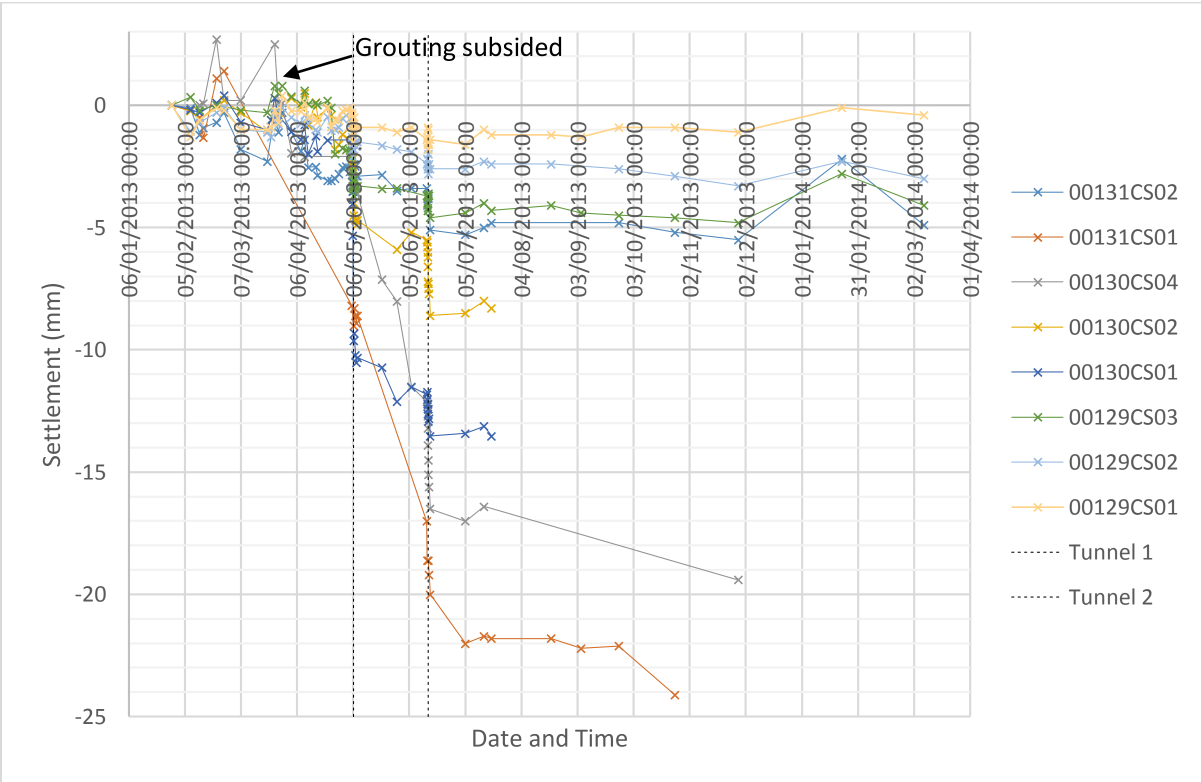

Figure C.6 Settlement against time-Green field, Building 3