UNIVERSITY OF CAMBRIDGE

Department of Engineering

IMPROVING THE ENGINEERING KNOWLEDGE

FROM CROSSRAIL DATA

by

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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."

Signed: .... (Harry Brunton)

Date: ....

ARUP

Crossrail

IMPROVING THE ENGINEERING KNOWLEDGE FROM CROSSRAIL DATA

- SCL TUNNELLING IN LONDON CLAY -

- MEng Project Technical Abstract -

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TA.1 – Outline of Study

Ground movements caused by tunnelling can lead to the damage of local buildings and structures. The extent of the damage depends on the magnitude of the ground movements and, in particular, differential settlements. It is therefore common practice to assess the likely ground movements at the design stage in order to predict the severity of any damage and thus employ reasonable levels of monitoring and mitigation strategies.

Typically, surface settlements are assumed to fit a Gaussian profile, with a maximum settlement (S_{max}) above the tunnel crown. The settlements can be estimated by assuming a volume loss (V_L) value, which is defined as the ratio of the area under the Gaussian curve to the cross-sectional area of the tunnel. V_L is proportional to S_{max}.

The V_L assumed at the design stage is often a conservative approximation based on empirical relationships and needs validating by field data during construction. If the V_L can be estimated more accurately, less conservative values can be assumed in design and a corresponding reduction in monitoring and mitigation (and therefore cost) can be realised.

This study examines the Crossrail tunnelling at the Whitechapel site, focussing on the use of Sprayed Concrete Lining (SCL) tunnels in London Clay (LC). This study uses the recorded ground surface settlement data to determine a representative V_L estimate for the site.

The V_L estimate obtained for the Whitechapel site is reviewed alongside similar case studies of SCL tunnelling in LC. Three case studies are highlighted; Heathrow Trial Tunnels (HTT), Jubilee Line Extension (JLE) and Heathrow Terminal 4 (HT4). This enables comparisons to be drawn between the different projects, with an emphasis on the effect of technological advances and developments in tunnelling methodology on achievable V_L values. The representative Crossrail V_L estimate is also used to populate Macklin (1999)'s Load Factor (LF) graph and thus validate this empirical approach for predicting the V_L.

TA.2 – Report Structure

The main report outlines the aims, objectives, and purpose of the study in greater detail before providing theoretical and practical context for the study. Particular attention is given to the LF approach, the Crossrail project, and similar case studies of SCL tunnelling at LC sites. The Whitechapel data is analysed to determine a representative settlement profile and corresponding V_L, which then facilitates comparisons and a review of current theoretical models and understanding.

TA.3 – Key Results and Conclusions

The representative V_L for the Whitechapel site was found to be 1.06%, based on the surface settlement data during the construction of the eastbound pilot tunnel. The LF was estimated to be 0.41 from the assumed ground conditions and equivalent tunnel properties. This has been used to populate Macklin (1999)'s LF graph.

This study concludes that, whilst providing a good initial estimate for the order of magnitude of the V_L, the LF approach should be treated with caution due to the logarithmic nature of the relationship and variation in V_L observed for case studies with similar LFs. However, for SCL tunnelling in LC the V_L values obtained for the various case studies are similar (about 1-1.1%), with the exception of the HT4 tunnels which have much smaller V_L values. The V_L values for SCL tunnels in LC did not appear to develop over time, despite advances in technology and tunnelling methods. It is proposed that the LF approach should be used in design to determine and initial V_L estimate which must then be reviewed alongside similar case studies to determine a reasonably conservative estimate.

TA.4 - References

MACKLIN, S. R. (1999). The prediction of volume loss due to tunnelling in overconsolidated clay based on heading geometry and stability number. *Ground Engineering, 32, No. 4, pp30-32.*

IMPROVING THE ENGINEERING KNOWLEDGE FROM CROSSRAIL DATA

- SCL TUNNELLING IN LONDON CLAY -

- MEng Project Preliminary Sections -

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i – CONTENTS

ii – NOMENCLATURE

CUED	*Cambridge University Engineering Department*
CSCS	*Construction Skills Certificate Scheme*
EIA	*Environmental Impact Assessment*
HSE	*Health and Safety Executive*
HT4	*Heathrow Terminal 4*
HTT	*Heathrow Trial Tunnels*
JLE	*Jubilee Line Extension*
LC	*London Clay*
LP	*Levelling Point*
MG	*Made Ground*
NATM	*New Austrian Tunnelling Method*
PPE	*Personal Protective Equipment*
RTD	*River Terrace Deposits*
SCL	*Sprayed Concrete Lining*
TBM	*Tunnel Boring Machine*
C	*Cover [m]*
D	*Equivalent Tunnel Diameter [m]*
i	*Point of Inflection [m]*
K	*Trough Width Parameter*
LF	*Load Factor*
N	*Stability Ratio*
N_c	*Critical Stability Ratio*
P	*Unsupported Cut Length [m]*
S_v	*Vertical Surface Settlement [mm]*
S_{max}	*Maximum Vertical Surface Settlement [mm]*
s_u	*Undrained Shear Strength [KPa]*
V_L	*Volume Loss [%]*
V_s	*Volume of Surface Settlement Trough* [m^3/m]
V_T	*Tunnel Face Volume* [m^3/m]
y	*Horizontal Distance from Tunnel Centreline [m]*
z_o	*Tunnel Axis Depth [m]*
γ	*Unit Weight of Soil* [KNm^{-3}]
ε_{max}	*Maximum Tensile Strain [%]*
σ_s	*Surcharge Pressure [KPa]*
σ_v	*Total Vertical Stress [KPa]*
σ_v'	*Effective Vertical Stress [KPa]*

iii – KEYWORDS

Tunnels; Crossrail; Sprayed Concrete Linings; Surface Settlement; Volume Loss; Load Factor

IMPROVING THE ENGINEERING KNOWLEDGE FROM CROSSRAIL DATA

- SCL TUNNELLING IN LONDON CLAY -

- MEng Project Final Report -

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1 – INTRODUCTION

1.1 – Aims and Purpose

This study will ultimately form part of a larger investigation into the tunnelling impacts of London's Crossrail development. There is a continuing need to improve the current understanding of tunnelling impacts on existing structures and validate common design assumptions and procedures. Tunnelling schemes are often large-scale projects and are therefore regularly associated with high costs and significant risk levels, hence requiring comprehensive and costly mitigation measures. Although there have been major developments in tunnelling in recent decades, there remain numerous opportunities for substantial improvements with associated financial savings. This potential to contribute to the wider field has been the primary driver behind this study, and forms the basis of the study's main aims. These are:

- to inform the industry and clients about the potential ground movements associated with tunnelling (particularly using the sprayed concrete lining (SCL) method);
- to revisit previous case studies and reassess common knowledge;
- to enable better decision making using the information from recent projects, and potentially highlight future savings.

1.2 – Focus

This study focusses on SCL tunnels in London Clay (LC). Such tunnels feature in several Crossrail sections, providing numerous opportunities for detailed investigation. This study concentrates on the Whitechapel site. The key parameters are the ground surface settlement and corresponding volume loss (V_L) (defined as the volume of the settlement trough as a percentage of the tunnel volume; see Section 3.2). This report describes how these parameters relate to the possible damage of structures within the zone of influence of the tunnel and corresponding protective measures that are required to mitigate the impacts of tunnelling. By improving the understanding of V_L and correlating this to field observations, better estimates for ground settlement can be obtained. This should enable engineers to make better decisions, potentially adopt less conservative design approaches for future projects, and therefore reduce costs.

1.3 – Objectives

Primary objectives were established at the beginning of the study, and continually reviewed and refined throughout in collaboration with the Geotechnical Research Group at Cambridge University and the industrial partner, Ove Arup Ltd. The objectives provided tangible targets for the study, and were developed in line with the main aims. Ultimately, the main objectives of the study were confirmed as:

- to analyse Crossrail settlement data to obtain a representative Crossrail V_L associated with SCL tunnelling;
- to populate existing theoretical models with the new representative Crossrail V_L from a specific site;
- to draw comparisons between Crossrail and previous case studies;
- to draw comparisons between the different SCL tunnelling approaches adopted in practice.

2 – BACKGROUND

2.1 – SCL Tunnelling in London Clay

SCL tunnelling (sometimes referred to as the New Austrian Tunnelling Method (NATM)) is best suited to excavations in stiff clay. Excavation begins by making an unsupported cut, typically a heading of about 1-2m, using a backhoe excavator (Figure 2.1.1a). Reinforcement is positioned around the cut perimeter, which is then sprayed with a layer of concrete, typically about 350mm thick. SCL tunnelling, unlike Tunnel Boring Machines (TBMs), is particularly suited to the construction of irregular-shaped excavations. Historically, the concrete is sprayed manually by an operator; however recent technological developments have enabled the SCL to be applied robotically (Figure 2.2.1b).

Figure 2.1.1a: Backhoe excavation of SCL tunnel on Crossrail project.

Figure 2.1.1b: Crossrail robotic SCL machine.

For larger tunnels it is often necessary to carry out the excavation in stages. This can be achieved in several different ways, examples of which are explored in Section 4 and Section 6.2 for case study examples and the Crossrail project respectively. Three previous case studies of SCL tunnels in London Clay have been identified as being particularly pertinent to this study; Heathrow Trial Tunnels (HTT), Jubilee Line Extension (JLE), and Heathrow Terminal 4 (HT4).

2.2 – Tunnelling Impacts

The most tangible impact of tunnelling is damage to structures and services at ground level, as well as other tunnels, foundations, and excavations beneath the surface. This damage is caused by vertical and lateral ground movements during the tunnel construction, and further ground movements in the long term. A particular concern is differential settlement, which leads to tensile strains in the building and potentially cracking of the structure. Mair et al (1996) assessed the potential impact of tunnelling on buildings, reporting on the work of Burland et al (1977), who classified visible damage, and the work of Boscardin & Cording (1989), who related the tensile strains in a building to the potential damage categories. Mair et al (1996) recommended a three-stage approach to assessing the risk of damage whereby buildings subjected to a settlement of less than 10mm are eliminated from further assessment at an initial stage, whilst the remainder are subjected to a tensile strain assessment and, in the most serious of cases, a detailed assessment.

The anticipated severity of damage for a particular structure is used to determine the level of mitigation required, if any. It is common practice to install instrumentation for medium and high risk cases, with the level of instrumentation dependent on the foreseen risk and possible consequences. In the most critical cases, it might be appropriate to inject grout into the soil to correct for displacements caused by the excavation. This is carried out in conjunction with the monitoring of ground movements so that the grout can be injected in a controlled manner. Both instrumentation and grouting have high financial costs, and therefore reducing the need for these processes could realise significant cost savings.

3 – REVIEW OF CURRENT UNDERSTANDING AND PRACTICE

3.1 – Ground Surface Settlement – The Gaussian Distribution

Irrespective of the method of construction, tunnelling inevitably leads to ground movements in the surrounding soil. The ground above the tunnel is liable to settle vertically, whilst the surrounding soil also moves laterally towards the centre of the excavation. The magnitudes of the ground movements vary between projects and depend upon factors including; the ground conditions, method of excavation, tunnel properties, and quality of workmanship.

This study focuses on ground surface settlements. Mair & Taylor (1997) summarised much of the theoretical understanding of tunnels. In particular, they reported that the work of Peck (1969), Schmidt (1969), and numerous other authors had shown that the ground surface settlements can be approximated to a Gaussian profile, with a maximum settlement (S_{max}) above the tunnel crown (Figure 3.1.1). It should be emphasised that these are vertical settlements, and occur along a section which is perpendicular to the direction of the tunnel in plan-view. It is also possible to obtain similar settlement profiles for sub-surface settlements and horizontal ground movements, as presented by Mair & Taylor (1997) following the work of various authors, but these are not of immediate concern to this study.

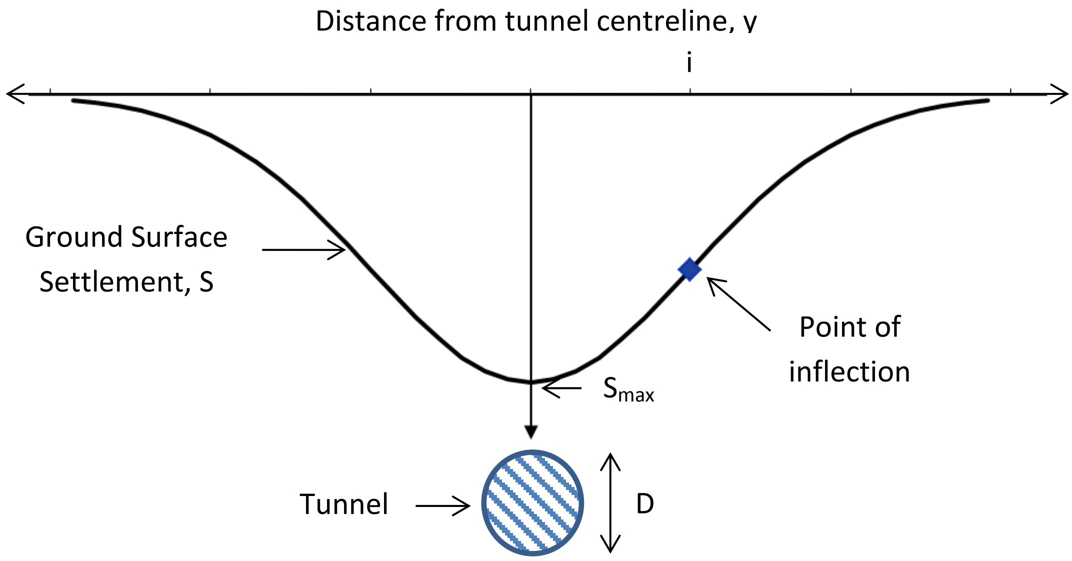


Figure 3.1.1: Gaussian settlement profile above a tunnel. (Note: not to scale - maximum settlement typically 3 orders of magnitude less than tunnel diameter)

The shape of the Gaussian settlement profile in Figure 3.1.1 is related to the maximum settlement and the lateral distance from the tunnel axis (Equation 3.1.1):

$$S_v = S_{max} exp\left({}^{-y^2}/_{2i^2}\right) \tag{3.1.1}$$

In practice, the maximum settlement is either estimated or taken from ground observations. O'Reilly & New (1982) presented data for cohesive and granular soils to show the relationship between the horizontal distance to the point of inflection (i) and the depth of the tunnel axis (z_0). They concluded that, as the linear regression lines for both soil types they examined passed close to the origin, the relationship could be simplified to:

$$i = Kz_0 \tag{3.1.2}$$

The value of the trough width parameter (K) is dependent on the soil. O'Reilly & New (1982) suggested a value in the range of 0.4 (stiff clays) to 0.7 (soft, silty clay) would be appropriate for cohesive soils.

3.2 – Volume Loss

Mair & Taylor (1997) also reported thee definition of V_L as the trough volume (V_s) as a percentage of the tunnel face area (V_T) (Equation 3.2.1). V_s is the area under the Gaussian curve, found by integrating Equation 3.1.1.

$$V_L = \frac{V_S}{V_T} \times 100\% = \frac{\sqrt{2\pi}i\left({S_{max}}/{1000}\right)}{\left({\pi D^2}/{4}\right)} \times 100\% \tag{3.2.1}$$

3.3 – The Load Factor Approach

Obtaining an accurate V_L prediction has been the subject of many studies reported in the literature. Mair et al (1981) used centrifuge data and finite element analyses to relate the V_L to a dimensionless parameter called the load factor (LF). They defined the LF as the stability ratio (N) divided by the stability ratio at collapse (N_c) (Equation 3.3.1), where N took the original definition proposed by Broms and Bennermark (1967). However, for SCL tunnels, the tunnel support pressure is zero and therefore the definition of N can be simplified (Equation 3.3.2). The inverse of the LF can be used as a measure of safety.

$$LF = \frac{N}{N_c} \tag{3.3.1}$$

$$N = \frac{\sigma_s + \gamma z}{s_u} \tag{3.3.2}$$

Mair (1979) used centrifuge tests to show that that N_c could be related to three important tunnel properties; the cover (C), equivalent diameter (D), and length of unsupported cut (P). He presented graphical empirical relationships, showing that N_c was larger for higher values of C/D and lower values of P/D. The graphs were reproduced by Mair & Taylor (1997) (Figure 3.3.1).

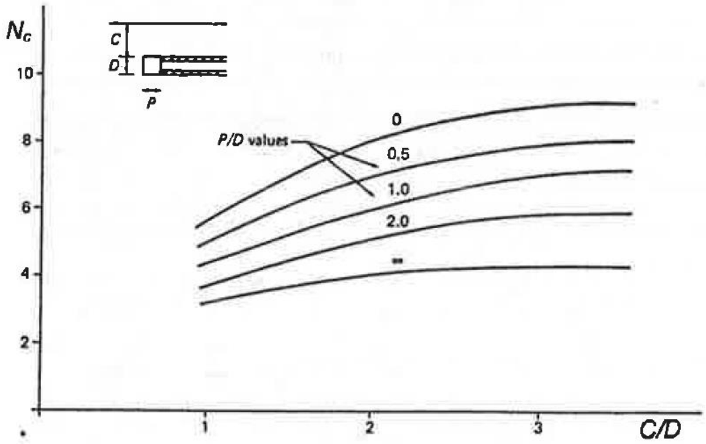


Figure 3.3.1: Relating critical stability ratio to tunnel geometry. (Source: Mair & Taylor (1997))

Macklin (1999) collated V_L and LF data from case studies and laboratory work (primarily centrifuge tests). He concluded that the relationship between V_L and LF was exponential, presenting the data in graphical form. Here, this graph has been digitised and reformatted to highlight which data points correspond to case study data (Figure 3.3.2).

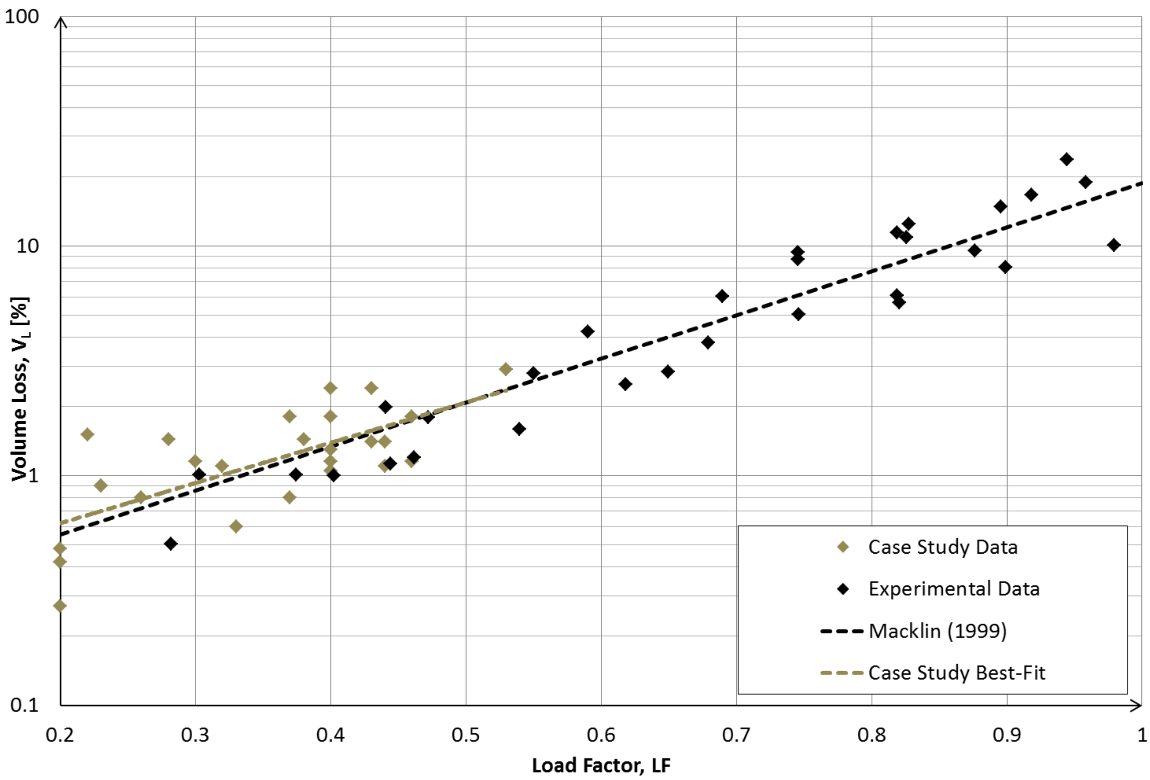


Figure 3.3.2: Case study and experimental data showing empirical relationships between volume loss and load factor. (Adapted from Macklin (1999))

It can be seen from Figure 3.3.2 that the case study data V_L data is in the range of 0.27% to 2.96%, whilst the theoretical centrifuge models have data corresponding to much higher V_L and LF values. Macklin (1999) established a best-fit approximation for all the data, as shown in Figure 3.3.2 and described by Equation 3.3.2. An additional exponential best-fit has been added to Figure 3.3.2, representing the case study data only. This line is described by Equation 3.3.4.

$$V_L = 0.23exp(4.4LF) \qquad (3.3.3)$$

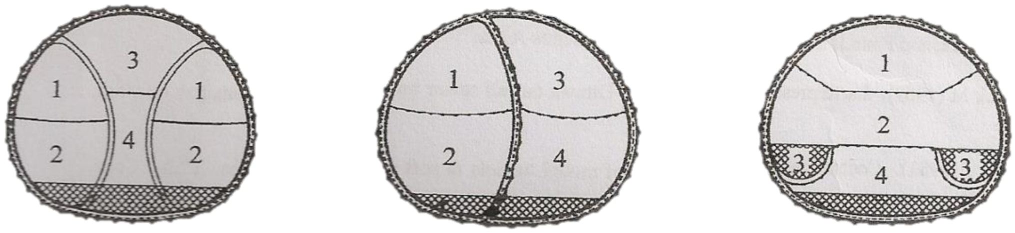
$$V_L = 0.28exp(4.0LF) \qquad (3.3.4)$$

It can be seen from Figure 3.3.2 that the best-fits are similar in the most densely populated part of the graph, with Equation 3.3.4 providing a more conservative V_L estimate for smaller LFs. This is possibly a more appropriate best fit line to use in design as it is based entirely on case history data. As more case study data becomes available, it is anticipated that Figure 3.3.2 can be populated further to develop a more accurate empirical relationship between V_L and LF. Laboratory studies can be used to validate the relationship, but field data should eventually be adopted as the most reliable information once sufficient data is available. This study will aid this development by providing the new Crossrail data.

4 – PREVIOUS CASE STUDIES

4.1 - Heathrow Trial Tunnels (HTT)

The HTT project was the first use of the SCL approach in LC, involving the construction of three 30m long tunnels, with an equivalent diameter of 8.66m and axis depth of 21m, between January and May 1992 (Bowers et al (1996)). The three tunnels were used to examine three different subheading construction sequences (Figure 4.1.1).



Type 1

(Twin sidewall drifts)

Type 2

(Single sidewall drifts)

Type 3

(Crown bench & invert)

Figure 4.1.1: HTT excavation sequences. (Source: Bowers et al (1996))

New & Bowers (1994) reported on the HTT short term surface settlement for each tunnel type. They used a non-linear regression analysis routine to determine values for S_{max}, V_L, and K. These are reproduced in Table 4.1.1.

TABLE 4.1.1: SUMMARY OF HTT GROUND SURFACE SETTLEMENT (Data from: New & Bowers (1994))			
Subheading Sequence	Maximum Settlement, S_{max} [mm]	Volume Loss, V_L [%]	Trough Width Parameter, K
Type 1	27.9	1.15	0.45
Type 2	26.8	1.05	0.45
Type 3	40.3	1.26	0.35

4.2 – Jubilee Line Extension (JLE)

The JLE project used SCL tunnelling in LC for twin tunnel sections at Elizabeth House, Waterloo between January and March 1995. Dimmock & Mair (2007a) examined the 5.6m diameter eastbound tunnel, which was constructed using the heading and invert excavation sequence (Figure 4.2.1).

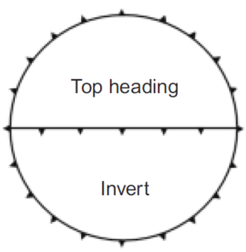


Figure 4.2.1: JLE SCL excavation sequence. (Source: Dimmock & Mair (2007a))

Dimmock & Mair (2007a) reported a total V_L of 1.1%, of which approximately 0.8% was attributed to ground movement ahead of the tunnel lining. These values align with those of Standing (2001), who studied both the eastbound and westbound tunnels, estimating a total V_L of 1.3% for both tunnels combined and a representative V_L (for one tunnel) of 1.1%.

4.3 - Heathrow Terminal 4 (HT4)

Van der Berg et al (2003) examined the HT4 concourse tunnel and platform tunnels, constructed using a heading bench and invert SCL tunnelling approach (Figure 4.3.1) in LC between May 1994 and November 1996. A typical V_L for the concourse tunnel of 0.8% was reported. Van der Berg (1999) initially examined the ground surface settlements of both the concourse and platform tunnels, suggesting a V_L between 0.5% and 0.6% for the platform tunnel.

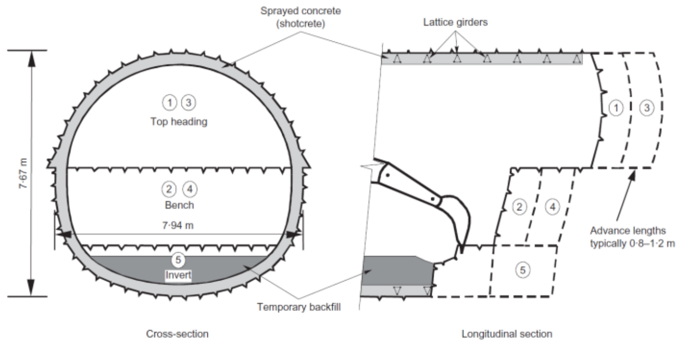


Figure 4.3.1: HT4 SCL construction sequence. (Source: Van der Berg et al (2003))

5 – CROSSRAIL (NEW CASE STUDY PRESENTED IN THIS REPORT)

5.1 – Crossrail Programme

Crossrail presents itself as the largest construction project in Europe (Crossrail 2014a), with a budget of £14.8bn and the intention to expand London's rail capacity by 10% to link Reading and Heathrow in the West to Shenfield and Abbey Wood in the East (Figure 5.1.1). Groundwork began in May 2009 and rail services are due to commence running from 2018. The scheme will incorporate a total of 42Km of new tunnels.

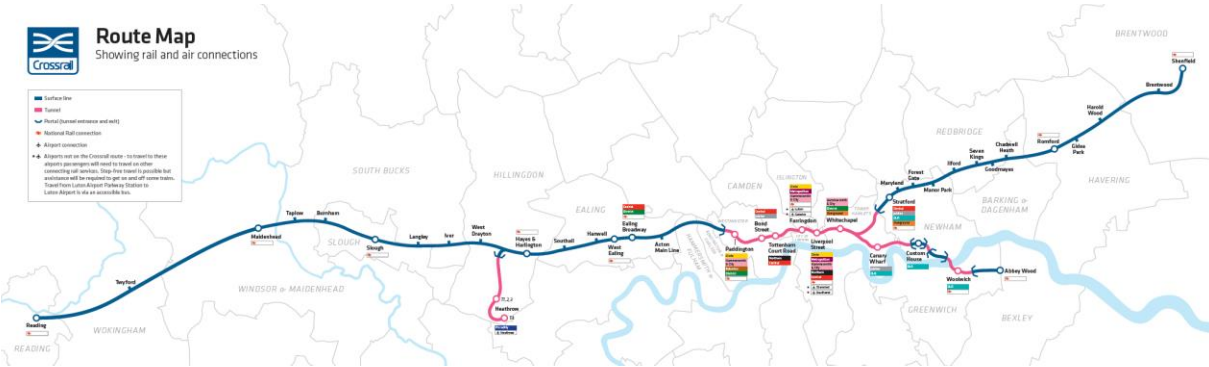


Figure 5.1.1: Crossrail Route (Source: Crossrail (2014b))

5.2 – Site Selection

Although the majority of the Crossrail tunnels are being constructed with TBMs, SCL tunnelling is being adopted along some sections. In order to obtain useful and representative ground surface settlements, it is appropriate to study sections which have not adopted grouting techniques. On this basis, the Whitechapel site has been selected for this study as it adopts the SCL tunnelling approach in LC and has only limited compensation grouting, thus providing conditions most similar to that of a greenfield site.

5.3 – Crossrail Design Approach

Crossrail's Ground Settlement Information Paper (Crossrail (2008)) details Crossrail's plan to adopt a version of the three phase approach discussed in Section 2.2 in order to determine the potential damage to buildings caused by tunnel construction. The paper specifies that the initial stage is to determine the predicted ground movements and then deduce the potential building damage. Five risk categories are identified, alongside corresponding values of maximum tensile strain (ε_{max}) (Table 5.3.1). The paper states that buildings in category 3 and above will be subject to extensive monitoring, whilst general background surface monitoring will be undertaken at all locations. The report identifies the need for good tunnelling practice in order to minimise ground movements, but also indicates the need for intrusive mitigation (e.g. grouting) for the more severe cases. Crossrail's Kempton Court Phase 3 Report (Crossrail (2011a)) indicates how the magnitudes of the ground settlements at the Whitechapel site were predicted at the design stage. For the SCL platform tunnels, the focus of this study, the document states that a V_L of 1.5% was assumed, and ground movements estimated accordingly. This V_L is noted as being moderately conservative and in accordance with Crossrail's Civil Engineering Design Standards. However, beyond meeting this legislative requirement, little information is provided to support the 1.5% V_L assumption. The new Crossrail data presented in this report is a direct attempt to validate and investigate this assumption.

TABLE 5.3.1: CROSSRAIL BUILDING DAMAGE CLASSIFICATION
(Adapted from: Crossrail (2008))

Risk Category	Description of Degree of Damage	Maximum Tensile Strain, ε_{max} [%]	Approximate Crack Width [mm]
0	Negligible	< 0.05	<< 0.1
1	Very Slight	0.05 → 0.075	0.1 → 1
2	Slight	0.075 → 0.15	1 → 5
3	Moderate	0.15 → 0.3	5 → 15, or many > 3
4	Severe	> 0.3	15 → 25
5	Very Severe	> 0.3	> 25

6 - THE WHITECHAPEL SITE

6.1 – Location Examined

The Whitechapel site forms one of the central stations along Crossrail's route. The groundwork includes the construction of the main platform tunnels (eastbound and westbound) as well as a series of cross passages, escalator tunnels, and shafts. A 3D section through the site is given in Figure 6.1.1.

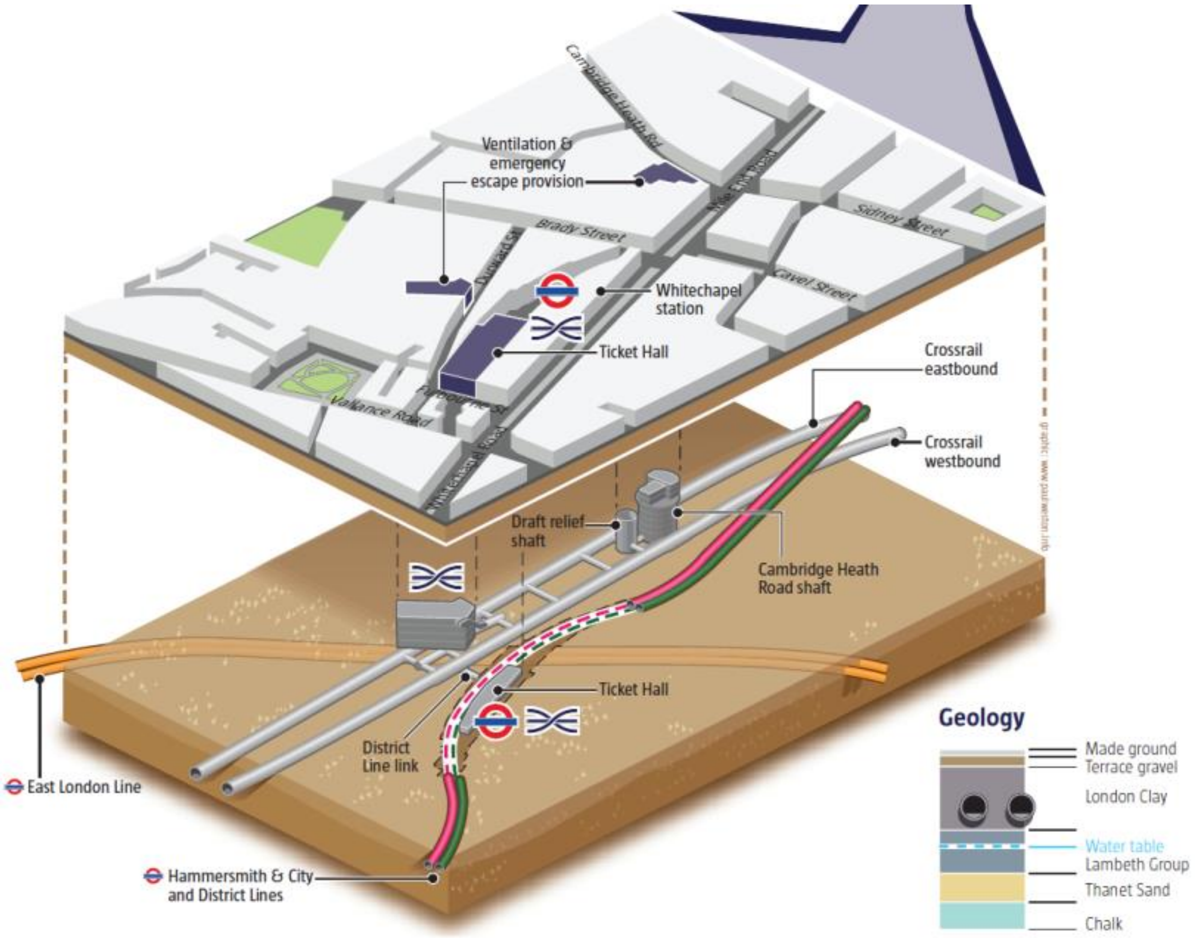


Figure 6.1.1: The Whitechapel site. (Source: New Civil Engineer (2009)).

6.2 – Tunnel Details

Although a 10.05m diameter cross passage tunnel is also considered briefly, this study focusses on the main eastbound and westbound tunnels. These two tunnels are being constructed using the pilot and enlargement tunnelling methodology (Figure 6.2.1). The initial pilot tunnel has a V_T of approximately 30.2m^3/m (i.e. an equivalent diameter (for a circular cross-section) of 6.20m). The enlargement phase sees this increase to give a V_T of 89.5m^3/m (i.e. an equivalent diameter of 10.68m). The tunnels are being excavated through LC at an axis depth of 23.5m.

Figure 6.2.1: Example of Crossrail pilot and enlargement SCL tunnelling.

6.3 – Tunnelling Programme

For the purposes of this study, in order to provide baseline dates for settlement measurements, the tunnel construction progress has been divided into four main stages as illustrated in Figure 6.3.1.

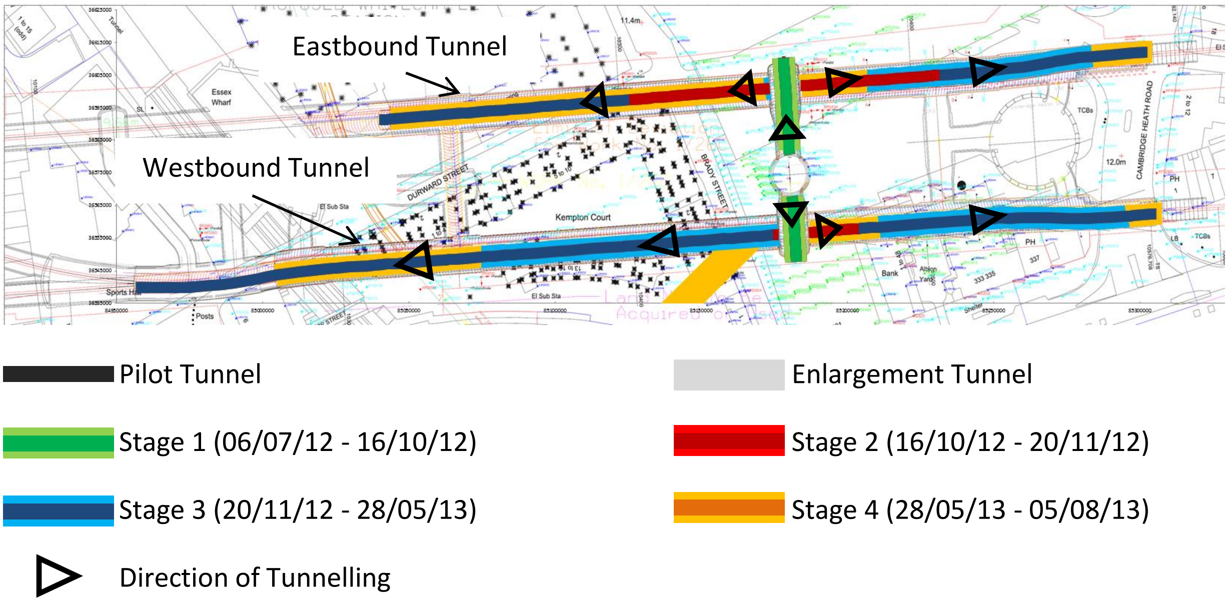


Figure 6.3.1: Whitechapel construction programme - plan view.

Following the initial shaft excavation, the first tunnelling stage identified (stage 1) is the construction of the cross-passage tunnels that link the eastbound and westbound tunnels.

Stage 2 is of primary interest as it represents the excavation of the eastbound pilot tunnel (in both an easterly and westerly direction), with minimal additional concurrent construction. This therefore ensures that the ground movement measurements are solely representative of the construction of this individual tunnel.

Significant excavation occurred during stage 3: The eastbound pilot tunnel was expanded in both directions and the majority of the westbound tunnel (both pilot and enlargement) was constructed, again in both directions. It is considered that, at a specific location, ground movement data for this stage will be less reliable due to the influence of concurrent tunnelling nearby.

Stage 4 primarily represents the enlargement of the eastbound tunnel, although it includes an element of enlargement towards the west of the westbound tunnel. No ground movement data beyond the end of this stage was available at the time of this study, although construction has continued. It is possible that a future study might examine the subsequent tunnelling and the development of ground movements with time.

6.4 – Ground Conditions

The tunnels at the Whitechapel site are being constructed in LC. Whilst the project is ongoing, the detailed site investigation data remains highly confidential, and it is therefore not possible to reproduce extensive data in this report. However, sufficient analysis can be undertaken by assuming several key parameters and studying representative soil profiles. The ground conditions quoted here therefore originate from internal sources, but can be verified as reasonable estimates by comparison with the other case studies.

Using borehole data, the representative soil profile for the Whitechapel site has been assumed to comprise a 2.5m thick layer of Made Ground (MG) and a 5.5m thick layer of River Terrace Deposits (RTD), underlain by a 29.8m thick layer of London Clay (LC). The ground below the LC is not important in this study. Figure 6.4.1 illustrates this representative soil profile alongside the soil profiles given in the literature for the case studies considered in Section 4. It is crucial to observe that the representative Whitechapel soil profile is very similar to those of the other case studies, with the tunnelling occurring in the LC layer. This enables direct comparisons between the projects.

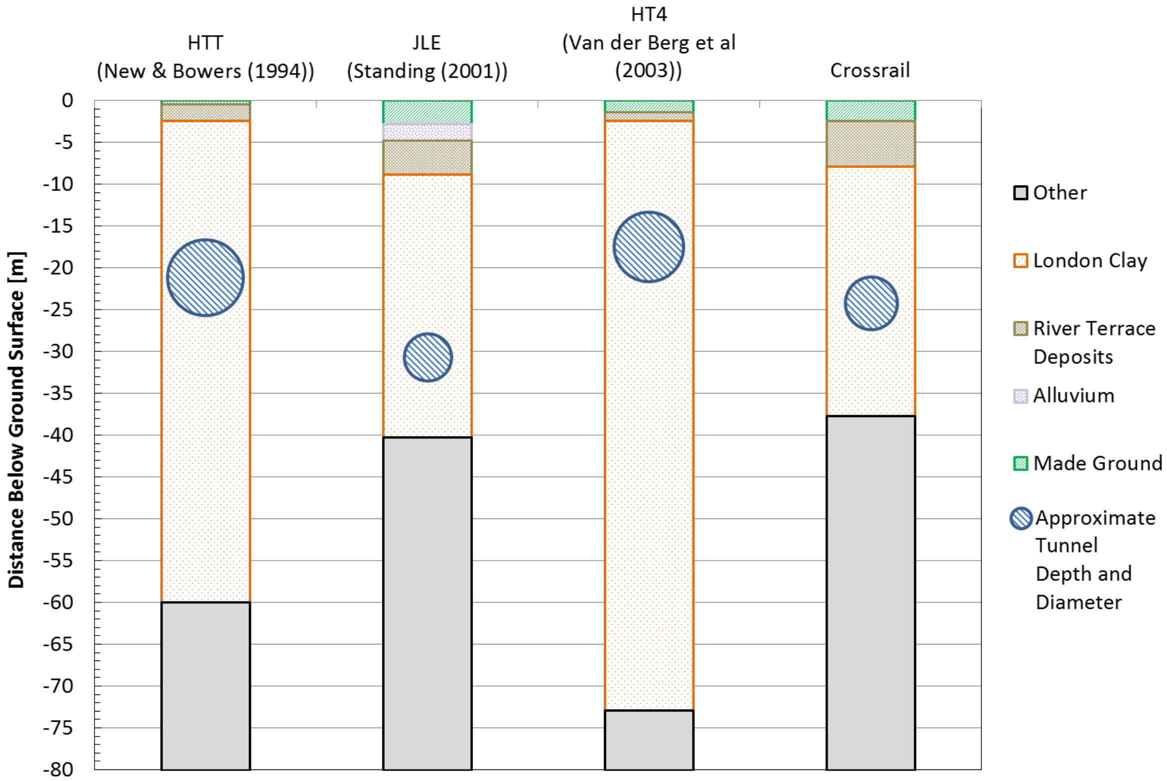


Figure 6.4.1: Representative soil profiles for Crossrail and other SCL case studies.

The unit weights of the MG, RTD, and LC have been assumed to be 19KNm^{-3}, 19KNm^{-3}, and 20KNm^{-3} respectively. The undrained shear strengths (s_u) of both the MG and RTD have been assumed to be negligible. The s_u of the LC has been taken as 30KPa at the top of the layer, increasingly linearly with depth at a rate of 7.5KPam^{-1}. During construction, the water table is held at a depth sufficiently greater than the bottom of the tunnel to ensure a safe construction. The variations with depth of vertical stress (σ_v) and s_u are given in Figure 6.4.2.

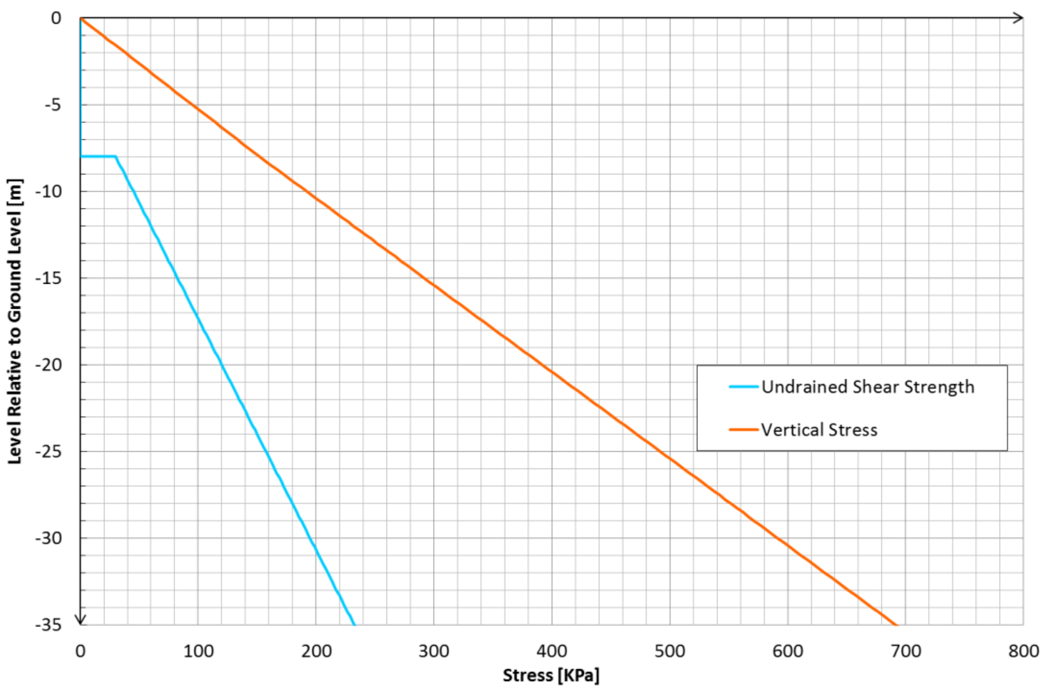


Figure 6.4.2: Representative vertical stress and undrained shear strength profiles.

The value of N has been computed using Equation 3.3.2, assuming zero surcharge pressure (σ_s). Due to the assumption of zero undrained shear strength in the MG and RTD layers, N is theoretically infinite in these layers; however it is the LC layer that is of primary concern. The variation of N with depth for the LC layer is presented in Figure 6.4.3.

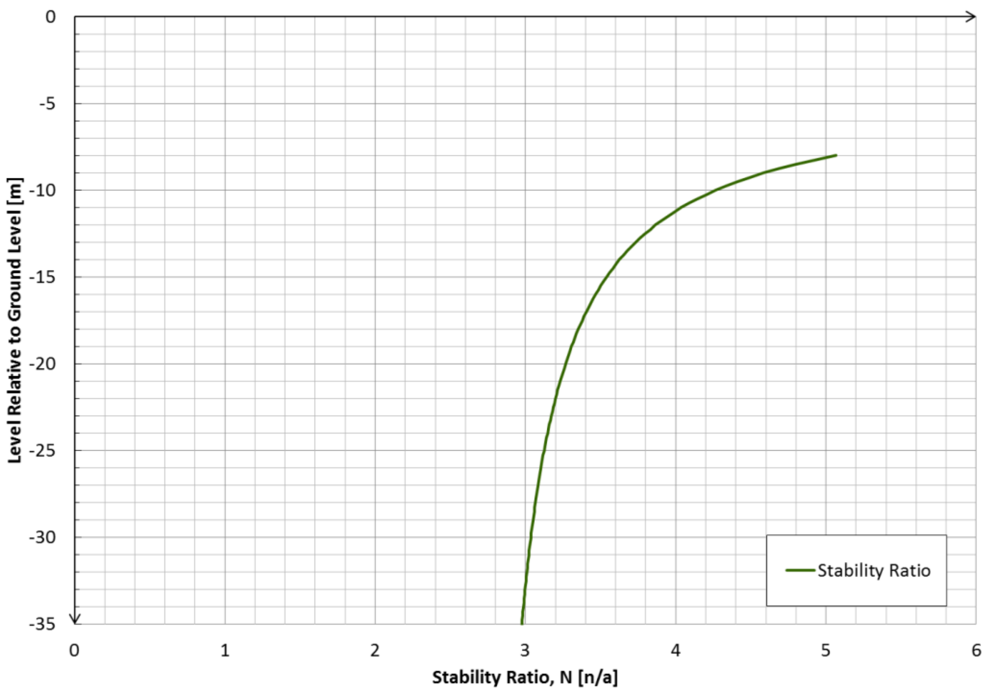


Figure 6.4.3: Variation of Stability Ratio with depth for the LC layer.

6.5 – Instrumentation and Monitoring

A substantial level of instrumentation and monitoring was specified for the Whitechapel site following an analysis of the potential damage to the surrounding structures (see Section 5.3). Consequently, a large quantity of ground movement data is available. This study is particularly interested in the ground surface settlement data which has been obtained using levelling points (LPs) with readings taken on a daily basis. LPs have been combined to form a series of transects across the tunnels, four of which are addressed in this study as they were deemed to be the most significant and informative (Figure 6.5.1). Although the LPs do not always align exactly, the impact of any misalignment is considered negligible.

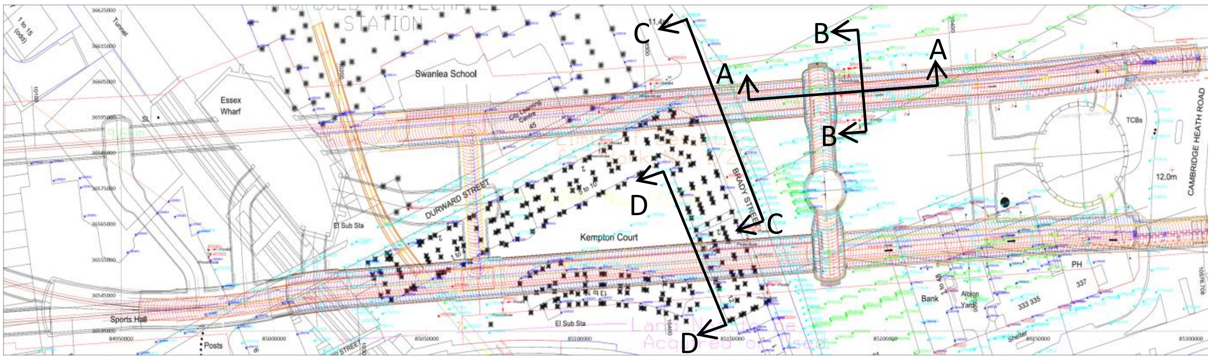


Figure 6.5.1: LP section transects.

Monitoring began prior to any construction, and LP data is available from 10 February 2012. Monitoring will continue until the measured ground movements are considered negligible; however no LP data was available to this study beyond 5 August 2013.

7 – PRESENTATION OF CROSSRAIL DATA

7.1 – Collation and Extraction of Relevant Data

Section 6.5 highlighted the four LP transects that have been considered in this study. The following section (Section 7.2) presents the ground surface settlement data for each transect and stage of construction. In order to arrive at the settlement profiles, several assumptions have been made and the LP data has been processed to ensure accurate and informative results.

The LP transects are not perpendicular to the direction of the tunnelling; particularly transects C and D which form an intersection angle of approximately 16 degrees. The effect on the shape and size of the settlement profiles has been considered negligible. Nevertheless, the V_L results obtained from the analysis must be modified accordingly (see Section 8.2).

In Section 7.2, the magnitudes of the ground surface settlements have been plotted along each transect section. Although the raw data is presented initially, the focus of the subsequent analysis is on the net settlement profiles, where ground movements have been found for each construction stage by simply subtracting the LP readings at the start of the construction stage from those recorded at the end of the stage.

Where possible, best-fit Gaussian curves have been added to the net settlement plots. These curves have been fitted using an empirical approach: Initially, estimates were made for V_L and K, from which i and S_{max} were calculated. The corresponding Gaussian curve was then plotted alongside the raw data and its accuracy assessed. By adjusting the K and V_L values iteratively, and re-plotting the Gaussian curves, the most accurate curve fit was established.

7.2 – Ground Surface Settlement Profiles

Transect A, unlike the other transects, has been identified to examine the ground movements caused by the construction of the cross passage, rather than the platform tunnels. The total settlement for transect A after each of the first three construction stages (see Section 6.3) is illustrated in Figure 7.2.1. Stage 4 data was not available for transect A.

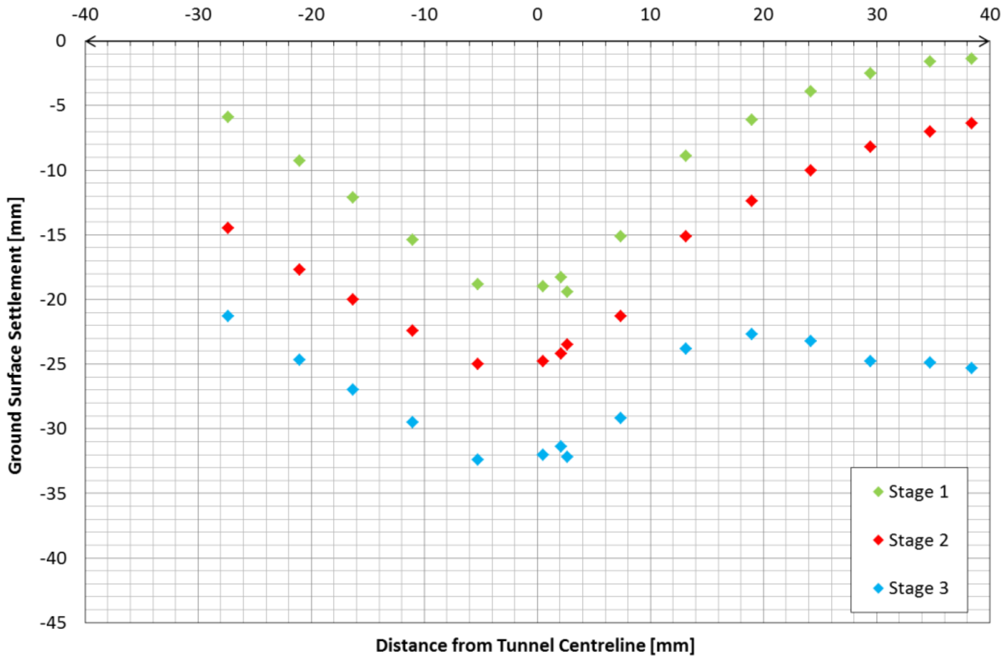


Figure 7.2.1: Total settlements after each construction stage for transect A.

Figure 7.2.1 shows that the total short term settlement after the three construction stages is approximately 33mm. The net settlement for each stage is presented in Figure 7.2.2.

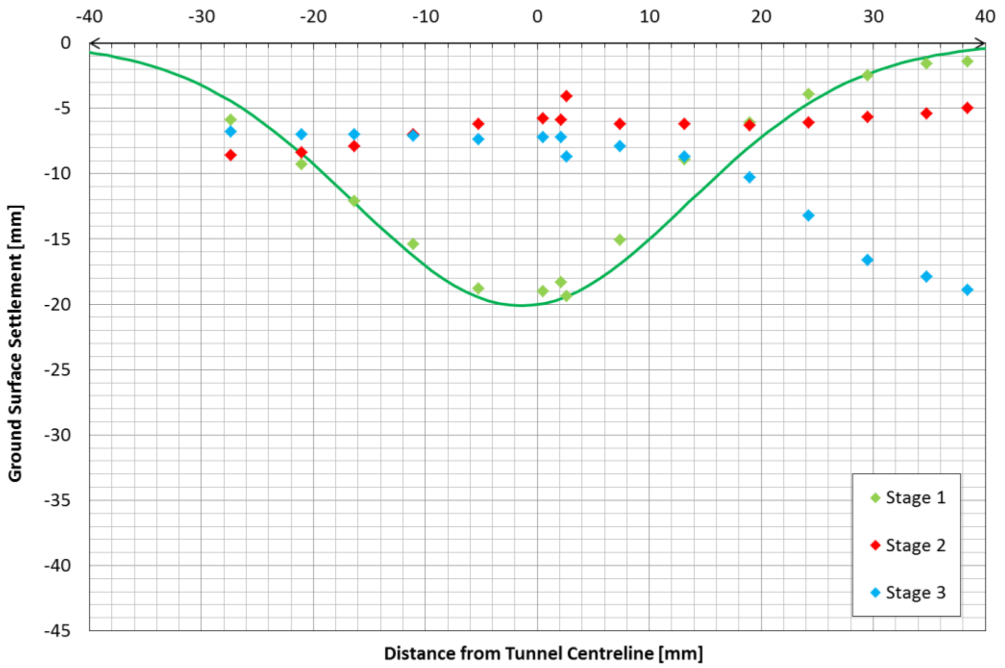


Figure 7.2.2: Net settlement due to each construction stage for Transect A.

For transect A, stage 1 provides the most interesting phase of construction as it corresponds to the construction of the 10.05m diameter cross passage. The settlement observed during stages 2 and 3 is most likely due to the construction of the eastbound platform tunnel as well as the continued settlement from the initial cross passage construction. A Gaussian curve has been fitted to the stage 1 settlement presented in Figure 7.2.2. This curve corresponds to a K of 0.65 and a V_L of 0.95%. The curve appears to provide a reasonable fit to the data.

Transect B intersects the eastern section of the eastbound tunnel approximately perpendicularly. The progression of total settlement over the 4 construction stages is shown in Figure 7.2.3.

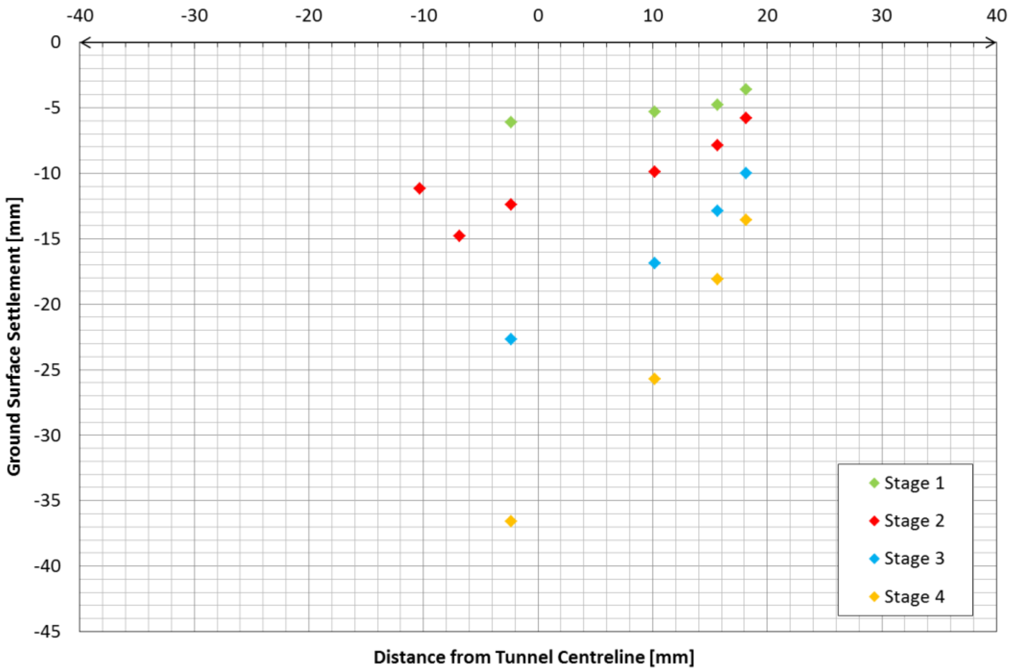


Figure 7.2.3: Total settlements after each construction stage for transect B.

Figure 7.2.3 highlights the relative scarcity of data available for transect B, caused by the desire to identify a perpendicular transect. Nevertheless, the net short term settlement caused by each construction stage is presented in Figure 7.2.4.

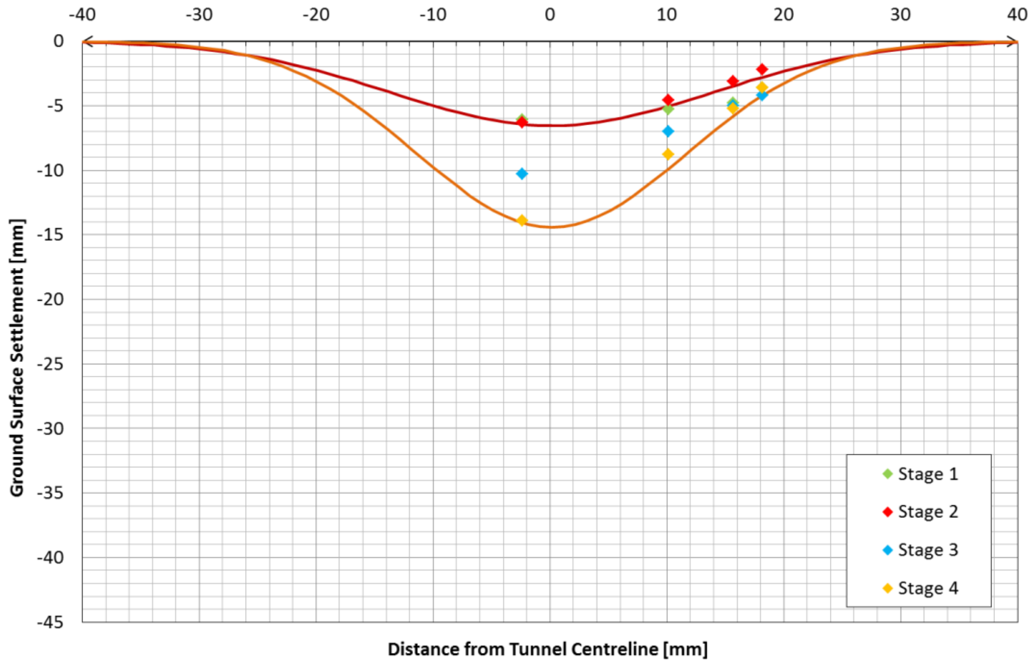


Figure 7.2.4: Net settlement due to each construction stage for Transect B.

Two Gaussian settlement curve fits have been added to Figure 7.2.4, corresponding to construction stages 2 (pilot tunnelling) and 4 (enlargement tunnelling). Although the curves show a reasonable fit with the data, a degree of caution is necessary given that the data is limited and, particularly for the enlargement tunnelling, the constructed tunnel section is relatively short. The curves for stages 2 and 4 were produced using K values of 0.60 and 0.50 respectively, and V_L values of 0.75% and 0.70% respectively.

Transect C provides substantially more data points than transect B, as is illustrated in Figure 7.2.5.

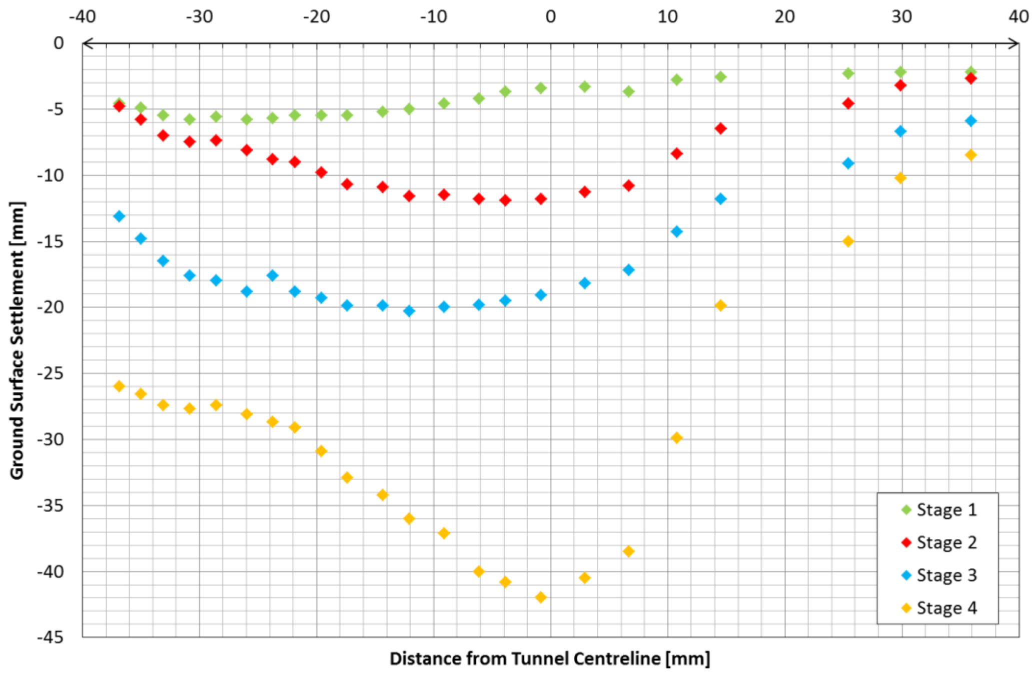


Figure 7.2.5: Total settlements after each construction stage for transect C.

Figure 7.2.5 shows a total settlement of about 42mm, the largest total settlement recorded for any transect. The figure also reveals a significant increase in settlement for each construction stage. Figure 7.2.6 identifies the net settlement caused by each construction stage.

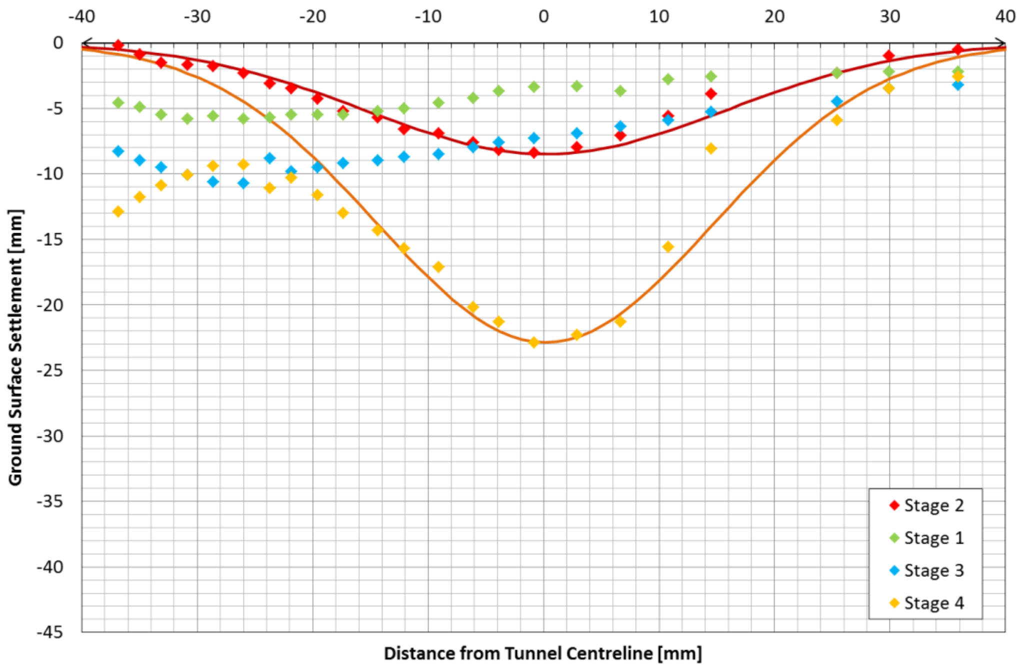


Figure 7.2.6: Net settlement due to each construction stage for Transect C.

Again, two Gaussian curve fits have been added for Figure 7.2.6. These match the settlements caused by the pilot tunnel construction (stage 2) and enlargement tunnel construction (stage 4). For the stage 2 curve, a reasonably accurate fit has been achieved using a K value of 0.68 and a V_L of 1.10%. The stage 4 curve corresponds to a significantly higher V_L of 1.40%, and a K value of 0.63. The curve has been fitted primarily by considering the right hand portion of the curve as it is possible that left hand section could have been influenced by the presence of the westbound tunnel, which had been constructed by stage 4. This is evident from the fact that the data still shows significant stage 4 settlement in the left hand trough wing.

Transect D examines the westbound platform tunnel. The total settlement after each construction stage is plotted in Figure 7.2.7.

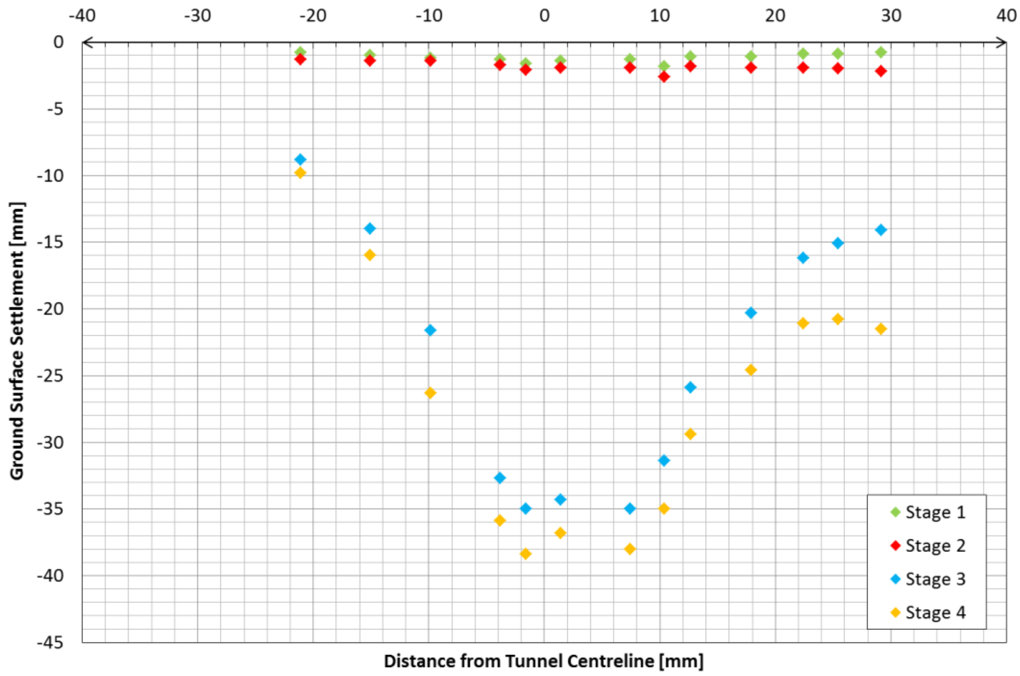


Figure 7.2.7: Total settlements after each construction stage for transect D.

It is evident from Figure 7.2.7 that very little settlement occurred during stages 1, 2 and 4. However, as expected due to the construction of both the pilot and enlargement tunnels, significant settlement was recorded during stage 3. Figure 7.2.8 breaks down the net settlement for each stage.

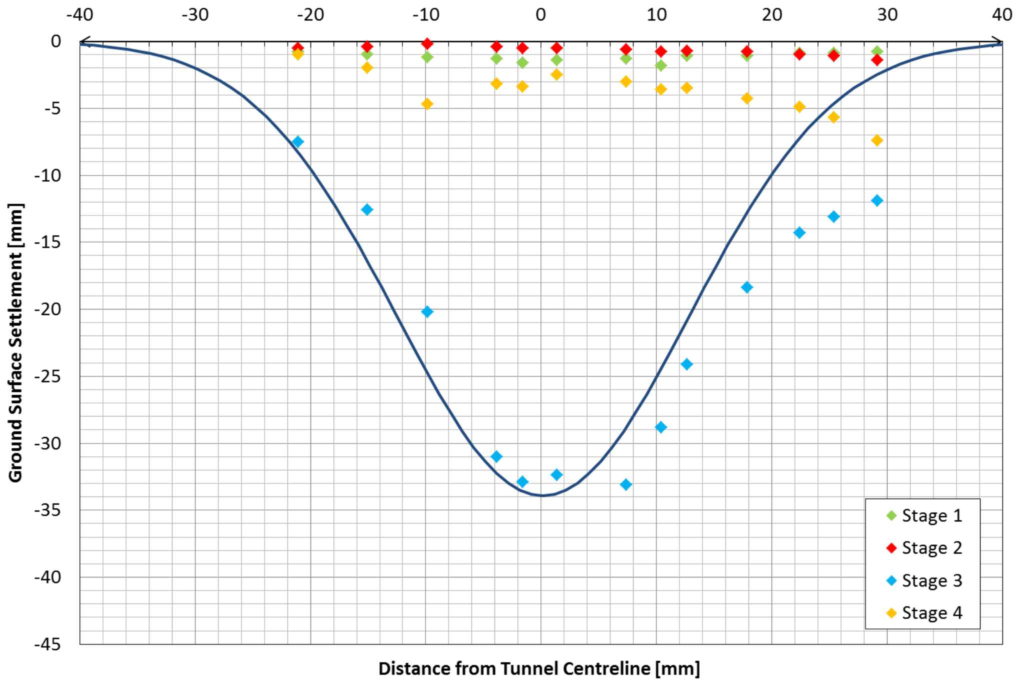


Figure 7.2.8: Net settlement due to each construction stage for Transect D.

The Gaussian curve added to Figure 7.2.8 corresponds to a K value of 0.50 and a V_L of 1.20%. On this occasion, the curve was fitted to the data in the left portion of the graph due to the potential of the eastbound tunnel to influence the settlement recorded in the right portion of the chart. The curve fit is not as accurate as those observed for the other transects.

Table 7.2.1 summarises the values used to generate the Gaussian curve fits identified in this section, for the various transects and tunnel construction stages. Table 7.2.1 also notes the construction affecting each set of readings, and the corresponding equivalent tunnel diameters.

TABLE 7.2.1: SUMMARY OF WHITECHAPEL GAUSSIAN CURVE FIT DATA

Transect	Construction Stage	Relevant Construction Completed	Equivalent Tunnel Diameter [m]	Maximum Settlement, S_{max} [mm]	Trough Width Parameter, K	Volume Loss, V_L [%]
A	1	Full cross passage	10.05	19.4	0.65	0.95
	2	n/a	n/a	8.6	n/a	n/a
	3	n/a	n/a	18.9	n/a	n/a
	4	n/a	n/a	0.2	n/a	n/a
B	1	n/a	n/a	6.1	n/a	n/a
	2	Pilot tunnel	6.2	6.3	0.60	0.75
	3	n/a	n/a	10.3	n/a	n/a
	4	Enlargement tunnel	8.69	13.9	0.50	0.70
C	1	n/a	n/a	5.8	n/a	n/a
	2	Pilot tunnel	6.2	8.4	0.68	1.10
	3	n/a	n/a	10.7	n/a	n/a
	4	Enlargement tunnel	8.69	22.9	0.63	1.40
D	1	n/a	n/a	1.8	n/a	n/a
	2	n/a	n/a	1.4	n/a	n/a
	3	Pilot & Enlargement tunnel	10.68	33.1	0.50	1.20
	4	n/a	n/a	7.4	n/a	n/a

8 – DATA ACCURACY AND INTEGRITY

8.1 – Accuracy and Precision of Raw Data

It is difficult to quantitatively assess the accuracy of the results because of the nature of the analysis. The precision of the LPs has been observed to be approximately ±0.5mm. This value has been arrived at by considering data on consecutive days and observing the fluctuation between the points. This precision is a reasonable estimate, with various other case studies reporting similar accuracies (e.g. Van der Berg et al (2003)). For the maximum settlement of the eastbound pilot tunnel (8.4mm), this would be an uncertainty of 5.9%.

8.2 – Interpretation, Assumptions, Corrections, and Reliability

The selection of the most representative settlement profile, being least affected by external factors, is justified in Section 9.1. This helps to eliminate inaccuracies in the settlement data arising from the effects of other concurrent excavations or mitigation measures. Even so, interpreting the raw data inevitably leads to additional sources of uncertainty. In particular, manually fitting a Gaussian curve to the settlement data requires an element of judgement. By using slightly different K values it is possible to fit a range of V_L values to any given settlement plot. In the extreme cases this could lead to an uncertainty of about ±0.025% (e.g. for a V_L of 1.1%, a potential error of 2.3%).

When the above errors are combined, a potential error estimate of 7.2% is obtained, which is relatively high. However, it is reasonable to expect some errors to counteract, particularly the errors in the LP data. This is because the curve fit aims to match as many points as possible and will therefore inevitably overestimate some points and underestimate others. With so many data points it is unlikely that the errors will accumulate, but the theoretical possibility should not be discounted.

One potential source of inaccuracy can be corrected for. In several instances the LP transects intersect the tunnels at an angle, yet the relevant theory is derived from settlements in a plane perpendicular to the tunnel's direction. As part of this study, a theoretical assessment of this inaccuracy has been undertaken. By considering the cross-sectional area of the tunnel intersected by the LP transect at an angle θ, it can be shown that the measured V_L is an overestimate and must be reduced by a factor of cos(θ). Therefore, for a typical transect angle of 16°, a discrepancy of about 3.9% is predicted.

9 – ANALYSIS AND DISCUSSION

9.1 – Analysis of Crossrail Settlement Profile Transects

The Crossrail data was presented in Section 7 for four LP transects. The general shapes of the net settlement profiles were largely as expected, aligning closely with Gaussian curve fits. Transect A suggests a V_L of 0.95% for the cross passage tunnel. This is a reasonable estimate, but must be treated with caution. The cross passage tunnel is relatively short and has a varying cross section. Theoretically, the settlement recorded when the excavation is directly below a particular point should be approximately 50% of the total short term settlement, with the remaining half occurring as the excavation continues past the transect (Attewell & Woodman (1982)). As transect A is relatively close to the end of the cross passage, it is possible that this may result in lower settlements being recorded in comparison to if the excavation had continued further. The stage 1 settlement profile used for transect A did not account for the tunnelling approach adopted; it assumed a tunnel excavation of constant circular section as opposed to the enlargement of an initial tunnel to form a non-uniform cross-section. This might have affected the settlements recorded. Nevertheless, transect A is a sufficient distance from any grouting activity and, during stage 1, sufficiently far from other excavations and groundwork. The potential effects of these can thus be assumed negligible. Therefore, overall, the stage 1 settlement curve gives a reasonable model for the settlement caused by a typical cross passage tunnel, despite not necessarily being widely representative of the Whitechapel SCL tunnelling as a whole. Although stage 4 settlement data was unavailable for transect A, it is anticipated that a similar rate of increase in settlement would have continued to have been observed. The excavation of the eastbound pilot tunnel during stage 2 appeared to result in significant ground settlement at transect A, so it is reasonable to assume that similar results would be associated with the enlargement of the eastbound tunnel during stage 4.

Fitting Gaussian curves to stages 2 and 4 for transect B gave V_L estimates of 0.75% and 0.70% for the pilot and enlargement tunnels respectively. Transect B provides the advantage that the LPs are located perpendicular to the direction of the tunnelling, and so the V_L estimates can be considered to be realistic values. Despite this, the V_L values recorded are unexpectedly low (in comparison with the case history and other transects). This is possibly caused by a lack of sufficient data as only 4 data points have been used to fit the Gaussian curves in each case. Indeed, the additional total settlement data available for stage 2 (see Figure 7.2.3) indicates a larger total settlement to the left of the tunnel centreline than observed at the tunnel centreline and towards the right; however this value has not been included when plotting the net settlement as no baseline data was available. The fact that significantly larger total settlements are present away from the tunnel centreline confirms that the four points used to fit the Gaussian curve are insufficient to provide an accurate assessment of V_L. Furthermore, during stage 4, transect B is relatively close to the lateral limit of the enlargement tunnelling (as the majority of the enlargement of the eastbound tunnel towards the east was undertaken during stage 3). Therefore, like transect A, the relatively small length of tunnel affecting the settlement is likely to cause the recorded settlement to be less than that which would be recorded for a longer length tunnel.

Transect C has been used to investigate the western part of the eastbound tunnel. The net settlement for stage 2 has been shown to accurately fit a Gaussian settlement curve corresponding to a V_L of 1.10%. Unlike transect B, a substantial amount of data was available for transect C and thus the accuracy and reliability of the Gaussian curve can be validated at several points along the section. Although the curve fit sometimes differs slightly from the raw data, it is observed that the curve provides an underestimate of the settlement to the left of the centreline, and an overestimate to the right of the centreline. It has been assumed that these effects counteract each other, and so the overall V_L recorded remains unaffected. During stage 2, the only significant excavation work was the construction of the eastbound pilot tunnel; therefore the settlement recorded has not been influenced by any additional underground construction. A potential source of error is the continued (medium-term) settlement arising from the shaft and cross passage construction. Additional ground movements caused by these excavations would lead to increased settlement measurements and thus an overestimate of the V_L. Here, this effect has been considered negligible and conservatively ignored.

The stage 4 Gaussian curve fit for transect C suggests a volume loss of 1.40%, associated with the enlargement of the eastbound tunnel. Although the same amount of data is available for this stage, several other factors suggest that the Gaussian curve is a less accurate representation of the true settlement caused by the enlargement of the tunnel.

Firstly, as the pilot tunnel had already been constructed at this stage, it is likely that the ground would have continued to settle significantly, even if the enlargement had not taken place. This is not accounted for in obtaining the net settlement for stage 4. Furthermore, towards the left of the centreline it is observed that the settlement data remains significant (rather than diminishing to zero as would be expected for a Gaussian curve). This is likely to be due to the construction of the westbound tunnel during stage 3 and subsequent other excavations also occurring during stage 4. To account for this, the Gaussian curve was primarily matched to the data furthest from the westbound tunnel (i.e. to the right of the centreline), but this reduced the quantity of data available.

One additional transect considered initially was effectively an extension of transect C to include the data for the LPs above the westbound tunnel and close to the cross passage. However, it was noted that a limited amount of compensation grouting was present in this region which, although not considered to affect the data elsewhere, led to misleading settlement data including hogging above the westbound tunnel close to the cross passage. As such, this transect was not selected for additional consideration in this study and will not be examined further here.

Transect D was sufficiently far from the cross passage to allow the effects of the aforementioned compensation grouting to be neglected. In Section 7.2, one Gaussian curve fit was established for the stage 3 data, during which time both the westbound pilot and enlargement excavations took place. From the Gaussian curve fit, a V_L of 1.20% has been obtained. This appears to be a reasonable estimate, and indeed a significant amount of data was available at transect D to facilitate comparisons with the Gaussian curve fit. However, it can be seen from Figure 7.2.8 that the best fit Gaussian curve does not give an accurate representation of the recorded data. During stage 3, a section of the eastbound pilot tunnel was also constructed, and it is therefore almost certain that this would have affected the recorded settlement to the right of the centreline. Despite attempting to fit a Gaussian curve to the data in the left portion of the plot, it is inevitable that there is a degree of inaccuracy in the curve fit.

The above analysis of the Gaussian curve fits for each construction stage and LP transect indicates that the net settlement recorded at transect C during stage 2 of the construction is the most representative settlement plot available for the SCL tunnelling at the Whitechapel site. Whilst the other transects and construction stages provide interesting analysis, as well as indicating the true settlements experienced, they are each affected by a number of factors which limit their wider applicability. The Gaussian curve fitted at transect C to the stage 2 net settlement data indicates a V_L of 1.10%. However, a correction must be made to account for the fact that transect is not perpendicular to the direction of excavation. Applying the theory discussed in Section 8.2 reduces the value of the V_L by 3.9%. Thus, the representative V_L for the SCL tunnelling at the Whitechapel site can be quoted as 1.06%. Clearly this value is only a best estimate and, as outlined in Section 8.2, an error of up to 7.2% is possible, although in reality a smaller error is more likely. Nevertheless, this V_L can be compared with case history and used to populate theoretical models (Sections 9.2 and 9.3).

It is possible to verify the choice of construction stages established in Section 6.3 by analysing the progression of the recorded surface settlement with time. As transect C provides the most representative settlement profile, the construction stages have been validated using this transect. By considering the variation in settlement readings for the LP closest to the tunnel axis is possible to observe the progression of the maximum settlement with time (Figure 9.1.1).

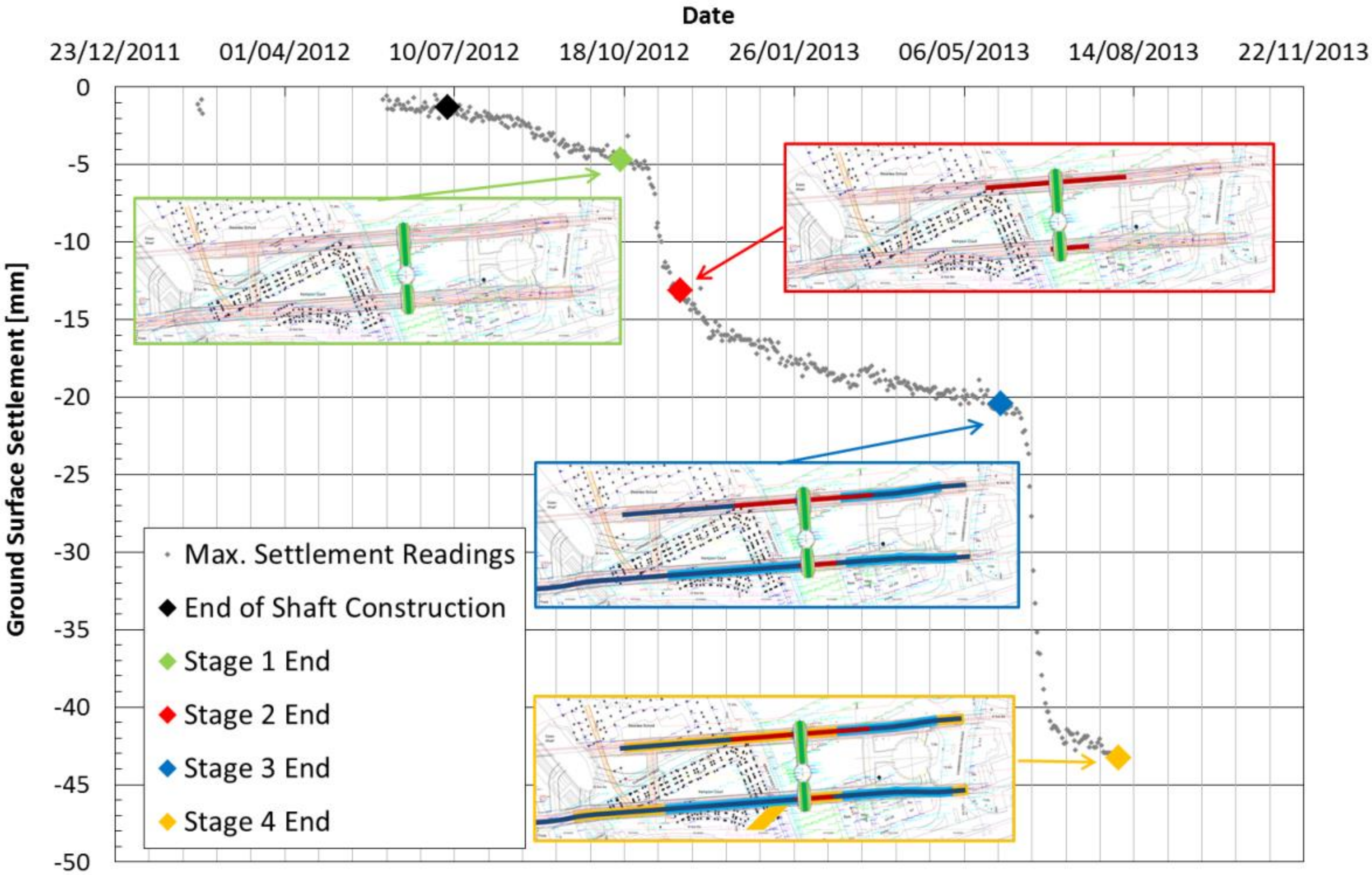


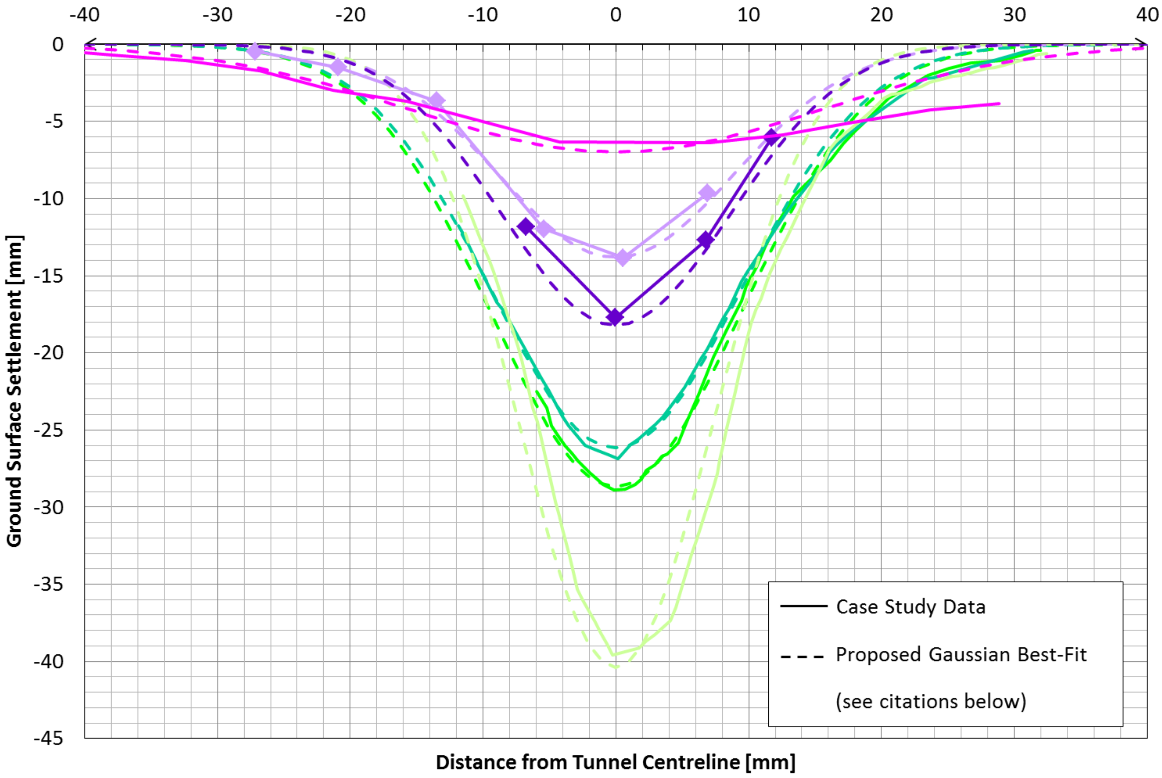
Figure 9.1.1: Progression of maximum settlement readings over time for transect C.

It can be seen from Figure 9.1.1 that a limited amount of ground settlement was experienced at transect C during the stage 1 excavation of the cross passage. The increase in settlement is relatively gradual, which is expected as the excavation is a short distance from the LP transect. Between the end of stage 1 and the end of stage 2 there is a sharp increase in the settlement recorded. This is caused by the construction of the eastbound pilot tunnel in stage 2, and corresponds to the stage 2 net settlement profile identified in Section 7.2. It is reassuring that the rate of increase in settlement drops significantly at the end of stage 2, and only a gradual increase in settlement is experienced between the end of stage 2 and the end of stage 3. This is likely due to the continued (medium-term) settlement caused by the stage 2 eastbound pilot tunnel construction, as well as limited settlement due to the westbound excavation. Figure 9.1.1 shows a significant increase in settlement between the end of stage 3 and the end of stage 4. This is expected, and corresponds primarily to the excavation of the eastbound enlargement. However, the gradual settlement increase between the end of stage 2 and the end of stage 3 suggests that an increase in settlement would have continued beyond the end of stage 3, irrespective of the additional tunnelling. It is thus difficult to quantify the particular settlements caused by this, excavation elsewhere (i.e. the westbound enlargement), and the excavation of the eastbound enlargement. To summarise, Figure 9.1.1 confirms that the start and end dates chosen for each construction stage are appropriate and can be used to give the best indication of the net settlement caused during each stage. The points identified as marking the end of one construction stage and the start of another align closely with the turning points on the graph, with sharp increases in maximum settlement corresponding closely with the stages of tunnel construction directly below the LP transect.

9.2 – Review of Case Studies alongside Crossrail Results

Three case studies of SCL tunnelling in London Clay were presented in Section 4; HTT, JLE, and HT4. Together with the results of the Crossrail Whitechapel site discussed in Section 9.1, these can be used to assess any developments in SCL tunnelling in LC and determine any trends. Moreover, focussing only on the data for SCL tunnelling in LC reduces the number of variables that may be affecting the results, thus allowing for more specific theoretical models to be developed (see Section 9.3 for the application of the data to the LF approach).

It is important to provide evidence to validate the quoted V_L values referenced for each case study. Therefore, the settlement profiles for each case study, as presented by their original sources, have been digitised and collated onto one graph, Figure 9.2.1.



HTT – Type 1	(New & Bowers (1994))	V_L = 1.15%, K = 0.45
HTT – Type 2	(New & Bowers (1994))	V_L = 1.05%, K = 0.45
HTT – Type 3	(New & Bowers (1994))	V_L = 1.26%, K = 0.35
JLE – Elizabeth House	(Standing (2001))	V_L = 1.10%, K = 0.5
HT4 – Platform	(Van der Berg (1999))	V_L = 0.50%, K = 0.5
HT4 – Concourse	(Van der Berg (1999))	V_L = 0.80%, K = 0.5

Figure 9.2.1: Validation of case study volume losses from settlement profiles.

Figure 9.2.1 includes a Gaussian curve fit for each settlement profile which has been calculated based on the quoted V_L values and other relevant information such as the tunnel properties and K values. It can be seen from Figure 9.2.1 that, as expected, the Gaussian curves fit accurately with the raw data.

Although it is difficult to compare the settlement curves and maximum settlements for different projects directly, there are a few important observations to note. For the HT4 platform and concourse tunnels, it can be seen that only very few data points were available. Therefore, although the Gaussian curve fits match the data very accurately, it could be argued that insufficient data was available to draw any firm conclusions. The settlement profiles for JLE and HTT do not show any individual data points. This is because they have been interpreted from curves given in the literature, as opposed to actual data points. A V_L of 0.50% was used for the Gaussian curve fit for the HT4 platform tunnel settlement shown; however in the literature additional settlement data was presented and the V_L was found to range between 0.50% and 0.60% (Van der Berg (1999)). For the HTT Type 3 tunnel, it can be seen that the Gaussian curve fit appears to underestimate the settlements in the trough wings. This was also observed by New & Bowers (1994), who proposed that a numerical analysis gave a more accurate V_L of 1.37% in comparison to the original estimate of 1.26%.

The V_L values recorded by the case studies have been plotted alongside the 1.06% representative V_L for Crossrail's Whitechapel site in Figure 9.2.2. The figure shows the historic development of V_L for SCL tunnelling in LC. In drawing comparisons with long-term settlement data, Bowers et al (1996) used a short-term V_L of 1.26%; however in this study the short-term V_L has been taken to be 1.37% for the reasons discussed above. The HT4 platform tunnel V_L has been taken as an average value of 0.55%.

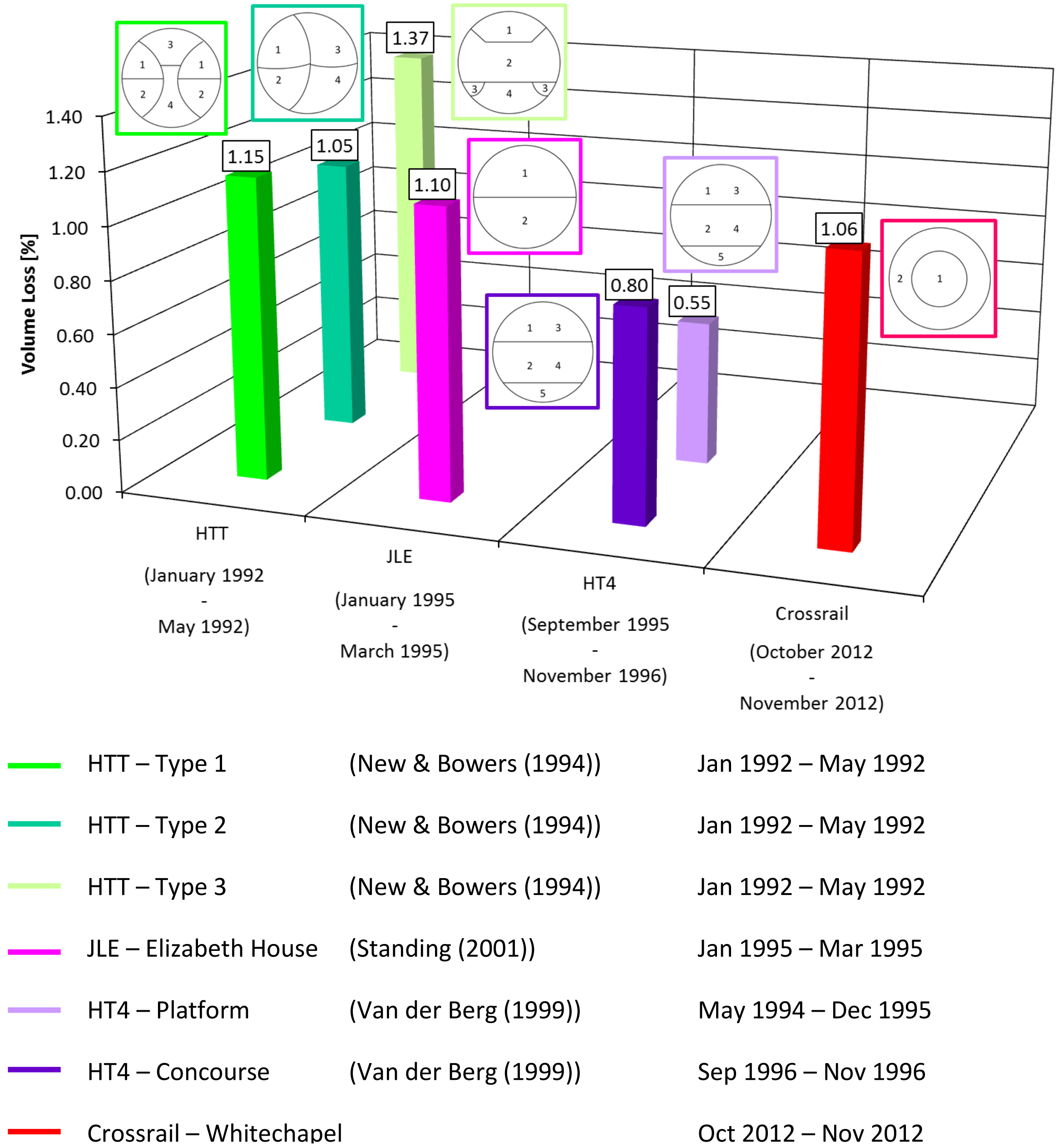


Figure 9.2.2: Historic development of volume losses for SCL tunnelling in London Clay.

The first important observation to note from Figure 9.2.2 is that all the quoted V_L values fall in the range of 0.5% to 1.5%. Furthermore, despite not necessarily being reliable (for the reasons discussed in Section 9.1), all the V_L values given in Table 7.2.1 also lie within the 0.5% to 1.5% range. This range was identified by Mair & Taylor (1997) as being typical for SCL tunnelling in LC. Indeed it is of further interest to note that Dimmock & Mair (2007a) concluded that 0.5% would be the minimum V_L achievable for open-face tunnelling in London Clay. This claim is supported by the evidence that no V_L values below 0.5% have been identified. Crossrail's use of a 1.5% V_L assumption appears conservative on the basis that no V_L values greater than this have been found.

It is reasonable to expect that the advancements in technology and tunnelling construction methods discussed in Section 2.1 would lead to better management of ground movements over time, and thus a reduction in the ground surface settlements experienced in later projects. It is therefore perhaps surprising that Figure 9.2.2 does not show any clear development of V_L measurements over time. A possible explanation for this is that the various sites cannot be directly compared in terms of V_L due to additional parameters affecting the settlement. The calculation of V_L does not adequately account for the tunnel properties, ground conditions, or method of excavation.

The V_L values recorded for HT4 are considerably smaller than those for the other case studies, including the Whitechapel site. One possible explanation is the use of grouting. This was considered at length by Van der Berg (1999), who noted that approximately 200,000 litres of grouting was used at the HT4 site. Van der Berg (1999) concluded that the ground surface settlements (and therefore V_L values) were not affected by the grouting, reasoning that no grouting was applied in the area around the tunnel heading. However, it is possible that the grouting injected ahead of the excavation strengthened the soil and thus limited the future ground movements as the tunnel heading passed through. It was noted in Section 9.1 that the representative Crossrail V_L was unaffected by grouting. It is also reported in the literature that grouting did not affect the V_L results for the JLE (Dimmock & Mair (2007a)) or HTT (New & Bowers (1994)). As such, the ground conditions at HT4 can be considered to be dissimilar to the Crossrail site and the other LC case studies. This is the most likely explanation for the small V_L values recorded.

9.3 – The Load Factor Graph

The LF approach was discussed in Section 3.3 and an adaptation of Macklin (1999)'s LF graph was given in Figure 3.3.2. It is interesting to populate this graph with the representative V_L data for the Crossrail Whitechapel site, and indeed the other case studies.

For the Whitechapel site, the V_L was based on the settlement profile of the 6.20m diameter eastbound pilot tunnel, which was constructed at an axis depth of 23.5m. Therefore, from Figure 6.4.3, the stability ratio is 3.16. The cover (i.e. depth of London Clay above the tunnel crown) is 12.4m, and so the C/D ratio is 2.0. Approximately 1m excavations were made during the eastbound pilot construction, and so P can be taken as 1m. This gives a P/D ratio of 0.16. Hence, from Figure 3.3.1, N_c is estimated to be 7.8 and thus, from Equation 3.3.1, the load factor for the Whitechapel site is 0.41.

Macklin (1999) has already included the data presented by New & Bowers (1994) for the HTT on the LF graph. In doing so, he assumed a P/D ratio of 0.1 and estimated the undrained shear strength at the tunnel axis from the work of Powell et al (1997).

Dimmock & Mair (2007b) examined the LF approach for the JLE SCL tunnelling at Elizabeth House, concluding a LF of 0.33. They compared a V_L of 0.8%, occurring prior to the shotcrete lining, with Macklin (1999)'s LF approach. However, in this study the overall V_L for the tunnel of 1.1% has been used to draw comparisons and populate the LF graph.

For the HT4 concourse and platform tunnels, no LF analysis has been found in the literature. However, an approximation for the LF can be found by assuming the same soil properties described in Section 6.4 for the Whitechapel site, and adapting these to the geometry of the two tunnels. For the 8.88m concourse tunnel, the cover is 10.7m and the advance length is 1m (Van der Berg et al (2003)). This gives P/D and C/D ratios of 0.11 and 1.20 respectively. The undrained shear strength and stress at the tunnel axis have been approximately calculated as 140KPa and 341.5KPa respectively. These results lead to an N_C of 6, an N of 2.43, and therefore a LF of 0.41. Similar calculations for the 7.9m platform tunnel yield a LF of 0.39. It must be stressed that these values are only approximations, and in practice it is possible that grouting strengthened the soil as discussed in Section 9.2.

Figure 9.3.1 is a copy of Macklin (1999)'s LF graph which has been populated with the results of the Crossrail study and the other SCL London Clay case study results (note that the results for the HTT were already included in Macklin (1999)'s figure).

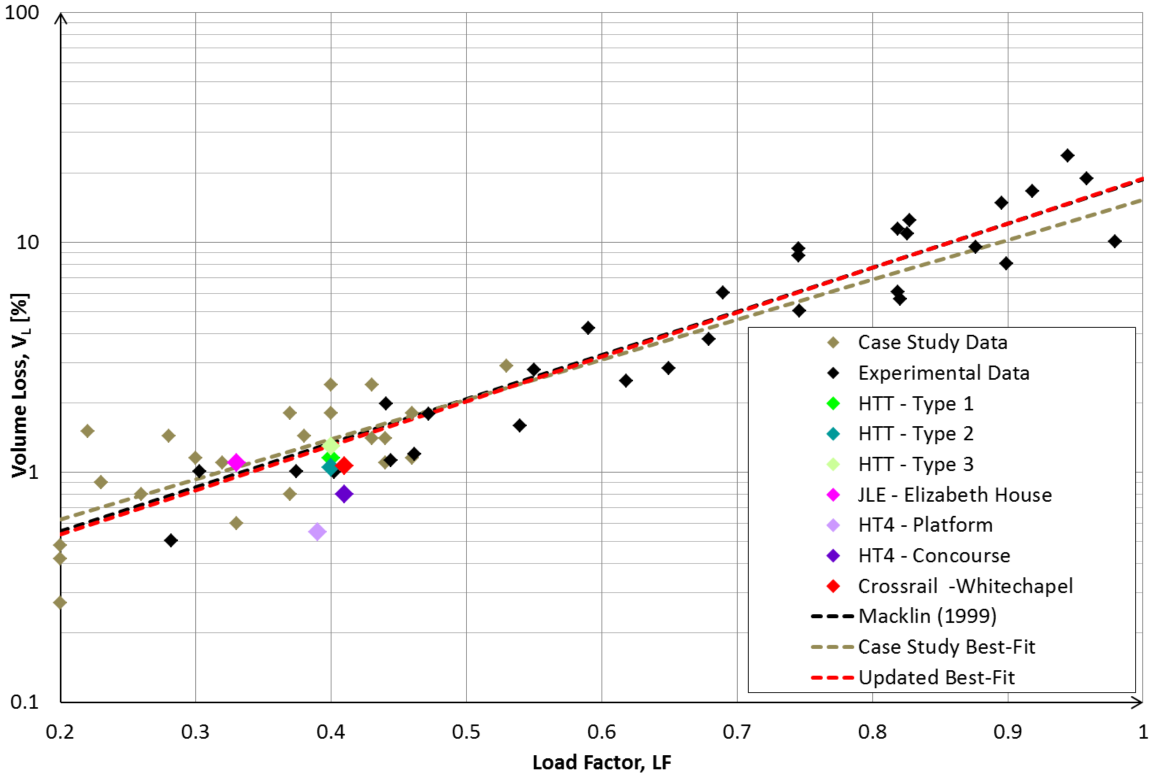


Figure 9.3.1: Population of LF graph with SCL LC data. (Adapted from: Macklin (1999))

On first inspection of Figure 9.3.1, the Whitechapel V_L appears to correlate closely with the other data, despite falling below the two best fit lines. However, for a LF of 0.41, Equation 3.3.1 predicts a V_L of 1.40, whilst Equation 3.3.2 predicts a V_L of 1.45. In comparison to the 1.06% V_L actually observed, these are relatively significant errors of 32.1% and 36.5% respectively. Such large overestimates of V_L predicted by Equations 3.3.3 and 3.3.4 raise concern over the wider application of these best fit lines. The LF approach appears to give a reasonable estimate of the broad range in which the V_L might lie, but it is difficult to obtain an accurate result due to the logarithmic nature of the relationship. The LF approach is only validated by the experimental data which allows higher LFs to be reached.

The Whitechapel data has given very similar results in comparison with the HTT and JLE data. The LF in all three cases is similar, and the V_L values are also close together. In the case of HT4, the LF estimate is also similar to the other SCL London Clay studies, despite having significantly lower V_L values. A third trend line has been added to Figure 9.3.1, to account for the additional case study data (Equation 9.3.1).

$$V_L = 0.22exp(4.45LF) \tag{9.3.1}$$

This trend line is very similar to that originally proposed by Macklin (1999) and could be used to generate V_L estimates when the LF is known. However, it must be stressed that the variation in data illustrated in Figure 9.3.1 mandates that any V_L estimate must not be treated as absolute. Instead, it should be used to indicate the order of magnitude.

9.4 – Implications in Tunnel Design

One of the aims of the research is to enable and support future savings. The Crossrail design approach was discussed in Section 5.3, with the emphasis clearly shown to be on the effect of ground movements on the surrounding structures. It was noted that, at the design stage, buildings are assigned categories based on the likely damage they will experience (Table 5.3.1). Levels of the mitigation measures and monitoring employed are based on these risk categories. The key factor in assigning a building to a particular category is the expected ε_{max}. It can be shown that ε_{max} is proportional to the ground surface settlement, and therefore also to the V_L. Consequently, it is possible to make significant cost savings if a smaller V_L can be assumed at the design stage. The 1.5% V_L assumed by Crossrail for the Whitechapel site (see Section 5.3) is significantly greater than the 1.06% actually observed. If a V_L of 1.06% had been known from the onset, it is possible that less monitoring and mitigation would have been needed and thus the overall cost would have been reduced.

There is a clear need to be able to obtain an accurate estimate for the V_L prior to construction whilst also ensuring that this estimate is sufficiently conservative to account for all potential ground movements and building deformations. This challenge is made substantially more difficult by the fact that all tunnelling projects differ. Not only are tunnelling methods different (e.g. SCL and TBMs), but the actual method of each excavation is also usually different. Furthermore, the tunnel dimensions, tunnel axis depth, and soil conditions are usually different between projects. However, even if all these attributes could be adequately accounted for, several further variables exist; the presence of other excavations, the use of grouting, and the quality of work.

At present, the LF approach provides arguably the best method for estimating the V_L as it attempts to account for the majority of the above factors. However, it was observed in Section 9.3 that the results of a LF analysis were only approximate. Nevertheless, the LF approach remains an important tool for assessing the V_L and as more data becomes available it can be verified and refined so that it can be used more widely. On the basis of this study, it is recommended that Equation 9.3.1 is used initially to find an approximation for the V_L. For the eastbound pilot tunnel at the Whitechapel site, this would have yielded a V_L of 1.36%. It is then crucial to examine a copy of Figure 9.3.1 and identify any case studies where similar methods of excavation and ground conditions were present. For the Whitechapel site, the JLE and HTT case studies would have been highlighted (HT4 would likely be ignored due to its low V_L values). Both the JLE and HTT projects suggest that 1.36% is a reasonable V_L estimate for similar projects. At this point, engineering judgment must be exercised to confirm a conservative value. The 1.5% adopted by Crossrail (see Section 5.3) appears to be reasonable given the need to exercise caution.

For financial savings to be realised, a reduction in the assumed V_L must be sufficient for a building to be re-classed into a lower risk category. For Crossrail, the key distinction is between categories 2 and 3, due to the need for further investigation and monitoring for buildings in categories 3 and above. Assuming that a building originally classed as category 3 has a mid-range ε_{max} estimate of 0.225%, this would need to be reduced by a third in order for the building to be re-classed. For Crossrail, this would correspond to a V_L of 1.0%. This highlights that minor changes in V_L estimates are unlikely to be significant in terms of generating large financial savings, and thus current estimates are probably efficient. Despite this, it is recommended that greater consideration is given to the V_L estimate at the design stage for buildings that are on the borderline between two risk categories (particularly where the associated cost discrepancy is significant). Indeed, if it had been possible to accurately predict the 1.06% V_L for Crossrail, this might have been sufficient to enable less monitoring to be installed in some cases. There will inevitably always be a need for a compromise between financial risk and the costs of monitoring and mitigation methods.

There is a need for a continued review of the LF approach, with the emphasis on obtaining new and repeatable data for a variety of projects. It might then be possible to draw more accurate relationships to account for specific excavation techniques. Statistical analysis of similar projects might provide further understanding. Ultimately however, every project is unique and a purely empirical relationship will only ever provide a best estimate.

10 – ADDITIONAL POINTS FOR CONSIDERATION

10.1 – Sustainability and Environmental Considerations

The environmental impacts and sustainability of any construction project plays a key role throughout the design, implementation, and post-construction phases. An Environmental Impact Assessment (EIA) was undertaken for the works at the Whitechapel site. Crossrail's Whitechapel Environmental Statement (see Crossrail (2011b)) reviewed the EIA conducted and presented an environmental statement focussing on several potential impact areas; townscape and built heritage, visual amenity, archaeology, ecology, water resources, traffic and transport, noise and vibration, air quality, contaminated land, and the community. The report concluded that the most significant impacts would be during the construction phase (i.e. not long-term) and would include visual and noise impacts from construction activities and increased transport, leading to the re-housing of 15 properties. Although these sustainability considerations do not directly relate to the findings of this study, they provide useful context to the scheme as a whole and highlight the scale of the project.

10.2 – Health and Safety

Tunnelling inevitably imposes significant health and safety risks. In proposing a six-factor safety management system for future projects, HSE (2006) drew on the experiences of 25 SCL emergency tunnel events occurring in soft ground, including HSE (1996)'s report on the tunnel collapse at Heathrow in October 1994. This history highlights the dangers of SCL tunnelling. As part of this study, a risk assessment was carried out. A brief review of this risk assessment process is provided in Appendix A.

11 – CONCLUSIONS

11.1 – Outcome of Study

The study has been successful in meeting its primary aims and objectives. The analysis contained within this report leads to the following key conclusions:

- The most representative V_L for Crossrail SCL tunnelling at the Whitechapel site, corresponding to the eastbound pilot tunnel construction, is 1.06%.
- The 1.06% V_L estimate correlates well with previous case studies and experimental data.
- No clear trend showing an improvement of volume losses with time is identifiable for the case studies of SCL tunnelling in LC.
- Existing LF approaches for estimating the V_L are supported by the additional case study evidence and a slightly modified best-fit line is proposed.
- In the majority of cases, sizeable cost savings would require a significant reduction in the assumed V_L at the design stage. However, all assumptions must remain sensible, realistic, and achievable on site.

11.2 – Recommendations

Following from the conclusions in Section 11.1, the following recommendations are made:

- The LF approach should continue to be used to obtain an order of magnitude estimate for the V_L at a site.
- The LF approach should only be used in conjunction with sound engineering judgement, with attention given to similar case studies and the data presented on the LF graph.
- When a structure is close to the boundary between two risk categories, consideration should be given to whether the V_L estimate used initially is accurate. If it is possible to reduce this estimate and specify careful control of the tunnelling, then an assessment should be made of the required percentage reduction of V_L for the structure's risk category to change.

11.3 – Potential Areas for Further Study

There is an obvious need for continued research into SCL tunnelling in LC. Such research may provide more data and help validate current empirical models. Future projects, such as High Speed 2 (HS2) and Crossrail 2, could provide ideal platforms on which to base further research. Some particular areas for further study arising from this report are:

- Collation of V_L and LF data for additional studies and subsequent population of the LF graph.
- A continued analysis of the development of V_L for SCL tunnelling in LC over time.
- An analysis of the long-term ground surface settlements at the Whitechapel site.

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APPENDIX A – REVIEW OF PROJECT RISK ASSESSMENT

The initial project risk assessment identified that, although no laboratory-based practical investigations were required, a significant amount of computer-based work was anticipated. Consequently, ergonomic risks arising from work station posture and risks to eyesight due to exposure to screens were considered to be the primary risks. On review, despite the high demand on computer-based activity, this risk has been well-managed with appropriate measures (including an adjustable working environment and a sensible working routine with regular short breaks) have been implemented throughout. Nevertheless, it was important that this risk was identified at an early stage.

The possibility of a site visit was also identified initially, but not discussed in detail. Therefore, when a site visit was arranged, it was necessary to carry out a further risk assessment. This supplementary assessment was site-specific and gave significantly greater detail. The supplementary risk assessment was approved internally. It identified essential steps to be taken prior to, and during, the site visit. A CSCS Construction Site Visitor card was mandatory for access to the site, and had been obtained pre-emptively several months prior. Extensive PPE was required and was also obtained in advance. A site-briefing was attended on-site and additional safety equipment was issued. Adhering to the correct safety procedures minimised the potential risk throughout the site visit.

Retrospectively, the initial risk assessment was successful in highlighting the most frequent risks. However, the assessment failed to provide sufficient detail for the site visit. Therefore, in future studies, it may be beneficial to confirm any site-based work as early as possible in order to facilitate a more accurate initial risk assessment and avoid additional work and time delays later in the project.