Field measurements above twin tunnels in London Clay

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Abstract

The Jubilee Line Extension (JLE) project in London, UK provided a useful opportunity to make detailed studies of the ground response due to bored tunnels. This thesis describes an instrumented greenfield site located in central London at St. James's Park, Westminster (Standing et al., 1996) where surface and subsurface measurements of displacements in three dimensions, and subsurface measurements of pore pressure and total stress changes were made above two 4.85m diameter JLE running tunnels which were driven from Waterloo, and through Westminster, to Green Park. The tunnels were excavated using open-faced shields through stiff overconsolidated London Clay, and were supported with expanded linings.

The aim of this research is to investigate and characterise the 3-dimensional ground response above and around single and twin tunnels in stiff clays through the field measurements. This includes the surface and subsurface movements for separate tunnelling events, the intermediate- to long-term displacement patterns, and the superposition of tunnel disturbance for twin tunnels. Comparisons are made with: case history data for London Clay tunnels; surface monitoring performed by the JLE contractor; and finite element modelling results presented by Addenbrooke (1996) and Addenbrooke et al. (1997). Conclusions are drawn regarding the applicability of using the empirical methods for predicting ground displacements which are used often.

The second aim of the research is to investigate the probable causes for the large ground disturbances which are observed above the two tunnels through the Westminster area and at the instrumented site. The settlement magnitudes were nearly twice the predicted values – the predictions being based on observations above tunnels with similar excavation methods and in similar ground. However, only 200m north of the site, the settlement magnitudes are observed to reduce by between one-third and one-half for both tunnels. This variation along the length of the route potentially holds the key to explaining the large settlements, and is investigated using the JLE site investigation results, observations at the tunnel face, ground displacements during and after construction, and long-term lining performance.
Acknowledgements

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Symbols list

\( C \)  
cover to tunnel crown (m)

\( D \)  
evacuated tunnel diameter (outer diameter of tunnel lining, m)

\( E_u \)  
undrained young's modulus (MPa)

\( E' \)  
drained young's modulus (MPa)

\( F_s \)  
factor-of-safety

\( G \)  
bulk shear modulus (MPa)

\( G(\alpha) \)  
cumulative probability function for settlement profile parallel to a tunnel axis given as:

\[
G(\alpha) = \left( \frac{1}{\sqrt{2\pi}} \right) \int_{-\alpha}^{\alpha} \exp \left( -\frac{\beta^2}{2} \right) d\beta
\]

For long tunnels where the face position is well advanced from the initial position (\( x_f \) much larger than \( x_0 \)), \( \alpha = (x-x_f)/l_x \). For positions behind the tunnel face where \( x < x_p \), \( G(\alpha) = 1 - G(\alpha) \). Note that \( G(0) = 0.5 \) (e.g. \( x-x_f=0 \), directly above the tunnel face), and \( G(\alpha) = 1 \) (e.g. at large distances behind the tunnel face and for large distances of tunnel progress)

\( i_y \)  
trough width parameter, equals the offset (in metres) from the tunnel centre-line to the point of inflection for a transverse Gaussian settlement profile

\( i_x \)  
trough length parameter

\( k \)  
permeability (m/s)

\( K \)  
trough width parameter, \( K = i_y/(s-z_0) \)

\( K_0 \)  
coefficient of effective earth pressure at rest

\( n \)  
percentage of support provided by tunnel lining immediately after excavation

\( N \)  
stability ratio

\( p, p' \)  
mean total stress, \( (\sigma_v+2\sigma_h)/3 \), and mean effective stress, \( (\sigma_v'+2\sigma_h')/3 \)

\( P \)  
shield length (m)

\( q \)  
deviatoric stress (kPa); \( (\sigma_v-\sigma_h) \) or \( (\sigma_v'-\sigma_h') \)

\( R \)  
evacuated tunnel radius (m)

\( R_{pl} \)  
plastic radius of plastically deforming zone

\( s_u \)  
undrained shear strength (kPa)

\( s' \)  
mean effective stress, \( (\sigma_v'+\sigma_h')/2 \)

\( t \)  
deviatoric stress, \( (\sigma_v-\sigma_h)/2 \) or \( (\sigma_v'-\sigma_h')/2 \)

\( u \)  
pore water pressure (kPa)

\( u,v,w \)  
ground displacements in \( x,y,z \) directions (m). Sign convention for displacements follows that of the coordinate system described above. Subscripts \( w_0 \) are for centre-line movements and \( w_{max} \) are for maximum movements.

\( V_1 \)  
the volume of the settlement trough per unit length \( (V_s) \) expressed as a percentage of the excavated tunnel volume per unit length of tunnel

\( V_2 \)  
the volume of the settlement trough per unit length of tunnel \( (m^3 \text{ per metre tunnel advance}) \)

\( x_{50} \)  
location of \( w_0/w_{0,\text{max}} = 50\% \) for the centre-line displacement profile

\( x_0,x_f \)  
initial or tunnel starting point, and the face or final tunnel position

\( x,y,z \)  
cartesian coordinates of any point in space. \( x=y=0 \) vertically above the tunnel face on the tunnel centre-line, \( z=0 \) at ground level. positive (+) \( x \) in the direction of tunnel advance, positive (+) \( y \) at right angles to \( x \) and towards the right when positive (+) \( x \) direction is into the page. Positive (+) \( z \) is upward vertically
\( z \) depth below ground surface
\( z_0 \) depth below ground surface to the tunnel axis being considered
\( z_{\text{focus}} \) depth below ground surface to the focus of displacement vectors in the transverse plane
\( z_g \) depth below ground surface to gravel-clay interface
\( \gamma_{\text{max}} \) maximum shear strain
\( \gamma_{xz} \) engineering shear strain in \( x-z \) plane
\( \gamma_{yz} \) engineering shear strain in \( y-z \) plane
\( \Delta/L \) deflection ratio
\( \varepsilon_I \) major principle strain
\( \varepsilon_{II} \) minor principle strain
\( \varepsilon_v \) volumetric strain
\( \varepsilon_x \) parallel horizontal strain
\( \varepsilon_y \) transverse horizontal strain
\( \varepsilon_z \) vertical strain
\( \sigma_h, \sigma_h' \) horizontal total stress and horizontal effective stress (kPa)
\( \sigma_t \) in-tunnel support pressure (kPa)
\( \sigma_v, \sigma_v' \) vertical total stress and vertical effective stress (kPa)
\( \sigma_{x'}, \sigma_{x'} \) horizontal total and effective stress (kPa) parallel to the tunnel axis
\( \sigma_{y'}, \sigma_{y'} \) horizontal total and effective stress (kPa) transverse to the tunnel axis
\( \sigma_z, \sigma_z' \) vertical total and effective stress (kPa)
\( \phi' \) angle of shearing resistance
I Introduction

1.1 General background

Maintaining and expanding urban infrastructures often necessitates tunnelling beneath densely populated areas to minimise the disruption at the ground surface. Projects aimed at improving utility services such as sewerage and storm-water systems, and transport system projects aimed at reducing congestion in city centres - both for road transport and metro schemes - are all utilising tunnelling. The prohibitively high cost of land in urban centres often makes tunnelling a financially more viable option. Athens, Los Angeles, Hong Kong, Sao Paolo, Toronto and London are just a few cities where major tunnelling projects for metro systems have either been undertaken in the past few years or are planned for the near future.

The construction of new tunnels is often performed in close proximity to existing subsurface structures (e.g. existing tunnels, deep foundations, services) and/or beneath surface structures (e.g. buildings, rail lines, embankments) for which small amounts of displacement could be unacceptable. In 1969, Peck reviewed the construction and effects of tunnels in his state-of-the-art report. He identified three categories which should be considered for tunnelling works:

- constructability
- performance
- influence on the existing infrastructure.

*Constructability* is considered as the overall design of the tunnels, adits, and shafts comprising the works, ensuring that they can be built safely and economically. It includes addressing the compatibility of the tunnelling system with the ground conditions and satisfying the design criteria in terms of lining loads and ground displacements, and it is also reflected in the capability of the soil to withstand the complex stress changes without tunnel instability or failure. Revision of the initial designs through value engineering or the observational method often form part of an iterative design process.

*Performance* reflects the ability of the constructed tunnels to withstand all anticipated influences. These may constitute operational forces, geological conditions (i.e. water pressures) or the possibility of future construction works as new tunnels are often constructed near to existing tunnels.

The last category, *influence on the existing infrastructure*, requires identification of buildings and structures at risk of damage. This itself is an onerous task for projects with extensive tunnelling as the variety of structure types encountered can be large, and the influence on structures can only be assessed if we have knowledge of (a) how the ground responds and (b) how the different structure types react to the disturbance. The public-at-large are increasingly aware both of construction activities and their potential impact, making the
accountability of the project designers and constructors to minimise any disturbance to the activities of groups or individuals very demanding.

Successfully satisfying the requirements for each of the above categories requires combinations of empirical assessments, model testing, and rigorous analytical and numerical analyses to sufficiently bound the potential ground disturbance likely to result from construction. Several authors in the past thirty years (e.g. Peck, 1969; Ward and Pender, 1982; Rankin 1988; Lake et al., 1992; Mair and Taylor, 1997) have noted that observing the response of the ground and of surface and subsurface structures during real tunnelling projects is the key to furthering our understanding of tunnelling-induced disturbance, and that there continues to be an inadequate database of case studies for tunnelling engineers and tunnel designers to learn from. This information gap gives the purpose to the research described in this dissertation.

1.2 Soft ground tunnelling
The majority of urban tunnelling comes under the heading of 'soft ground tunnelling': these are usually relatively shallow tunnels (i.e. less than 40m below ground level) which are excavated in potentially unstable soils. The excavation process may significantly disturb the surrounding soil near the tunnel, which in turn may be transmitted as settlements and straining to the overlying or adjacent structures.

Soft ground tunnels have been excavated throughout history, but modern tunnelling methods have only been developed in the past 200 years. Marc Isambard Brunel in 1819 was the first to utilise a shield for tunnel construction. In 1869 a shield of circular section used in advance of lining erection was introduced by Greathead: this shield was the model for most of the tunnelling machines to follow. Even today, shields of this type (incorporating modern advancements into the excavation, support, and jacking systems) are used for tunnelling in cohesive uniform soils such as clays. In the past 30 years tunnelling has split into two basic approaches: the New Austrian Tunnelling Method (NATM) or Sprayed Concrete Lining (SCL) method, and tunnel boring machines (TBMs).

NATM was originally used for hard rock tunnels and follows a general principle of smaller sequential headings supported after excavation by sprayed concrete support and rock bolting. The method allows for stress relief through ground displacements into the tunnel around the extrados which reduces the risk of rock bursts for deep tunnels. Applied to soft ground, the system has been modified severely, but still maintains the idea of a staggered heading advance with immediate support. The method has great flexibility for dealing with changes in cross-section with only minor interference to the tunnelling operation. It also has advantages for short tunnels, adits, cross-passages and tunnels of non-circular section.

TBMs have developed from open-face shields to full-face mechanised shields which provide temporary support at the tunnel face: these have enabled tunnel excavation to be performed through difficult or highly variable ground which would otherwise have been deemed unsuitable. TBMs can be grouped into open-face, earth pressure balance (EPB) and slurry machines as schematically shown on Figure 1.1. The open face TBM provides no continuous
support at the face, but may be equipped with doors that can be closed in case of water-bearing or unstable ground, or breasting plates that can be lowered to support the upper half of the heading. They are used primarily in competent and clay soils only. Compressed air may also be used in combination with an open-faced shield to reduce groundwater inflows and provide some face support.

EPB machines (Figure 1.1) can provide positive face pressure during tunnel advance. They have a separate sealed chamber at the front of the shield which fills with soil cuttings, and pressure may be exerted by pushing the cuttings against the face by a bulkhead wall separating the sealed chamber from the rest of the TBM. An archimedes type screw conveyor is used to remove spoil from the chamber. Mechanical transmission of force from the cutting face and the pressurised spoil in the front chamber provides the support at the face; pressures are controlled by manipulating the tunnel advance speed, the cutting rate, and the spoil removal. EPB machines are generally more successful in soils with fairly uniform stiffness (Eisenstein, 1995). The machines may also be run in 'open' mode where the front chamber is not sealed and therefore little or no positive pressure is exerted at the face.

The bentonite slurry TBM operates with hydraulic face support provided via a fluid pressure in a chamber at the front of the shield and sealed from the rest of the machine. The chamber also contains the cutting wheel. Spoil enters the slurry chamber and is circulated with the bentonite to a separation plant - usually at the ground surface - where it is removed. The fluid support is effective in controlling face stability even in highly variable ground, but the circulation and separation systems for the slurry complicate the tunnelling and support processes.

For an open-faced shield the potential ground disturbance above a tunnel may be considered qualitatively in terms of the instability or risk of failure relative to a progressing tunnel face as shown in Figure 1.2; the most critical position in the ground is found immediately above the tunnel face and prior to lining erection. This conclusion is arrived at from the available case history data which show that there are very few incidents of tunnel collapse for tunnels of conventional lining design which were not related to initial problems at the tunnel face. Ground response (e.g. settlement) reflects the risk, with the majority of settlement and overall ground disturbance during construction occurring near the open excavation. In this thesis the response of the ground around, above, and in advance of open-faced shields in stiff clay is investigated.

1.3 The Jubilee Line Extension Project
The beginning of the Jubilee Line Extension (JLE) Project in 1994 provided the opportunity to observe in detail tunnelling-induced ground disturbances; nearly 15km of twin bored tunnels, five deep station excavations, three tunnelled stations, and enlargements to existing stations were constructed as part of the underground works on the project. The route alignment is shown on Figure 1.3.

The extension starts from the existing Jubilee Line at Green Park and passes southward through St. James's area, St. James's Park and Westminster, and beneath the river Thames to
Waterloo. The line then continues along the south side of the river from Waterloo, through London Bridge and Bermondsey, and on to Canada Water. The tunnels then cross under the Thames another three times, joining Canary Wharf and North Greenwich stations before surfacing prior to Canning Town. The line continues on the surface north-eastward to meet the existing British Rail station at Stratford. The extension to the Jubilee Line is the first of the underground system to progress into London's East End which has previously remained outside the underground system.

The project geology through which the running tunnels were driven is shown on Figure 1.4 and is conveniently divided into three distinct sections: the western part of the route (Green Park to London Bridge) lies almost entirely in the London Clay formation; the central portion from London Bridge to North Greenwich was excavated in the more variable and water-bearing beds of the Lambeth Group (formerly Woolwich and Reading Beds), or in the Thanet Sands; the eastern-most tunnelling works beyond North Greenwich were in London Clay before the tunnels rise to the surface.

The excavations were beneath and adjacent to some particularly sensitive and historic landmarks of the United Kingdom. JLE set its tolerances for potential damage to structures in the 'Slight' damage category of the UK Building Research Establishment's risk assessment classification system (Burland et al., 1977) and assessed the potential damage of over 4000 structures that might have been affected by the project works. Movement control systems external to the tunnel construction were used extensively to protect sensitive structures, although often these were originally intended as contingency systems only to be used if necessary.

1.4 Research description and aims
The main objective of this thesis is to contribute knowledge and understanding of greenfield surface and subsurface ground response due to bored tunnelling in stiff clay through the analysis of field measurements made using state-of-the-art instrumentation. The chosen greenfield site is between Westminster and Green Park in St.James's Park.

During tunnelling through the Westminster area, it quickly became clear that the tunnelling-induced movements were much larger than expected. Thus, in parallel with the characterisation of ground response, investigation of the possible causes for these large ground movements became an important focus of the research project. Although no further site investigation work has been undertaken, data presented in this thesis are aimed at guiding future work which should be undertaken to explain the unexpectedly large ground displacements. The research objectives of this thesis are detailed below:

- to review literature on the subject of ground response due to bored tunnelling in soft ground, with particular attention given to case histories for tunnels in stiff overconsolidated clays and the empirical methods for evaluating the ground response.
- to describe in detail the project works at St. James's Park, the site investigation results and the material behaviour of the London Clay.
to characterise the 3-dimensional ground displacements around the twin tunnels at St. James's Park and relate observations to the empirical soil response framework.

- to measure total stress and piezometric changes due to tunnel construction in a stiff overconsolidated clay.

- to measure the time-dependent ground response, including displacements and associated total stress and piezometric changes, after tunnel construction. Information from load cells incorporated into one of the newly built lining rings by the Transport Research Laboratory (TRL) will also be included to complement these data.

- to evaluate the precision and accuracy of the instrumentation and monitoring techniques used over both short- and long-time periods.

- to investigate the probable causes of the large displacements above the JLE tunnels at and near to the instrumented section.

Other research at Imperial College is being performed in parallel with this greenfield monitoring through a predominantly industry-funded cooperative LINK Construction Maintenance and Refurbishment Programme project entitled 'Subsidence damage to buildings: Prediction, protection and repair' (Burland et al, 1996). This complementary project monitored the response of numerous structures along the JLE route affected by tunnelling. The collection of buildings comprised a wide range of structure and foundation types, including masonry buildings with load-bearing walls on massive raft foundations, framed structures with piled foundations, and low-rise brick buildings with traditional strip footings. This thesis thus provides a frame of reference to assess soil-structure interaction for these buildings.

1.5 Organisation of the thesis

Chapter 2 contains a literature review in which the empirical methods currently used in the UK tunnelling industry to assess potential ground response are presented, with some less detailed comments on analytical and numerical approaches to the same problem. References are made to case histories of tunnelling-induced ground response, including both field measurements and physical modelling.

Chapter 3 describes the geological and recent history of the St. James's Park area, and summarises the relevant JLE site investigation results. Comparisons are shown with other data from London Clay sites to demonstrate geological differences and similarities.

In Chapter 4 the instrumentation used at the site and the monitoring techniques are described, and the precision and accuracy of measurement are established. Attention is also given to describing the methods of data analysis.

Chapter 5 describes the construction details for the site and includes the identification of construction periods, details of the instrumentation layout at the control site, locations of the contractor's surface monitoring points, and details of the tunnel shields and the lining used.

Chapter 6 presents the measurements made at St. James's Park which encompass: the base line data taken prior to tunnelling at the instrumented section to evaluate the instrument
performances; the ground response during construction of both tunnels relating the data to
collection activities and tunnel face positions; the time-dependent response seen in the interim
period between the two construction events; and the longer-term measurements after completion
of both tunnels.

Chapter 7 contains analyses of the measurements presented in Chapter 6, relating the
observed soil response to the expected behaviour, and to the empirical framework for ground
response. Particular emphasis is placed on the subsurface ground response above and around
the tunnel headings. Comparisons are made with other field data around tunnels in London
Clay.

Chapter 8 draws together the key aspects of the monitoring results and their applications
to tunnelling in urban areas. Future work is suggested which would give a better understanding
of the large settlements which were observed above these tunnels.

All of the figures and tables referred to in the text are contained in volume 2 of this
thesis.
2 Assessing potential ground response to bored tunnelling

The first priority for tunnel design and construction is stability with constructability; is the tunnel able to be constructed without failure? Stability analysis involves defining the modes of failure and the limiting criteria for suitable design. The analysis that follows could be a relatively simple assessment using empirical relations derived from previous tunnelling experience, laboratory work and model testing. Alternatively, it could also comprise in-depth numerical modelling using complex constitutive soil models to represent the strain response due to stress changes. In any event, once it is established that the tunnel can be built, the measure of stability (for example, a factor of safety) may be used to predict the potential disturbance to the overlying and surrounding soil based on previous experience, model testing, theoretical solutions and numerical analysis.

Ground response is divided in this thesis into surface displacements, subsurface displacements, and pore water pressure and total stress changes. Displacements are further divided into two separate idealised profiles: transverse to the tunnel axis and parallel to the tunnel axis.

Two phases of response are recognisable above a tunnel: the immediate phase where the monitored response reflects the construction activities, and the much slower time-dependent phase. The immediate phase is usually considered to have the greatest potential to affect surface and subsurface structures adversely, because of the uncertainties associated with tunnel construction. Even if the tunnel is excavated without incident, the speed with which immediate displacements occur is likely to be a major contributor to the potential for damage to urban structures. The second phase, time-dependent response, arises primarily from a combination of consolidation of clayey soils, redistribution of pore water pressures, lining deflection, void collapse, and possible recompaction of more granular materials. This response phase is regarded as having a much lower potential for adversely affecting surface structures even though the displacement magnitudes are often as large or larger than those resulting from tunnel construction. The lack of long-term field measurements makes predicting magnitudes and forms of time-related changes difficult. A growing number of good quality case studies are available for surface movements, but historical records of subsurface displacement measurements and pore pressure and total stress observations are rare because of the cost and difficulty in obtaining good data.

This chapter focuses on the prediction of tunnel stability, 3-dimensional surface and subsurface displacements, and pore pressure responses and total stress changes. Reference is made mainly to empirical methods of prediction applicable to tunnels in cohesive materials with some discussion of numerical techniques used.
2.1 Changes around a constructed tunnel

The excavation of a tunnel in soft ground provides an opening into which the soil can deform, and its doing so changes the stress regime in the excavated material. Excavation also introduces a new boundary where permeability and stiffness attributes are significantly different, and the soil at the boundary is significantly altered. In general, tunnelling will cause a reduction in mean total stress and an increase in the total shear stress in the soil near the tunnel (Atkinson and Mair, 1981; Ward and Pender, 1981). Careful consideration must also be given to the mode of soil response (drained vs. undrained), recent and formative stress histories, and the appropriate stress path followed by an element of soil near the tunnel. The soil response during construction may also alter the longer-term patterns of response after the tunnel has been completed.

In stiff clayey soils the response during construction is assumed to be undrained with significant pore water pressure and total stress changes around the tunnel extrados, but with no change in mean effective stress, $\rho'$. In general, tunnels in soils with permeabilities less than about $10^{-7}$ m/s and with excavation rates of about 1 m/h or more may be considered undrained; periods of stand-still can lead to drained conditions becoming more relevant.

Soil near the excavated face and around the shield may experience severe and rapid rotation of the stresses from the in-situ conditions (i.e. $\sigma_i = \sigma_\text{ax}$, $\sigma_{\text{mx}} = \sigma_\text{tr}$) to a minor stress oriented radially to the tunnel and a major stress oriented tangential to the tunnel excavation. At the tunnel face and around the shield the excavation is unsupported for a short time, and the soil moves and expands into the unsupported face and extrados. The vertical and horizontal stresses redistribute and 'arch' around the excavation. In some areas the soil is almost invariably disturbed beyond its elastic region and plastic deformations occur. As the cutting edge is pushed forward through the soil, further deformation may occur as a result of frictional (shear) stresses along the shield if the gap around the shield is closed quickly. By the time the lining is erected and the tail skin has passed, the soil near the extrados may have been disturbed to a remoulded state.

The effect of tunnel construction on the surrounding soil may be simplified by considering the idealised effective stress response under plane strain conditions at (1) the tunnel crown and (2) the tunnel spring line: these are shown on Figure 2.1a in engineering stress space ($\sigma_i', \sigma_\text{tr}'$) and on Figure 2.1b in plane strain deviatoric stress space ($\tau', s'$).

The depositional history of a typical overconsolidated clay is represented by the path OAB on both plots. At A the deposition of additional material above the point of interest has ceased; A to B represents the removal of overburden stresses to reach a moderate to heavily overconsolidated state at B.

During tunnel construction the soil near the tunnel crown (1) experiences a rapid drop in $\sigma_\text{tr}'$, while $\sigma_i'$ increases: this is comparable to an undrained vertical triaxial extension test with the vertical (axial) stress reduced while increasing the confining stress. In engineering stress space the path travels from B to the idealised failure envelope: if the stress changes are sufficient, plastic yielding occurs as the path moves along the failure envelope to C1. Overconsolidated London Clay is anticipated to dilate along the yield surface leading to negative...
pore pressures and an increase in the mean effective stress. In $t'$-$s'$ space the same path for soil at the crown (1) is traced as a reduction in both $t'$ and $s'$. The path moves from the initial position at B to reach an idealised failure envelope. Plastic yielding follows as the path moves along the failure line to C1. At the crown only small changes to the initial directions of principal stress may be realised during excavation for this idealisation. The path change from C1 to D1 represents loading due to expansion of the lining.

At the tunnel spring line (2) the excavation induces a large decrease in $\sigma'_h$ (see Figure 2.1) and a smaller increase in $\sigma'_v$. In engineering stress space the decrease in $\sigma'_h$ brings the soil from B to a point where $\sigma'_h=\sigma'_v$, beyond which $\sigma'_v$ becomes the larger stress. The path continues towards the failure line; the soil may yield along the failure envelope line to point C2. In $t'$-$s'$ space these changes are reflected as a positive increase in $t'$ and a decrease in $s'$. The stress path moves upwards across the $s'$ axis ($t'=0$ at $\sigma_h=\sigma_v$) to reach the failure envelope, and the soil yields along the envelope to C2. Again C2 to D2 represents loading due to expansion of the lining for both stress paths.

These sorts of stress paths may be applied to samples during laboratory testing to best reproduce the different soil responses for the tunnel crown and axis. Comparisons of laboratory stress paths and paths estimated by finite element analyses were reported by Ng and Lo (1985) with particular reference to normally consolidated clays. Burland and Hellings (1986) noted that the soil response is likely to be much more complicated than these typical stress paths. At the tunnel crown, they suggested that overconsolidated soils follow a vertical extension-type path which they defined as a 'passive stress relief path' (see Figure 2.1a). The soil, on reaching a state of failure, responds to unloading by yielding along an idealised failure envelope towards the origin. The failure is localised and is notionally contained by the interaction of stresses and strains acting within the soil, and by the soil surrounding the area of failure, such that overall tunnel stability is maintained: dilation results from shear strain development and swelling while moving along the failure envelope. Laboratory tests on London Clay where samples were subjected to a 'passive stress relief path' (Burland and Fourie, 1985) showed very close agreement with the failure envelope defined by vertical extension tests (Tedd and Charles, 1985) but showed much larger strengths at low mean stresses.

Clearly the response is extremely complex; the simplification above provides only a general idea of the soil response in overconsolidated clays. Ng and Lo (1985) emphasise that the testing procedures must closely resemble the real stress changes occurring in the ground so that the strength and deformation characteristics of the soil in both drained and undrained conditions may be better understood. The use of Finite Element Methods (FEM) which employ the constitutive soil models developed from laboratory testing are extremely useful for predicting both the displacements and total stress changes during construction near a tunnel. The influence of soil macrofabric (joints, fissures, etc.) on the bulk properties of the soil may also be significant to the overall response; this subject is discussed in Chapter 3 and Appendix A.
2.2 Tunnel stability

Assessing the stability of a tunnel heading requires an understanding of the tunnel influence on the soil, but without complex analysis it is impossible to estimate accurately the stress changes in the ground. However by using simplified and idealised geometries as shown in Figure 2.2, important features of stability can be illustrated.

2.2.1 Undrained stability

Broms and Bennermark (1967) investigating the stability of vertical openings in walls retaining cohesive material described a stability ratio, \( N \), to be applied to tunnel face stability as

\[
N = \frac{(\sigma_v - \sigma_i)}{s_u}
\]

where \( \sigma_v \) = vertical stress at the tunnel axis level (including any surcharge)

\( \sigma_i \) = tunnel support pressure (i.e. compressed air, EPB and slurry machine support)

\( s_u \) = undrained shear strength at tunnel axis level

The lower the value of \( N \), the higher the stability of the tunnel face. Atkinson and Mair (1981) discussed tunnel stability in a factor-of-safety framework similar to that employed for foundation and retaining wall design. For tunnels in cohesive materials, for which excavation occurs under undrained conditions, the tunnel support pressure required to maintain a factor of safety, \( F_s \), is given by

\[
\sigma_i = \sigma_v - \frac{s_u T_c}{F_s}
\]

where \( T_c \) is a dimensionless stability number. They suggest that for soils with varying undrained strength it would be sensible to use a mean value of \( s_u \) over the depth to the tunnel axis. It can be shown by rearranging Equation 2.2 that

\[
\frac{(\sigma_v - \sigma_i)}{s_u} = \frac{T_c}{F_s} = N
\]

where \( N \) is the stability ratio of Broms and Bennermark. At failure \( (F_s=1) \) \( T_c \) equals a critical stability ratio, termed \( N_{nc} \) by Mair et al. (1981). Values of \( N_{nc} \) based on theoretical analysis of plane sections through circular tunnels were given by Davis et al. (1980) who developed lower bound solutions for perfectly plastic material for two important cases: (a) the stability of a circular tunnel lined right up to the tunnel face, and (b) the stability of a long unlined circular tunnel for varying cover (C) and diameter (D) ratios (see Figure 2.2). Centrifuge tests reported by Kimura and Mair (1981) show good agreement with these, but demonstrated that tunnel stability in cohesive soils also depends strongly on the length of unsupported ground at the heading (P). Values of \( N_{nc} \) determined from these model tests and theoretical analyses for different ratios of \( P/D \) and \( C/D \) are given in Figure 2.3. Increasing \( P/D \) for a given geometry...
results in a significant reduction in $N_{eq}$ (i.e. increasing stability) for a specific cover to depth ratio. Increasing $C/D$ tends to increase $N_{eq}$ (i.e. reduces stability) but the changes become less significant for $C/D$ ratios greater than 3.

**Figure 2.4** shows various guidelines for using the stability ratio. Broms and Bennermark (1967) suggested that a tunnel with a stability factor greater than about 6 is likely to be unstable. Peck (1969) states that a stability factor of 5 marks the limit for tunnelling without unusual difficulties in saturated plastic clays, based on ten case histories of tunnelling in stiff, soft, and sensitive plastic clays. Attewell and Boden (1971) extended the laboratory work of Broms and Bennermark and showed that instability could occur for ratios above 4.5. For typical $C/D$ and $P/D$ ratios, Mair (1983) suggested a limit value of about 5 for most tunnelling geometries. At the other extreme, Ward and Pender (1981) argued that for stability ratios of about 1 (i.e. deep tunnels) the movements were likely to be small and elastic.

Lake *et al.* (1992) contended that generally, where $N$ is greater than 6 there is likely to be general face instability. Between $N=4$ and 6, general plastic yielding is most likely to occur, while between 2 and 4 only localised plastic yielding will occur. For stability ratios less than 2, the response is more likely to be essentially elastic and the face stable.

### 2.2.2 Drained stability

Drained stability for the tunnel idealisation depicted in **Figure 2.2** was investigated by Atkinson and Potts (1977) who derived upper- and lower-bound plasticity solutions for dry granular soils. Centrifuge model tests, also performed by Atkinson and Potts, showed that support pressures were nearly independent of the $P/D$ ratio. Plane strain modelling results were generally consistent with their plasticity solutions and they inferred that the support pressure was also nearly independent of $C/D$. Later investigations by Leca and Dormieux (1990) on fully lined tunnels ($P/D=0$) showed an increase in support pressures with $C/D$ which contrast with the experimental results. Nevertheless, it was concluded by Atkinson and Potts that the support pressure required for tunnel stability in dry cohesionless soils is only a small percentage of the overburden stresses acting at the tunnel.

In contrast, tunnelling undertaken beneath the water table in granular material generally requires much larger support pressures to prevent water inflow. This is achieved increasingly often by using closed-faced EPB or slurry machines. The author refers the reader to Anagnostou and Kovari (1996) who considered full-face TBM stability based on limit equilibrium, and Jancscoz and Steiner (1994) who investigated the face stability of slurry machines. In general, considerable judgement must be made; defining the performance of these shields may be difficult for complex ground conditions, and simplifications of the tunnelling scenario for limit equilibrium calculations should be viewed with caution.

### 2.3 Numerical modelling of ground response

Assessment of ground response is generally divided into three groups: empirical methods, closed-form solutions and Finite Element Methods (FEM). Good practice in tunnel design would be
to utilise all three methods of analysis to establish a complete picture of the potential ground response.

The empirical methods and closed-form solutions are discussed in more depth in Section 2.4. However, it is worth noting here that applications of these methods are very much constrained by experience, either from physical modelling of the tunnelling scenario or from case histories where tunnelling has been performed in similar ground and using similar construction techniques.

FEM is a significantly more powerful tool for assessing the ground response, and its usage in tunnelling has seen a dramatic rise in the past 20 years. However, modelling the complex problem of tunnelling is often dependent on the same judgement that is required for the other predictive techniques.

At present, 2-dimensional (2D) idealisations of the 3-dimensional (3D) tunnelling problem are the most common way of utilising FEM. 2D analysis is particularly effective if face movements into the shield are small and the response pattern is dominated instead by movements around the tail void after shield passage (e.g. full-face machines). Clough and Leca (1989) reviewed the use of numerical analyses in tunnelling: they resolved that the cost and time necessary to assess the construction aspects of a tunnelling geometry with a realistic soil model using 3-dimensional modelling were prohibitive. Mair and Taylor (1997) note that the situation has changed very little in ten years, even with the enormous increase in computing power.

2-dimensional FEM commonly uses the convergent-confinement method to model the tunnel excavation (Panet and Guenot, 1982). Forces are applied at the tunnel boundary to match those imposed by the soil, and are then reduced incrementally to allow movement into the tunnel. The method may be stress controlled (i.e. to install the lining in the analyses after a specified unloading) or displacement controlled where the movements into the tunnel are prescribed and unloading is permitted until specified levels are achieved. Rowe et al. (1983) proposed using a 'gap' parameter for displacement-controlled modelling which defines the amount of movement at the tunnel boundary; the gap is derived from physical attributes of the TBM, an estimate of work quality, and an estimate of 3-dimensional movements or out-of-plan effects (see Section 2.4.4). With either approach, the limit values selected require considerable engineering judgement and experience of both real tunnelling and numerical modelling of tunnels.

Refinements which are based on back-analyses of previous tunnelling works may be applied to improve predictions of the ground response above tunnels. Progressive softening (Svoboda, 1979) necessitates a reduction in the soil stiffness near the tunnel to mimic the 3-dimensional effects in advance of the tunnel face. Local reduction of the coefficient of earth pressure at rest \( K_0 \) adjacent to tunnel spring lines is used for modelling in overconsolidated clays (Addenbrooke, 1996) to improve predictions of near surface settlement profiles.

The constitutive soil models have also undergone major advancements in the past 20 years. Isotropic linear-elastic perfectly-plastic models severely overpredict the trough width and underestimate the observed displacement magnitudes. Lee and Rowe (1989) demonstrated that using anisotropic elastic models with lower relative stiffness ratios of \( G_{th} E' \), better model
surface profiles of settlement in soft normally consolidated clays. In stiff overconsolidated London Clay it was found by Gunn (1993) that isotropic non-linear 'small-strain' elastic stiffness models (e.g. Jardine et al., 1986) significantly improve the predictions relative to the isotropic linear elastic models. Simpson et al. (1996) showed the shape of the settlement trough to be sensitive to anisotropic non-linear shear moduli.

Stallebrass et al. (1996) showed numerical analyses of 2D centrifuge tests of tunnels in overconsolidated kaolin (Grant and Taylor, 1996). Their soil model utilises kinematic surfaces in $q'$-p' space which specify a plastic yield surface and a recent stress history surface to account for the effects of previous loading; the result is a non-linear stress-strain response where shear and volumetric straining are linked. Their numerical results overestimate the trough widths and underestimate the settlement magnitudes for the centrifuge models even though there are no 3-dimensional effects.

Numerical 2D modelling results of the tunnel geometry at St. James’s Park using several different pre-failure soil models were presented by Addenbrooke et al. (1997). They showed that the non-linear models produce predictions of deeper and narrower settlement troughs than linear models, although the isotropic non-linear models still showed wider settlement troughs at the surface than those observed at the site. The introduction of anisotropic parameters realistic to the London Clay into the non-linear models made only small improvements to the comparison with the field data. Using a very soft shear modulus $G_s/E' = 0.2$ (note: $G_s/E'$ is about 0.44 from the field and laboratory measurements presented by Simpson et al., 1996) in the anisotropic model improved the back-analysis comparison at the surface considerably. Near the tunnel this change yields larger movements inwards at the tunnel spring lines and generally larger horizontal displacements over the full trough. The data presented by Addenbrooke (1996) and Addenbrooke et al. (1997) are compared with the field data in Chapter 7.

2.4 Empirical methods for predicting surface displacements

2.4.1 Vertical displacements

The most commonly used approach for predicting vertical displacements at the surface was presented by Peck (1969), reporting on work performed by Schmidt (1969). He proposed that a displacement trough occurs transversely above constructed tunnels and that these vertical movements roughly follow an inverted mean-normal or inverted Gaussian distribution as pictured on Figure 2.5a. Backed with additional field measurements, O'Reilly and New (1982) and Attewell and Woodman (1982) developed Peck’s proposition into a simple mathematical form, given as

$$w = w_{max} e^{-\frac{x^2}{2\sigma^2}}$$

which describes the 2-dimensional vertical surface displacement profiles transverse to an infinitely long bored tunnel. $w_{max}$ represents the maximum surface displacement at the tunnel
centre-line, \( y \) is the offset from the tunnel centre-line, and \( i_1 \) is the trough width parameter equal to the offset from the tunnel centre-line to the point of inflection in the profile. The normal distribution has theoretical justification (Schmidt, 1969) based on the idealised mass behaviour of discrete particles or discs as shown on the lower half of Figure 2.5a. Figure 2.5b shows the normalised displacement, slope and curvature profiles derived from Equation 2.4. The maximum slope is found at the point of inflection \((i_1)\), the maximum curvature in hogging deformation at \(\sqrt{3i_1}\), and the maximum curvature in sagging deformation at \(y=0\).

Attewell and Woodman (1982) recognised that the profile described by Equation 2.4 develops in magnitude as a tunnel face progresses towards and beneath the section of interest. They proposed that a profile of vertical displacements parallel to the tunnel axis follows a cumulative probability function based on the normal distribution as defined in Figure 2.6a. The equation which describes the centre-line settlement profile is given by

\[
w_p = w_{\text{max}} \left( G\left[\frac{x-x_1}{i_t}\right] - G\left[\frac{x-x_1}{i_2}\right]\right)
\]

where \(G(n)\) is the cumulative probability function evaluated for \(n\) from statistical tables, \(i_1\) is the trough length parameter, \(x\) is the position of interest, \(x_1\) is the start of the tunnel, \(x_2\) is the end of the tunnel or the position of the tunnel face. The function and its application to longitudinal displacements (settlement, slope and curvature) are shown on Figures 2.6b and 2.6c. The maximum slope for this idealised case occurs immediately above the tunnel face, when the ratio of \(w_p/w_{\text{max}}\) is equal to 0.5. The maximum curvatures for hogging and sagging deformation are found at \(x=\pm i_1\). Measured profiles above a tunnel in London Clay reproduced in Figure 2.7 show good agreement with Equation 2.5, but generally Attewell and Woodman (1982) indicated that in firm to stiff clays \(w_p/w_{\text{max}}\) at the face position lies between 0.3 and 0.5 in practice; this almost certainly depends on the tunnelling methods employed.

An idealised 3-dimensional surface trough is found by merging Equations 2.4 and 2.5 to give the expression

\[
w_{(3d)} = w_{\text{max}} e^{-(1+2i_1)} \left( G\left[\frac{x-x_1}{i_t}\right] - G\left[\frac{x-x_1}{i_2}\right]\right)
\]

A dimensionless plot of Equation 2.6 given in Figure 2.8 shows the trough surface and the coordinate axis system with the origin at the ground surface immediately above the tunnel face. The practical limits of the trough are also identified, corresponding to a ratio of settlement to maximum settlement equal to about 0.01. The maximum trough half-width of the transverse profile well behind the tunnel face is \(3i_1\), while in the longitudinal section the trough half-length lies between \(i_1\) and \(2i_1\). It should be noted that because the last term of Equation 2.6 is essentially a scalar value (if \(i_1\) is assumed constant), it is implicitly assumed that any transverse settlement profile has the same form given by Equation 2.4 irrespective of offset from the tunnel parallel to the axis, \(x\), but with reduced magnitude.
2.4.2 Horizontal displacements

During tunnel construction both vertical and horizontal displacements occur. For the transverse trough it is often assumed that the vectors of displacement are directed at the tunnel axis level although there is very little evidence to support this. Furthermore, numerical modelling using non-linear stiffness models suggests that the assumption is likely to overpredict horizontal movements significantly (Lake et al., 1992). Nevertheless, for a point sink at the tunnel axis, predicted horizontal displacements for a transverse profile well behind the tunnel face at an offset $y$ are given by

$$v_y = w_y \frac{y}{z_o}$$  \hspace{1cm} (2.7)

where $z_o$ equals the depth to the tunnel axis, and $w_y$ is vertical displacement as defined in Equation 2.4. It can be shown that the maximum magnitude of horizontal displacement is given by

$$|v_{\text{max}}| = |e^{-(0.5)w_{\text{max}} \frac{i}{z_o}}|$$  \hspace{1cm} (2.8)

Figure 2.9 shows the idealised distribution of $v_y/w_{\text{max}}$ with normalised offset. The maximum transverse horizontal displacement occurs at $y=i$, while no displacements are seen at the tunnel centre-line.

Horizontal movements parallel to the tunnel axis are almost always ignored in the literature as their measurement in practice is difficult and the movements are transient in nature. Displacement vectors along the x-axis in the empirical framework are assumed to be aimed at the centre of the tunnel face. This yields a mean normal distribution of parallel horizontal displacements, with the maximum movement occurring immediately above the tunnel face (Attewell and Woodman, 1982; Attewell et al., 1986) as follows:

$$u_x = \frac{w_{\text{max}} i_x}{z_0 \sqrt{2\pi}} \left( e^{-ix^2/2z_o^2} - e^{-ix^2/2z_o^2} \right)$$  \hspace{1cm} (2.9)

Immediately above the face of a long tunnel ($x=0, x=\infty, x=0$) it can be shown that the maximum displacement magnitude is given by

$$|u_{\text{max}}| = \frac{-1}{\sqrt{2\pi}} \frac{w_{\text{max}} i_x}{z_0}$$  \hspace{1cm} (2.10)

A normalised profile ($u_x/u_{\text{max}}$) is given in Figure 2.10 for a long tunnel. An attribute of Equation 2.9 is that well behind the tunnel face ($x>>x$), the net horizontal displacements parallel to the tunnel are zero. Equations 2.7 and 2.9 may be combined to obtain the total horizontal surface displacement at any $x,y$ location (see Attewell and Woodman, 1982).
2.4.3 Ground slopes, curvatures and strains

The potential effects of ground displacements on structures are assessed using displacements, slopes, strains and curvatures. Relationships for each of these can be derived from Equations 2.4, 2.5, 2.7 and 2.9 for both transverse and longitudinal profiles (see Attewell and Woodman, 1982; Lake et al., 1992); these are compared in Table 2.1 for the parallel profile on the centre line and the final transverse profile, maintaining the assumption of displacement vectors aimed at the tunnel axis. The ratios of parallel to transverse magnitudes imply that the transverse profile is potentially more damaging to structures.

Measurements presented by Bowers et al. (1996) given on Figure 2.11 show that transverse strain magnitudes above a tunnel in London Clay were less than predicted strains calculated for a point sink, but that the locations of maximum compressive and tensile strains were in reasonable agreement with the idealised offsets given in Table 2.1.

2.4.4 Estimation of $w_{\text{max}}$ from crown displacements

Maximum surface settlement can be estimated by assuming a ratio of $w_{\text{max}}/w_c$, where $w_c$ equals the settlement at the tunnel crown. Atkinson and Potts (1977) derived a simple relationship from model tests described by Equation 2.11:

$$ \frac{w_{\text{max}}}{w_c} = 1 - \alpha \left( \frac{C}{D} \right) > 0 $$

(2.11)

$C$ and $D$ are the diameter of the tunnel and the soil cover above the tunnel crown as given in Figure 2.2, and $\alpha$ is a measure of the dilation of the ground (large for dense sands and small for normally consolidated or lightly overconsolidated clays). Field data summarised by Ward and Pender (1981) are given on Figure 2.12 plotted as $w_{\text{max}}/w_c$ versus $C/D$; superimposed are the relationships from Atkinson and Potts (1977) for loose sands and overconsolidated kaolin. Their sand line provides a lower bound to the field data of Ward and Pender. Although there is significant scatter, the data suggest a ratio of $w_{\text{max}}/w_c$ of between 0.25 and 0.5 for $C/D$ ratios greater than about 4; Craig (1975) also concluded a similar range for observed settlements above open-faced shield tunnels in stiff clays.

Clough and Schmidt (1981) suggested the following empirical relationship between surface and crown settlement:

$$ \frac{w_{\text{max}}}{w_c} = \left( \frac{2R}{z_0} \right)^{0.8} $$

(2.12)

It is shown on Figure 2.12 that this equation lies between the two relationships given by Atkinson and Potts (1977).

Lo et al. (1984) define the gap parameter, $G$, as a measure of the displacements at the tunnel crown. The parameter has three components as seen in Equation 2.13: the physical gap left by the shield, $\Delta$ (i.e. the bead thickness or the thickness of the tail piece); the clearance required for erection of the lining within the tail piece, $\delta$; and a displacement parameter, $U$,
which accounts for the three-dimensional elasto-plastic deformation at the face, \( u_{e+s} \) (i.e. movements due to face losses) and workmanship, \( W \).

\[
G = 2\Delta + \delta + U, \quad U = u_{e+s} + W
\]  

(2.13)

Procedures for estimating the gap parameter are outlined by Lo et al. (1984); \( \Delta \) and \( \delta \) are derived from the tunnel machine (i.e. bead size) and lining selections (e.g. expanded or bolted rings). \( u_{e+s} \) is estimated empirically from plane strain analytical solutions of a collapsing cylindrical cavity in a linear-elastic perfectly-plastic soil. The work quality, \( W \), is assessed from a qualitative review of the tunnelling methods, potential difficulties and proposed degree of supervision. Lo et al. (1984) and Ng (1984) back-analysed case records for bored tunnels in clays by calculating the crown displacements using Equation 2.13 and found a strong relationship between the crown displacement normalised by tunnel radius (\( w_c/R \)) and stability ratio. They also noted that the relationship between the observed \( w_{\text{max}} \) and the calculated \( w_c \) is nearly linear with \( w_{\text{max}}/w_c = 0.33 \). Value is added to their assessment method as their back-analyses gave reasonable agreement for cases which showed a wide variation in observed displacement magnitudes but with similar tunnelling geometries (i.e. cover and diameter).

### 2.4.5 Estimation of \( w_{\text{max}} \) from ground loss

Tunnel construction inevitably necessitates excavating larger amounts of soil than the tunnel volume replaces in order to facilitate shield advance, steering, and ring construction. Volume loss or ground loss, \( V_1 \), for tunnel construction is given notionally as the difference between the volume of excavated material (\( V_e \)) and the volume of the outer diameter of the tunnel lining (\( V_n \), see Figure 2.13a) per metre of tunnel and is usually expressed as a percentage of the theoretical tunnel volume (\( \pi R^2 \) per metre of tunnel advance). \( V_1 \) is a useful representation of the total ground disturbance the tunnelling is causing in the surrounding soil, but it encompasses several independent and often unpredictable factors. The magnitudes of measured ground response and associated volume loss during tunnel construction are likely to reflect, at least in part, the following:

- tunnelling techniques (TBM types, NATM, etc.)
- geometry (C/D and P D ratios)
- work quality (encompassing over-cutting, duration of lining erection/unsupported excavation, control of face pressures)
- ground water conditions
- soil strength properties (soil strength, fissuring, stress histories, in-situ stresses)
- soil stress-strain properties (stiffness, bulk volume changes due to straining).

Sources of ground loss are schematically shown on Figure 2.13b and are divided after Cording and Hansmire (1975) into three groups: face loss, shield loss, and post-shield loss.

Face losses result from movements of soil inward into the unsupported excavation. They
are 3-dimensional in nature and can form a significant component of the overall volume loss for open-faced shield tunnels in clay soils. The use of EPB or slurry machines, or the utilisation of compressed air for open-faced tunnels, tends to reduce these losses by maintaining a balancing pressure whilst the soil is being removed. Breasting plates may be used in open-faced shields to provide some support at the upper-half of the tunnel face.

Radial ground loss above and around the shield generally results from closure of the bead gap left by the cutting edge, and closure of voids left by over-cutting. Larger volumes may be realised for gross over-cutting (i.e. poor workmanship) or for blocky soil which on excavation may leave large voids around the tunnel extrados. The effect of steering the machine (vertically or around horizontal curves) may also result in a larger gap behind the shield (Ng and Lo, 1985).

Post-shield ground losses generally occur around a constructed lining which inadequately replaces the cross-sectional area of the shield. In practice, bolted segmental linings are installed within the tail-skin of the shield or TBM, and thus a small gap is usually left. If the lining is of an expanded type, or if grouting is performed immediately after lining construction, the void will be reduced or eliminated. Deformation of the lining after the voids have been filled or closed results from the transfer of overburden stresses to the new boundary.

Attewell et al. (1986) provide an apparently rigorous method for calculating the three volume loss components which are functions of the rate of soil movement into the excavation (determined from laboratory tests; see Attewell, 1978), the rate of tunnel advance, the length of the shield and tail, and the unsupported length of soil behind the shield; other factors are introduced to account for doming around the tunnel face. Such a rigorous method of calculating volume losses is of limited value because tunnelling success is inevitably dependent on the working practice/workmanship, and shield/TBM performance.

Because $V_i$ is impractical to measure at the tunnel in practice, it is more usual to measure the surface settlement trough per unit length ($V_s$) and relate this surface measurement (see Figure 2.13a) to the excess excavated volume at the tunnel by the following relationship

$$V_s = V_i \frac{\pi D^2}{4} \quad (2.14)$$

For saturated normally consolidated clays this relationship is likely to be a close approximation. However, there may be significant error when comparing volumes at the tunnel face and surface profile volumes for overconsolidated soils, sensitive collapsing clays, and granular materials where volume changes on unloading and shearing around the tunnel extrados may be significant.

For a 2-dimensional transverse Gaussian displacement profile the settlement trough volume is the area under the normal distribution. Equation 2.4 cannot be integrated directly, but
a numerical solution (see for example Attewell and Woodman, 1978) is given by

\[ V_\nu = \sqrt[2\pi]{i \ w_{\max}} = 2.5 \ i \ w_{\max} \]  

(2.15)

For predicting \( w_{\max} \), \( V_\nu \) is often quantified using simple empirical relationships and engineering judgement gained from tunnelling in similar conditions. Estimates can be made for clay soils from Figure 2.14 which shows (a) the range for numerous case history data and (b) various suggested relationships between stability ratio and volume loss. The theoretical relationships are derived for undrained conditions and generally assume isotropic 2-dimensional response (i.e. taking \( K_0 = 1 \) and ignoring the three-dimensional effects), no gravitational stress difference from crown to invert, and an elastic-perfectly plastic soil. Addenbrooke (1996) usefully points out that when using these figures and making some simplifying assumptions, volume losses of between 1% and 3% are estimated for tunnels with stability ratios of about 2.

Kimura and Mair (1981) introduced the concept of a load factor \((LF)\) to relate the stability of similar shallow tunnels under differing working conditions as follows:

\[ LF = \frac{N}{N_f} \]  

(2.16)

where \( N \) is the stability ratio and \( N_f \) is the stability factor at failure. The ratio is essentially the inverse of the factor of safety, \( F_s \) described earlier (Equation 2.3). Figure 2.15 shows the range of volume loss for varying load factors and demonstrates that for factors of safety greater than 1.5 (LF less than 0.66) the volume loss is likely to be less than 4%. Lake et al. (1992) emphasise that Figure 2.15 should be used with caution because of the idealised conditions which do not account for workmanship, and that the concept does not take account of the 3-dimensional effects of a tunnel heading.

Typical values of volume loss recorded for open-faced shield tunnels constructed in London Clay range between 1.0-1.4% (New and O'Reilly, 1982). Values from about 0.7 to 1.6% were recorded above the 6.15m diameter running tunnels for the Heathrow Express (Barakat, 1996). New and Bowers (1994), and Deane and Bassett (1995) quote volume losses between 1 and 1.3% for SCL tunnels in London Clay at the Heathrow trial tunnel (some 25km west of central London). Similar values are estimated from the data above the JLE SCL trial tunnel at Redcross Way near London Bridge (Kimmance and Allen, 1996) for 5.3m and 11.3m diameter SCL tunnels, and for JLE SCL tunnels near Waterloo Station (Harris, 1996).

2.4.6 Trough width parameter \( i_y \) and trough length parameter \( i_x \)

With the transverse profile shape assumed to be of Gaussian form, the parameter \( i_y \) is needed which prescribes the distribution of displacements about the centre-line. An estimate of \( i_y \) from field measurements can be made by plotting settlement data as the natural logarithm of settlement normalised by the centre-line settlement \((\ln(w_y/w_0), w_0 = w_{\max} \text{ for the final construction profile})\) against the offset squared \( (y^2)\); the slope of the best-fit line to the data is equal to \(-1/2i_y^2\)
as shown on Figure 2.16. Alternatively, one may simply choose \( i_j \) as the point of maximum slope (or the point of inflection) or the point in the settlement profile where \( w_r = 0.61w_{ma} \). As field data tends to be not exactly Gaussian, these different definitions often yield different estimates of the trough width parameter; this is investigated in Appendix H for the field measurements presented in Chapter 6.

From field data an empirical relationship was proposed by Schmidt (1969) as

\[
\frac{2i_j}{D} = k\left(\frac{z_0}{D}\right)^n
\]

(2.17)

where \( z_0 \) is the depth to the tunnel axis, \( D \) is the tunnel diameter, and \( k \) and \( n \) are constants that depend on the material in which the tunnel is constructed. Peck (1969) asserted that \( n \) is usually between 0.8 and 1 for clays, but that \( n \) decreases with increasing material stiffness. As a result, smaller values of \( i_j \) are expected for tunnels in stiffer clays. \( k \) is often taken to be equal to 1 for stiff clays.

From case history data, O'Reilly and New (1982) proposed a simpler empirical equation relating the tunnel depth, \( z_0 \) and the trough width parameter for field measurements above UK tunnels in clay as

\[
i_j = 0.43z_0 + 1.1
\]

(2.18)

For simplicity it is often assumed that

\[
i_j = Kz_0
\]

(2.19)

with the value of \( K \) being dependent on the geology of the soil above and around the constructed tunnel. Values of \( K \) for surface profiles range between 0.4 to 0.7 for stiff to soft silty clays, and between 0.2 to 0.3 for tunnels in granular soils. For stratified geologies, O'Reilly and New (1982) proposed that \( i_j \) at the surface be represented by:

\[
i_j = \sum_{n=1}^{n} K_n T_n
\]

(2.20)

where \( K_n \) is the \( K \) value for the \( n \)th soil layer, and \( T_n \) is the thickness of the \( n \)th soil layer. Centrifuge model tests reported by Grant and Taylor (1996) on two-layered geometries (sand above clay) show that \( K=0.3 \) is reasonable for sands overlying clays at the surface, but that \( K \) values in the clay stratum near and above the tunnels vary significantly with depth (see Section 2.5.2).

Kimura and Mair (1981) reported \( K \) values of about 0.5 for centrifuge tests performed in overconsolidated kaolin. They also demonstrated that the shape of the trough at the surface is generally independent of the degree of support within the tunnel and hence independent of the construction techniques used.

Mair and Taylor (1997) presented case history data for trough shapes related to tunnel depth for different ground conditions; the data are reproduced in Figure 2.17. The authors
conclude that for practical estimation purposes the transverse distance to the point of inflection for tunnels in clay can be assumed to be equal to half the depth to the tunnel, or $K$ equal to 0.5; this result is also obtained from Equation 2.17 for $n$ and $K$ close to unity.

Cording and Hansmire (1975) presented a method of estimating the overall trough width using an ‘angle of draw’ ($\beta$) emerging from the spring line of a tunnel. The value of $\beta$ (measured from vertical) for tunnels in clay lies between 33° and 50°, with the larger values seen above tunnels both in soft clays and in London Clay where displacements were small and elastic. Peck (1969) suggested from limited data that the overall width is approximately equal to five times the trough width parameter, $i_\tau$. Rankin (1988) showed that the initial trough width is nearer six times the trough width parameter, $i_\tau$, or about three times the tunnel depth. The increase from Peck’s (1969) estimation of $5i_\tau$ may be attributable to improved measurement techniques and possibly to more case history data for deeper tunnels in clay soils.

The trough length parameter, $i_n$, was shown by Hurrell and Attewell (1985) to be slightly larger than $i_\tau$ for a number of case histories (example data presented in Figure 2.7). For practical estimation purposes they suggested that $i_n$ could conservatively be assumed equal to $i_\tau$. The data of the current research does not support their findings as discussed in Chapter 7.

2.4.7 Time-dependent displacements

The distribution and magnitude of time-dependent displacements depend on the equilibrium pore pressure profile evident after tunnel construction, the depth and diameter of the tunnel, the tunnel lining performance (permeable or impermeable), permeability of the soil (including anisotropy), the amount of soil disturbance caused by construction, and the flexibility of the lining. Generally, the settlement trough width increases after construction in clayey soils because consolidation of clay occurs over a much wider zone than ground loss into a tunnel excavation. However, very few measurements of this have been made above tunnels in London Clay.

Schmidt (1989) presented a methodology to estimate pore pressures around tunnels to evaluate the potential for consolidation settlement. Based on simplified plasticity solutions discussed in Section 2.6, he suggested that for most normally consolidated clays at least some consolidation settlement would be expected from construction alone, but for overconsolidated soils pore pressure changes due to construction are likely to be negative. Ward and Pender (1981) confirmed through field measurements (Ward and Thomas, 1965; Palmer and Belshaw, 1980) that most tunnels in clay soils act as drains in the long-term. Seepage into a tunnel will increase effective stresses in the surrounding ground and induce consolidation; only small amounts of flow from a low permeability clay are necessary.

For tunnels at different depths in the same clay, the time-dependent displacements above a shallow tunnel are likely to be more rapid than for deeper tunnels. The rate of consolidation in clays will be faster because of the shorter drainage paths and the permeability may be higher than at depth because of clay fissuring in overconsolidated clays, weathering, and lower mean stresses at shallow depths (Vaughan, 1989).
Observed forms of consolidation settlements in clays

Shirlaw and Copsey (1987) presented data above two open-faced tunnels driven in normally consolidated marine clay under compressed air. They observed 100mm of total settlement after 100 days, 50mm of which could be attributed to consolidation. The observed construction settlements were reasonably Gaussian in distribution but the consolidation settlements were considerably wider. They also noted that the rate of settlement development was unaffected by the release of the air pressure some time after construction.

O'Reilly et al. (1991) presented 10 years of measurements over a tunnel in soft silty clay at Grimsby, UK. They showed that total settlement magnitudes had increased to about twice the construction settlement and that the trough width parameter had increased by between 30% and 40%. The profile shapes were not exactly Gaussian in form, with larger displacements observed at offsets far from the tunnel axis.

Profiles for time-dependent settlement above tunnels in soft clays summarised by Shirlaw (1995) are given on Figure 2.18a. This plot shows normalised profiles of consolidation settlements against normalised offset from the tunnel centre-line. These data show narrower trough shapes for tunnels constructed using EPB or full-faced tunnel boring machines than for open-faced shields, but for all tunnels the settlement trough widens during consolidation. However, the maximum slopes for some of the long-term profiles are much higher than the Gaussian curve indicating that the potential for damage does not always reduce with time.

Bowers et al. (1996) presented long-term settlement measurements above the 8.7m diameter Heathrow Express trial tunnel in London Clay (New and Bowers, 1994) as reproduced in Figure 2.18b. Three years after construction the centre-line settlement had increased by between 16 and 42%, with the smallest increase observed above the tunnel length with the largest construction displacements. The profiles of displacement all widened with time, reflected by a 22% increase in the inflection offset, $i_c$. Displacements extend much further than the construction settlements and the profiles generally appear less Gaussian in shape. Transverse horizontal strains measured at the same site showed little significant change from the values measured for construction, implying that differential horizontal displacements are small. Bowers et al. (1996) note that total horizontal stress cells and piezometers positioned above and adjacent to the tunnel registered only small changes since tunnel completion.

Lake et al. (1992) summarised the case history data for initial and longer-term changes in $i_c$ above tunnels in clay soils; these results are reproduced as Figure 2.19. The ratios of final to initial trough width parameters generally fall between 1 and 1.5. Lake et al. note that most of the ‘final’ trough width monitoring were usually performed within a year of construction, and therefore 1.5 likely represents a lower-bound for the long-term case.

Prediction of time-dependent displacements

Yeates (1985) contended that the form of consolidation settlement could range from a near-Gaussian distribution of similar width to that seen during construction, to a profile with nearly similar displacements for all offsets. The latter of the two profiles indicates a much greater
horizontal permeability than vertical while the former may suggest radial drainage into the
tunnel, or concentrated consolidation around the tunnel perimeter as a result of clay remoulding
(Cording, 1991). As a starting point, the magnitudes of long-term movement in clay soils can
be crudely estimated from simplified calculations utilising consolidation theory. It is noted that
the construction induced changes may influence significantly the longer-term response and so
analytical calculations of this sort should be used in combination with judgement and
empiricism. A general predictive tool for time-dependent ground response is impractical as
consolidation behaviour, and particularly its magnitude, will be very site specific.

Glossop and O'Reilly (1982) indicated that consolidation profiles may be approximated
as a Gaussian distribution. From UK data, they defined an empirical relationship for the surface
trough volume as a percentage of the tunnel volume as

\[ V_s = 1.14 + 1.33(\text{OFS}) \]  
(2.21)

where \( \text{OFS} \) is the ratio of the initial effective stress to the undrained shear strength. Howland
(1980) proposed an empirical method for predicting the maximum long-term displacement based
on data from soft clays given as

\[ w_{\text{max}} = 2N A w_{\text{max}}, A = 0.39(1 - 0.01 w_{\text{max}}) \]  
(2.22)

where \( N \) is the stability ratio (Equation 2.1). \( w_{\text{max}} \) the maximum construction settlement, and \( A \)
is an empirical parameter related to the maximum construction settlement. It can be shown that
for values of \( w_{\text{max}} \) ranging between 10 and 20mm and a stability ratio of 2.5, the long-term
settlement is estimated to be 1.5 to 2 times the initial maximum settlement. Attewell (1988)
defined a long-term trough width parameter, \( i_{\text{LY}} \), as

\[ i_{\text{LY}} = \frac{i_y}{\left(-\frac{d^2}{2}ight)} \]  
(2.23)

Selby and Attewell (1989), noting that many natural soils have anisotropic permeabilities which
arise from bedding features, proposed that the ratio of horizontal to vertical permeability \( (k_h/k_v) \)
could be related to the long term trough width parameter, \( i_{\text{LY}} \), by

\[ i_{\text{LY}} = \frac{2}{3} \frac{k_h}{k_v} \]  
(2.24)

According to Equation 2.24. a soil with isotropic permeability yields a trough width parameter
of two-thirds of the tunnel depth. \( z_0 \). This might be considered a minimum value and is
consistent with the data trends on Figure 2.19.

Bowers et al. (1996) proposed a empirical logarithmic equation to fit their long-term
data (Figure 2.18b) but note that the relationship would only be expected to apply to tunnels

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of similar size, lining type, geometry, and in similar ground.

### 2.4.8 Multiple tunnels

Current practice when utilising the empirical approach for the assessment of multiple tunnels is to superimpose the effects (both surface and subsurface) of construction by treating each tunnel as a separate and unique event. In doing so, any influence a construction event might have on the ground response for later construction will be ignored. For tunnels in stiff clay there are very few observations to support or contradict the superposition of tunnelling events. Bartlett and Bubbers (1970) presented settlement data above two London Underground Victoria Line tunnels constructed in London Clay, which are given in Figure 2.20. Their data showed that the settlement profile for a second tunnel was shifted towards the first; the maximum settlement due to construction was observed nearly 5m from the centre-line towards the first tunnel. Addenbrooke (1996) demonstrated through FEM using soil models which account for recent stress history (i.e. construction of the first tunnel) that asymmetry and eccentricity of the settlement profile above a second tunnel might be anticipated. Figure 2.21a shows surface profiles derived from this work which indicate a shift of the maximum settlement towards the previously constructed tunnel; the spacing between the tunnels is a key aspect of the degree of influence which may be expected as illustrated on Figure 2.21b. It was also demonstrated by Addenbrooke (1996) and Addenbrooke et al. (1997) that a second modelled tunnel gives larger volume losses than for the same tunnel modelled for greenfield conditions at the same degree of unloading.

### 2.5 Empirical methods for predicting subsurface displacements

#### 2.5.1 Idealised framework for movements around a tunnel

Mair and Taylor (1992) proposed an approximate method for predicting the ground response around a tunnel heading by making use of two simplified and idealised models as shown in Figure 2.22: a collapsing spherical cavity at the tunnel face, and a partially unloaded and infinitely long cylindrical cavity in axisymmetric conditions. For an isotropic initial stress state and a linear-elastic perfectly-plastic soil, plasticity solutions can be derived for each of the simplified models. For a fully unloaded spherical cavity in undrained conditions the displacement pattern radially inward is given by

$$\frac{\delta}{R} = \frac{s_u}{3G} \left(\frac{R}{r}\right)^2 \exp(0.75N^* - 1), \ N^* = \frac{P}{s_u}$$

(2.25)

where

- $\delta$ = the inward radial displacement normalised by the tunnel radius.
- $s_u$ = undrained shear strength
- $G$ = shear modulus
For deep open-faced tunnels, \( p = \sigma \), can be assumed; in this case \( N^* \) corresponds to the stability ratio given by Broms and Bennermark (1967) with \( \sigma_r = 0 \). For this idealised model, a linear relationship between \( \delta/R \) versus \( (R/r)^2 \) would be expected as shown in Figure 2.23a. Soil movements observed in front of an advancing tunnel in London Clay presented by Ward (1969) are shown on Figure 2.24a and are re-plotted in this manner on Figure 2.24b; note the different symbol for radius (\( a = R \) in Equation 2.25) on Figure 2.24b. The measurements show a nearly linear trend. For movements close to tunnel face and for deep tunnels, the simplified model can provide a useful frame of reference for assessing the soil movements and for comparing tunnels in similar ground.

For the idealised cylindrical cavity, it was argued by Mair and Taylor that the presence of the lining means that the cylinder is not fully unloaded. In this case the deformations are given by

\[
\frac{\delta}{R} = \frac{Su}{2G} \left( \frac{R}{r} \right) \exp(N^*(1-n)-1), \quad N^* = \frac{p}{Su}
\]

(2.26)

where \( n \) represents the proportion of the initial mean total stress imposed by the ground on the lining immediately after construction. Field measurements have shown that the lining begins to carry about 30% of the total overburden (\( \sigma_r \)) stress only 2-3 tunnel diameters behind the advancing face (see Section 2.7). A linear relationship between \( \delta/R \) and \( (R/r) \) results for this idealised model as shown in Figure 2.23b.

Measurements of vertical displacements (\( w \)) on the centre-line of tunnels in London Clay, and horizontal displacements measured at tunnel axis level (\( v \)) presented by Mair and Taylor (1992) are plotted on Figures 2.25a and b in normalised form against \( R/(z_0-z) \) or \( R/y \). Both data sets show similar and approximately linear trends, but with different \( y \)-intercepts; this reflects the different far-field or boundary conditions for each direction considered.

The data on Figure 2.25a imply that for this idealisation the far field value of normalised vertical displacement (i.e. \( w \) for \( R/(z_0-z) \) near 0) is the same for all tunnels, and therefore the surface settlement for deep tunnels is dependent only on the tunnel radius. On Figure 2.25b the data imply that horizontal movements beyond a radius of 5R (\( R/r \) less than 0.2) are negligible.

### 2.5.2 Subsurface vertical displacement profiles

Attefell and Farmer (1974) and Barratt and Tyler (1975) presented some limited subsurface data for open-faced shield tunnels constructed in London Clay. Mair (1979) derived extensive data from model tests on lightly overconsolidated kaolin. Glossop (1978) measured subsurface profiles above tunnels in soft clay. New and Bowers (1994) obtained extensive data for subsurface settlements above a tunnel constructed in London Clay using sprayed concrete lining (SCL). Each of these sources report transverse displacement troughs narrowing with depth...
below ground level or with nearness to the tunnel crown.

Mair et al. (1993) summarised model tests and case history data for subsurface transverse vertical displacement profiles in clays as shown on Figure 2.26, where the normalised inflection offsets ($i_z/z_0$) are plotted against the normalised depth below ground level, $z/z_0$. These data clearly show a decrease in the trough width parameter with depth below ground level which can be described by the equation

$$i_z = 0.175 + 0.325 \left(1 - \frac{z}{z_0}\right)$$  \hspace{1cm} (2.27)

This is shown on Figure 2.26. Combining Equation 2.27 with the approximate volume of a Gaussian profile given by Equation 2.15 and the relationship between $V_1$ and $V$, given in Equation 2.14 yields the expression

$$\frac{w_{\text{max}}}{R} = \frac{1.25V_1}{0.175 + 0.325 \left(1 - \frac{z}{z_0}\right) z_0}$$  \hspace{1cm} (2.28)

where $w_{\text{max}}$ is the maximum settlement observed at depth $z$, below ground level. Figure 2.27 shows the normalised vertical displacements presented in Figure 2.25a and the calculated variations from Equation 2.28 for different ratios of radius to tunnel depth ($R/z_0$) appropriate to the tunnel depths at Regent’s Park and Green Park: some agreement is obtained between the two. The empirical forms of Equation 2.28 are non-linear and depend on the tunnel depth. Mair et al. (1993) state that closer agreement between the field data and empirical variations of settlement with depth would be realised if the dilatant behaviour of the clay was accounted for, as volume losses at depths closer to a tunnel may be larger than those seen at or nearer the surface.

New and Bowers (1994) fitted data from the Heathrow Express trial tunnel by dividing the total ground loss into point sinks evenly distributed over a horizontal plane at the tunnel invert, and assuming $i_0$ remained constant with depth. Total displacements were then determined from the superposition of calculated movements for each point sink. The model was shown to fit well with subsurface data collected above their large-diameter tunnel in London Clay, but has not been compared with other available data.

Heath and West (1996), using centre-line displacements measured above Docklands Light Railway tunnels in London, proposed a binomial variation of trough width given in Equation 2.29.

$$\frac{i_z}{i_0} = \left(1 - \frac{z}{z_0}\right)^{1/2}$$  \hspace{1cm} (2.29)

$i_0$ is the trough width parameter at the surface and $i_z$ is the same parameter at depth $z$. When
compared with Equation 2.27 (Mair et al., 1993) for the same depth and \( i_o \). Equation 2.29 predicts narrower troughs close to the tunnel crown (i.e. \( z/z_0 > 0.8 \), see Figure 2.26).

Longitudinal or parallel profiles at subsurface horizons are seldom presented in literature and there are no proposed frameworks for the potential changes in trough length parameter with depth below ground level. Hurrell and Attewell (1985) suggested that assuming \( i = i_o \) at the surface was likely to be conservative, and therefore the values of the trough width parameter estimated at different depths are usually taken for the trough length parameter as well.

### 2.5.3 Horizontal displacements

The assumption of undrained response during construction in clays also implies that the volumetric strains remain zero. Therefore, in plane strain conditions the vertical strain, \( dw/dz \), equals the horizontal strain, \( dy/dy \). By assuming that the subsurface profiles are Gaussian in form and that the trough width parameter variations presented in Section 2.5.2 are valid, the necessary distributions of horizontal displacements to satisfy the no-volume-change requirement can be determined.

Using Equation 2.27, it has been shown by Taylor (1995) that the vectors of displacement at any subsurface horizon are simply aimed at a point approximately \( 0.175z/0.325 \) below the tunnel axis level at the centre-line for the constant volume assumption. For Equation 2.29 the depth of the point sink varies with depth of the horizon considered and is described by the function \( 2(z_0 - z) \) (below the horizon of consideration). New and Bowers (1994) show clearly from their observations above the Heathrow Express trial tunnel that the point sink is not at a constant depth but instead varies with proximity to the tunnel crown. Appendix G discusses horizontal movements in more detail and relates numerical studies and available field measurements for tunnels in London Clay to the empirical framework.

### 2.6 Empirical methods for predicting pore pressure and total stress changes

The general pattern of response for pore pressures near an advancing tunnel shield have been observed by many authors (see Barratt and Tyler, 1975; Palmer and Belshaw, 1980; Ward and Pender, 1982; New and Bowers, 1994) and can be considered in three phases. The first is characterised by a small increase in pore pressures at some distance in front of the tunnel shield. This response is notionally an undrained reaction of the ground as the arching mechanism (i.e. doming) develops around the tunnel face.

The second phase is marked by a substantial drop in pore pressures around the shield in response to unloading at the tunnel boundary. In overconsolidated clays, additional negative pore pressures might be expected from swelling, dilatation while yielding.

The third phase is marked by a rebound of pore pressures as the erected lining begins to support overburden stresses and the soil is reloaded. Continued time-dependent changes are seen thereafter, as the disturbed ground around the extrados equalises with the new tunnel boundary and the far field stresses and pore pressures. This period is a battle between swelling...
of unloaded soil (or soil which has undergone dilatant shearing) and consolidation which may result from drainage into the tunnel.

The total stress changes generally show a reduction in radial stresses (largest at the tunnel boundary) and an increase in the circumferential stress as the soil load is redistributed around the tunnel excavation.

Stresses and pore pressure changes during construction may be analysed by assuming an idealised axisymmetric collapsing cylinder in a linear-elastic perfectly-plastic soil initially under isotropic stresses (see for example Mair and Taylor, 1992; Schmidt, 1989; Samarasekera and Eisenstein, 1992). The patterns of radial total stress, circumferential total stress and pore pressures are shown schematically on Figure 2.28 for this ideal case assuming plane-strain. The total stresses for elastic response are given by

$$
\sigma_r = p - s_u \left( \frac{R}{r} \right)^2 \exp \left( \frac{p[1-n]}{s_u} - 1 \right)
$$

(2.30)

$$
\sigma_\theta = p + s_u \left( \frac{R}{r} \right)^2 \exp \left( \frac{p[1-n]}{s_u} - 1 \right)
$$

(2.31)

where \(p\) is the initial mean total stress, \(s_u\) is the undrained shear strength, \(R\) is the tunnel radius, \(r\) is the offset from the tunnel centre, and \(n\) is the percentage of the initial mean total stress carried by the tunnel lining immediately after erection (i.e. \(1-n\) represents the percentage of unloading at the tunnel boundary). The changes in the total stresses are equal and opposite to satisfy the undrained assumption.

If the stress changes around the tunnel are large during the unloading, a plastic zone may develop around the tunnel extrados. By assuming a simple Tresca criterion for plastic yield (\(\sigma_\theta - \sigma_r = 2s_u\)) a plastic radius \(R_p\) may be estimated as

$$
R_p = R \exp \left( \frac{p[1-n] - s_u}{2s_u} \right)
$$

(2.32)

This is shown on Figure 2.28. At all positions within the plastic radius both the yield criterion and the undrained response assumption must be satisfied. Within the plastic zone the radial and tangential total stresses are defined by

$$
\sigma_r = np + 2s_u \ln \left( \frac{R}{r} \right)
$$

(2.33)

$$
\sigma_\theta = \sigma_r + 2s_u
$$

Excess pore pressures in the plastic zone can be estimated using an approach described by
where $a'$ is Henkel's (1960) pore pressure parameter determined from triaxial tests. In the elastic zone the pore pressures can be estimated using $a'$ applied to the calculated total stress change from Equations 2.30 and 2.31 above as

$$
\Delta u = a' \cdot \sqrt{\frac{R}{r}} \cdot s_u \left[ \frac{R}{r} \right]^2 \cdot \exp\left( \frac{P[1-n]}{s_u} - 1 \right)
$$

Mair and Taylor assumed $a'=0$ (isotropic elastic material) which gives no pore pressure changes in the elastic zone. They suggest incorporating a variation in stiffness increasing with distance from the excavation (notionally equivalent to a reducing stiffness with increased strain level) to predict excess pore pressures which occur in the elastic zone. Their formulation results in a smaller calculated plastic radius ($R_{pl}$) given by

$$
R_{pl} = R \exp\left( \frac{N}{2} - 1 \right)
$$

The pore pressures in the elastic zone are described by the relation

$$
\frac{\Delta u}{S_u} = -\frac{R_{pl}}{r}
$$

Immediately after tunnel construction, pore pressure equalisation will begin, with pore water moving into the plastic zone under the hydraulic gradient created during tunnel construction. In this instance, the permeability will be much higher than suggested by most laboratory tests because of the larger gradients. Also because fissures may have opened locally during unloading, and any zones of strain localisation may tend to be preferred paths for pore water flow.

On Figure 2.29 the idealisation is shown for drained conditions. Through the plastic zone, the effective stresses are shown as constant with offset. However, if a reasonable pore pressure profile on equalisation is assumed for the plastic zone (shown on the figure), the effective stresses reduce from their initial values throughout the plastic zone. Thus, any departure from the undrained assumption during construction may result in a reduction in shear strength locally near the tunnel.

An interesting set of total horizontal stress and pore pressure measurements were obtained by New and Bowers (1994) adjacent to the Heathrow Express trial tunnel in London Clay. The tunnel was lined with sprayed concrete and constructed using three different excavation sequences with monitoring performed above each.
The piezometric responses around the tunnel followed the expected pattern of response as the tunnel progressed past the instruments. Pressure drops during construction of between 50 and 90% of the initial levels were recorded, with largest changes seen closest to the tunnel extrados. In one case a piezometer about 3m above the tunnel crown registered an apparent zero, probably indicating that the soil likely went into suction (negative pore pressures) for a short period.

The observed total stress and piezometric measurements at the tunnel axis are reproduced in Figure 2.30 for the type 2 excavation sequence. Several observations are worth noting:

- Small increases in piezometric pressure are observed in front of the advancing tunnel face, but with small decreases in total horizontal (or radial) stress.
- The initiation of total stress and pore pressure reduction is simultaneous with the start of horizontal ground displacements towards the excavation.
- Large changes in total horizontal stress (up to 150% of the estimated total overburden stress) and smaller changes in pore pressures are observed near the left heading extrados, reducing with increasing distance from the tunnel edge. These equate to large net changes in the horizontal effective stress.
- Pore pressure changes and displacements are observed to continue on a similar trend for several days after the construction of the left heading, but total horizontal stress measurements are arrested almost immediately after the heading face is past the instrument line.
- Only a small additional change in both pore pressure and total horizontal stress are observed during construction of the right heading away from the instrumentation.
- Pore pressures and total stresses show some rebound towards their initial stress levels with time on tunnel completion.

The pattern is generally as expected. Interestingly, the instruments which are positioned on the left hand side of the tunnel show limited response to the construction of the right hand drift which means that the presence of the left hand drift shelters the soil left of the tunnel. This observation might imply a benefit in using staged excavations with SCL by reducing the disturbance to adjacent subsurface structures.

2.7 Tunnel performance

Circular tunnels built in clays generally deform so that the vertical diameter decreases and the horizontal diameter increases. This squatting deformation is observed for tunnels in overconsolidated London Clay (see, for example, Ward and Thomas, 1965), inferring that the mode of long-term tunnel deformation is apparently unrelated to the in-situ stress conditions prior to tunnelling. Squatting is likely to continue for several years after tunnel construction, corresponding to increases in the load carried by the lining. Ward and Thomas showed diametric changes in a large-diameter tunnels which were still measurable after 6 years.
Field measurements of lining loads in stiff clays have shown that about 30% of the overburden stresses are observed immediately after tunnel construction (Barratt and Tyler, 1976; Ward and Thomas, 1965; Thomas, 1976) and are accompanied by high rates of vertical shortening and horizontal elongation (see Ward and Thomas, 1965). After 1 year, between 40 and 60% of the overburden pressures are taken up in the lining thrust, while the rates of diametric change have reduced significantly.

For relatively flexible linings the hoop stresses are expected to be similar in magnitude around the ring while in more rigid linings significant variations within the ring may be expected (Eden and Bozozuk, 1969). Observations on cast iron segmentally lined tunnels in London Clay have shown circumferential thrusts equal to the entire overburden pressure over a longer time period (Peck, 1969).

Barratt et al. (1994) presented long-term measurements of hoop forces made in the northbound Jubilee Line tunnel beneath Regent’s Park in London Clay; these are reproduced in Figure 2.31. The tunnel was driven with an open-faced shield and lined with segmental concrete rings. The vertical loads measured at the spring lines for the Regent’s Park tunnel show about 60% of the overburden pressure after 19.5 years while the horizontal loads are at about 40% of the overburden load. The load build-up seems to have stabilised, showing only a 3% increase during the past 15 years of monitoring. The data implies that the large horizontal ground stresses which, in overconsolidated soils, are ‘locked’ in situ are dissipated by displacing and disturbing the clay during construction. As a summary, the authors state that the available evidence suggests that ring loads on single circular tunnels in stiff overconsolidated clays are unlikely to exceed the in-situ all-around pressure prior to excavation.

2.8 Potential damage to structures

The predictions of ground response are used to assess the potential for damage to structures above and near tunnels, and often a staged approach is utilised to determine quickly which structures are likely to suffer unacceptable deformations.

The first stage in the process looks at both the maximum slope or rotation of a structure and the maximum settlement. No account is made for building distortion, but instead empirical limits are established for different structure types (Rankin, 1988). The eliminating criteria for the JLE assessment process were set as a maximum rotation of 1:500 and a settlement of less than 10mm.

The second stage of assessment assumes the building acts as a deep beam - as proposed by Burland and Wroth (1974) - to assess the potential distortion within the structure. This distortion encompasses horizontal extension arising from differential horizontal movement, and both shear and bending deformation which arise from the differential vertical movements, slopes and curvatures in the displacement profile. A critical parameter for deformation is the deflection ratio, $\Delta/L$, which is shown schematically on Figure 2.32.

The building is assumed to move with the predicted greenfield ground movements at foundation level; the average horizontal strain, shear strain and bending strain for different
Partitioned building lengths are estimated. The maximum combined strains are determined and compared with critical tensile strain limits (identified from laboratory work and field monitoring on masonry structures) to estimate potential damage (see Burland, 1995). For this second analysis stage the suggested tensile strain limits for different categories of damage are given as follows:

<table>
<thead>
<tr>
<th>Category</th>
<th>Tensile strain range</th>
<th>Damage classification</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0-0.05%</td>
<td>Negligible</td>
</tr>
<tr>
<td>2</td>
<td>0.05-0.075%</td>
<td>Very Slight</td>
</tr>
<tr>
<td>3</td>
<td>0.075-0.15%</td>
<td>Slight</td>
</tr>
<tr>
<td>4</td>
<td>0.15%-0.3%</td>
<td>Moderate</td>
</tr>
<tr>
<td>5</td>
<td>&gt;0.3%</td>
<td>Severe</td>
</tr>
</tbody>
</table>

The third stage analysis is performed for structures which have been assessed and appear liable to suffer more than slight damage, and is aimed at specific features of the project works or the structures of interest. Matters considered in greater detail may include the tunnelling methods employed, information on the structure (i.e. foundation types, structure type, history of repair, etc.), soil-structure interaction, raking of the structure, previous ground disturbance and previous building movements. Precise calculation for many of these aspects is not possible, and in most instances engineering judgement is necessary.

In relation to the empirical framework for predicting ground movements, some general comments can be made regarding the choice of the various parameters when predicting the ground response and the predicted damage potential in a stage two type of assessment. Table 2.2 shows qualitatively the changes in the maximum potential damage criterion of a generic structure for different parameters if the value chosen for prediction is larger than the measured value (all other parameters being correct).

Generally, overestimation of the trough width ($i_1$) or trough length ($i_2$) parameters will underpredict both the maximum slope, and deflection ratios in both hogging and sagging. A smaller maximum strain would be calculated for a smaller deflection ratio which yields a potentially unconservative damage assessment. On the other hand, overestimation of the volume loss (or the maximum settlement) yields an overprediction of the potential damage. Overestimating the depth of focus for the transverse displacement vectors underpredicts the horizontal displacement magnitudes and horizontal strains which would result in an underprediction of the maximum tensile strain. Of course the significance of error for each parameter on the damage prediction varies considerably and depends largely on the relative positions of the structure to the tunnelling works.
2.9 Summary

In this chapter the empirical and analytical methods for estimating ground response are reviewed and some data from relevant case histories are presented and discussed. Irrespective of the method of analysis used, reliable estimates of the ground response due to tunnelling require experience of tunnelling in similar ground, along with specific details of the geology, project geometry, and tunnelling methods to be employed. Understanding the stress changes around the tunnel and the modes of soil response is critical to making advances in numerical analyses of tunnelling. Some notable gaps in the empirical framework are identified: the subsurface ground response (particularly the horizontal displacements); the response near to and in advance of a tunnel heading; the interaction effects of multiple tunnels; and the time-dependent ground response after construction above single and multiple tunnels in London Clay.
3 Project site investigation

3.1 Geology of the London Basin

London is contained within a basin which forms part of a large synclinal structure extending from Wiltshire in the west out into the North Sea; a simplified plan and section of the basin is given on Figure 3.1. The northern boundary is formed by the chalk of the Chiltern Hills which run northeast-southwest parallel to the axis of the syncline and dip gently under London to reappear as the North Downs and the southern boundary. The Chalk was formed in a warm shallow sea environment and is believed to have been folded during the Alpine orogeny at the end of the Cretaceous period (Mortimer and Pomerol, 1997) which resulted in the southwest-trending axis. Some major faulting is apparent in the basin (for example in Wimbledon and Greenwich, see section on Figure 3.1) which is likely to have left a stepped topography within the Chalk.

Following a period of erosion, sediments were deposited in this chalk basin; the strata chart a complex sequence of changing depositional environments beginning at the end of the Cretaceous period (see Table 3.1). Of particular interest to this thesis are those deposits which range from the mid- to late-Tertiary to present day.

The area of focus in this chapter is St. James's Park between Westminster and Green Park Stations (see Figure 1.3); the approximate position of the instrumented section can be seen on Figure 3.2. A number of investigation holes were bored near the site and are shown in relation to St. James's Park on Figure 3.2. Boreholes 109/109P were bored less than 50m from the instrumented section on the Westminster side. Self-boring pressuremeter tests were carried out in borehole 109P. The local stratigraphy derived from these JLE boreholes is given on Figure 3.3a. The sequence comprises approximately 0.2m topsoil above 1-2m sandy man-made fill. Beneath this Made Ground lies a succession of sandy alluvium (3-4m), coarse to fine flint gravel (Terrace Gravel, 3-4m) and very stiff brown to grey clay (London Clay) to depths in excess of 40m below ground level; deeper still lies the Lambeth Group (formerly the Woolwich and Reading Beds), the dense Thanet Sands, and the relatively permeable water-bearing Chalk.

Figures 3.3b and c show the soil profiles observed by Burland and Hancock (1977) and Burland and Kalra (1986) in the Westminster area nearer to the river Thames but within 500m of the instrumented section. Figure 3.3d shows a borehole profile derived from qualitative appraisals of cuttings from the clay cutter tool, and from a single U4 sample taken near the westbound tunnel crown (see Appendix A). The overall profiles are all similar except for identified variations within the London Clay, which may be largely attributable to different descriptive styles.

The Site investigation (SI) work performed by JLE (see Linney and Page, 1996; JLE Interpretative Report, vols.1-4) provided relevant properties and geotechnical design parameters. The parameters proposed for the length of tunnelling between Westminster and Green Park are summarised in Tables 3.2 to 3.4. Details pertaining to these parameters are discussed for each
stratum below for the SI work and for other relevant soil investigations in and near London.

### 3.2 Geotechnical characteristics of the site

#### 3.2.1 Made Ground or fill

Made ground at the surface occurs over most of the route except for St. James's Park. Boring at this site found only occasional brick and rubble fragments in the top metre.

#### 3.2.2 Alluvium

These recent deposits originate from the end of glaciation (within the last 10000 years) and are probably associated with glacial run-off carried locally by the River Thames or other river(s) that existed previously in the area. The deposits are predominantly sandy, but can comprise layers and channels of soft, compressible and highly variable clays, silts, sands, gravels and organics. They are found widely across the London Basin varying laterally and vertically in composition.

The relevant geotechnical parameters for the alluvium recommended from the SI are summarised in Table 3.2. The gradings were highly variable, ranging from a clayey silt to a sandy gravel. At St. James’s Park the deposit is predominantly a sandy silt and covers the whole instrumented site. The material state was estimated as loose to medium dense from a small number of standard penetration tests (SPT). The bulk unit weight determined from a limited number of samples lay in the range of 12.5 kN m\(^{-3}\) to 20 kN m\(^{-3}\), with the lower values probably reflecting samples with high organic content.

#### 3.2.3 Terrace Gravels

The Terrace Gravels are found extensively around the London Basin and are generally described as 'terraced' deposits having been left at different levels: they were formed as part of an ancient flood plain in response to seasonal snow-melt run-off during cold climatic periods of the Pleistocene. As a result, the unit is generally a well-graded mixture of sand and gravel with occasional silt and silty sand layers: the grading varies both laterally and vertically reflecting the energy of the changing fluvial environment during deposition. Geotechnical parameters recommended for the Terrace Gravels are given in Table 3.3.

Gradings showed a significant variation in particle size distribution across the project ranging from well-graded sand to sandy gravel. Between Westminster and Green Park stations the deposits were predominantly described as orange brown, very sandy (medium to coarse) sub-angular to sub-rounded, well graded (fine to coarse), flint gravel with occasional cobbles. The material state was estimated from SPT 'N' values as medium dense to dense. All the geotechnical parameters for the Gravels were derived from SPTs and empirical correlations from 'N' values, or from in-situ testing (e.g. falling head test for permeability).

#### 3.2.4 London Clay

The London Clay is a marine clay of Eocene and Palaeocene age which was deposited in a
relatively quiet offshore environment. Erosion during the later Tertiary and Pleistocene periods (beginning about 65 million years ago) removed many of the overlying deposits, which are estimated to have been about 200m thick (from the base of the London Clay) in the London area (Skempton and Henkel, 1957), leaving most of the unit in an overconsolidated state.

The JLE SI interpretative report divides the clay into three primary vertical zones: weathered London Clay, unweathered London Clay and the Basal Beds. Further divisions within the clay have been made based on visible and grading evidence (Burnett and Fookes, 1974) but are not considered here.

The weathered layer was only sometimes observed at the London Clay - Terrace Gravel boundary during the site investigation and tended to be less than 2m thick. Generally it is described as stiff, brown and very closely fissured with some red iron staining along fissures.

Unweathered London Clay is typically a very stiff, thinly laminated, very close to closely fissured, grey-brown clay of low to medium compressibility and high plasticity. It is moderately to heavily overconsolidated and often fissured. Pockets and partings of sand and silt; thin and moderately strong layers of claystone; calcareous nodules; and pyrite nodules are all relatively common at tunnelling elevations.

The Basal Beds comprise a very silty and fine sandy clay zone near the base of the London Clay. When compared with the unweathered clay, this zone is usually less plastic (attributable to the higher silt and sand content) and less fissured, but has a greater frequency of silt and fine sand partings and pockets with bioturbation. The zone is up to 10m thick and lies between 2m and 4m above the top of the Lambeth Group.

Claystones were encountered in 75% of all boreholes sunk and typically occurred as bands up to 0.3m thick of fresh to slightly weathered grey and brownish grey claystone. A single band could seldom be correlated between two adjacent boreholes, indicating a horizontal persistence less than 50 to 100m.

The recommended geotechnical parameters for London Clay are summarised in Table 3.4. The gradings for unweathered clay range from 40% to 70% clay content along with some silt and fine sand. Some coarser gradings (less than 15% clay and up to 50% sand) were correlated with the deeper Basal Beds.

Moisture contents and Atterberg limit profiles with depth determined from the JLE SI are presented on Figure 3.4 along with the ranges observed for the CrossRail investigation performed about 3 kilometres north of St. James's Park, and the data from the Heathrow Express trial tunnel about 25 kilometres to the west. The plastic limits are seen to be fairly consistent with depth but the liquid limits show a slight reduction in values nearer the stratum base which reflect the increasing silt content of the London Clay at depth. Moisture contents also reduce slightly with depth which may be related to under-drainage of the clay (see Section 3.2). When compared with the CrossRail and Heathrow data the trends are seen to be very similar.

In terms of plasticity, the data ranges from intermediate to very high plasticity clay. The deeper and more silty samples tend to be in the intermediate plasticity range while the unweathered London Clay data are typically in the high to very high plasticity range.

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Undrained shear strength

The undrained strengths of the London Clay were measured using several different laboratory tests which included standard quick unconsolidated undrained (UU) triaxial tests on 38 and 100mm samples and consolidated undrained (CU) triaxial testing with local axial strain measurements. Strengths were also estimated using empirical correlations for SPT ‘N’ values. **Figure 3.5** shows the approximate range of undrained shear strengths with depth for the Westminster area determined from UU tests from the JLE SI, and the strengths reported by Burland and Hancock (1977) at the House of Commons car park.

The strengths show a general increase with depth, but show wide scatter that often arises from stiff clay testing as a result of natural fissuring and sampling effects (Hight and Jardine, 1993). It is noticeable that the strength is nearly constant in the clay layer from -25m to -35m (through which the westbound tunnel was constructed). Burland and Hancock identified this zone to be a layer with numerous sand partings, and suggested that the low strengths probably arise from water within the partings being drawn into the clay on sampling. Variations in $s_u$ along the route of the tunnels from Westminster to Green Park at this level are investigated in Appendix A and indicate that the undrained strengths are greater on average north of the St. James’s Park lake than in the Westminster Area.

All methods of measurement showed occasional measured shear strengths greater than 300kPa at depths greater than about 25m below the top of the London Clay ($z=-35m$). A similar transition was identified at other London Clay sites (Hight and Jardine, 1993; Burland and Kalra, 1986) which may reflect a progression to the less fissured and more silty Basal Beds.

The JLE data show good agreement with the range observed in the CrossRail geotechnical interpretative report for Bond Street Station and superimposed on **Figure 3.5**. The CrossRail results also comprise SPT correlations, pressuremeter tests, and UU triaxial tests. However, the strengths measured at the Heathrow Express (UU tests on 100mm samples) tend to lie at the low end of the JLE range. This difference probably rests in the use of 38mm samples for the JLE SI, which can give less representative values for the bulk strength because of the absence of fabric (i.e. fissures and joints). Additionally, the liquid limit variations with depth at Heathrow are consistently at the higher end of the JLE data (see **Figure 3.4**). This implies a more silty clay in central London which can show higher strengths.

Effective stress strength parameters

Consolidated undrained multistage tests were carried out by the JLE on unweathered samples (both 38 and 100mm) to establish these parameters. The data (not presented) showed a scatter which fell between the lines $c'=0$kPa, $\phi'=14.5^\circ$ and $c'=100$kPa, $\phi'=30^\circ$. These lines represent the lower limiting ‘fissure strength’ and an upper limit peak strength. Hight and Jardine (1993), in reviewing test data on London Clay in central London, suggested a fissure strength of $c'=0$kPa, $\phi'=19^\circ$ applicable for depths to 24m below the top of the unit, noting that lower values were likely to be for ‘unfavourably polished surfaces, possibly caused by sampling’.

**Figure 3.6** shows the Hight and Jardine data grouped into three depth categories below...
the top of the London Clay (8m-12m, 16m-24m, and 27m-40m). It can be seen that the upper limit changes significantly with depth. They concluded that at shallow to moderate depths the fissure spacing may be sufficiently close that standard soil samples are satisfactorily representative of the bulk soil fabric, but at greater depths both the peak and fissure strengths have to be considered to obtain an appropriate bulk soil strength.

**Undrained stiffness**

Stiffness values were determined from UU compression tests, pressuremeter testing and locally instrumented undrained triaxial tests (both in tension and compression). Details of the testing procedures can be found in the JLE geotechnical interpretative reports. The UU tests yield an undrained secant stiffness, \( E_{u50} \), determined at 50\% of the peak deviator stress. The pressuremeter tests measure the horizontal undrained stiffness, \( E_{uh} \), on small unload-reload loops. The locally instrumented triaxial tests yield a variation of secant stiffness, \( E_{a} \), at small strains, the values of which are usually much greater than those estimated from the UU tests.

The profiles of \( E_{u50} \) and \( E_{uh} \) with depth are given on Figure 3.7a along with the range of measured \( E_{u} \) (at 0.01\% axial strain) from the locally instrumented triaxial tests. The values of \( E_{u50} \) are generally much less than the calculated \( E_{uh} \) from the pressuremeter tests. The \( E_{u0.01} \) range overlaps the high end of the pressuremeter results. Burland and Kalra's (1986) empirical relationships for undrained stiffness with depth in London Clay are superimposed on Figure 3.7a. These give values which are much less than the \( E_{uh} \) and \( E_{u0.01} \) ranges, but show some agreement with the \( E_{u50} \) values obtained from quick UU tests.

Small-strain variations of \( E_{a} \) normalised by \( s_{a} \) are given on Figure 3.7b for axial strains between 0.01\% and 10\%: a wide range is seen, particularly at strains below 0.1\%. This probably reflects a combination of the variability and quality of samples obtained in a stiff overconsolidated material, and the presence of fabric or discontinuities which would influence the strength and stiffness in varying amounts (Hight and Jardine, 1993). A typical result for a sample from central London published by Mair (1993) and given on Figure 3.7b falls near the centre of the observed range of the JLE data.

**Coefficient of earth pressure at rest**

In-situ earth pressures were estimated from suction measurements on specimens obtained from thin-walled pushed samples (see for example Chandler and Guiterrez, 1986), self-boring pressuremeter tests, and measurements of mean effective stresses in triaxial test specimens. Testing details can be found in the JLE geotechnical interpretative report. Values of \( K_{0} (=\sigma_{u}/\sigma_{v}) \) for all test results presented on Figure 3.8 range between 0.75 and 2.3 near the top of the London Clay to between 1 and 2 at -35m below ground level. However, the mean of these data is comparable with the profiles with depth concluded by other authors (for example Wroth and Hughes, 1973; Hight and Higgins, 1995) and is similar to estimates based on Burland, Simpson and St. John (1979) as shown. The CrossRail data superimposed on Figure 3.8 also show good agreement with the mean result of the JLE SI.
Consolidation behaviour

Oedometer tests were performed on thin-walled pushed tube samples and on standard U100 samples. Details of testing can be found in the JLE geotechnical interpretative report. The samples were tested over a range of 100 to 400 kPa stress range. The coefficient of compressibility, \( m_l \), was found to lie between 0.03 and 0.15 m\(^2\)/MN which compares favourably with the values of 0.05 to 0.1 m\(^2\)/MN for stiff unweathered London Clay given by Tomlinson (1980) over a similar stress range. The coefficient of consolidation, \( c_s \), was determined to vary from 0.2 to 0.8 m\(^2\)/year, with occasional values as high as 5 m\(^2\)/year at very low stress levels.

Permeability

The mass permeability of the unit is potentially controlled by the discontinuities and the presence of silt and sand partings, but the intact clay is relatively impermeable. The predominantly horizontal fabric gives rise to anisotropic permeability with the vertical being less than the horizontal, which will influence the dissipation of excess pore pressures during tunnel construction and the patterns of long-term settlement.

Permeabilities (as coefficients of hydraulic conductivity) were determined during the JLE SI from rising head tests carried out in piezometers installed within boreholes along the route and often at tunnelling level. The results given on Figure 3.9 lie in the range of \( 1 \times 10^{-9} \) to \( 3 \times 10^{-8} \) m/s, decreasing with depth.

Also on Figure 3.9 are the permeabilities reported by Burland and Hancock (1977) based on falling-head tests carried out about 400 m east of St. James’s Park at the New Palace Yard car park, data from Chandler et al. (1990) from piezometers in London Clay in Essex, and the range of permeabilities determined from rising- and falling-head tests performed between Hyde Park and Bond Street Station during the CrossRail SI. Each of these sources shows much lower measured permeabilities than the JLE SI. These differences may be the result of increased sand and silt parting frequency; many of the piezometers for the JLE were installed at depths which are noted to be considerably more silty or sandy. Increased permeability locally around piezometer holes may have also occurred due to stress relaxation arising from borehole installation and testing procedures (Chandler et al., 1990).

Geophysical borehole logging

Boreholes from which cores were obtained were also mapped using the following geophysical logging techniques: calliper, natural Gamma radiation, and Gamma-Gamma ray density (or Gamma back-scatter). Calliper logging measures the changes in the borehole width over the depth. A continuous log can give indications of the nature of the strata (i.e. relative coherence). Natural Gamma logging measures the naturally occurring gamma radiation in the ground around the borehole; clays with potassium (e.g. illite) show a strong response while more inert materials (e.g. quartz sand) show very little radiation; the response can be calibrated to give representative changes in the relative amounts of clay to sand. Gamma-Gamma ray density logging uses an active high-energy source which emits Gamma radiation into the ground. The receiver measures
the energy loss of the rays which reflect by collisions with other atomic particles; the more dense a material, the more rapid the energy loss. The relative positions of the energy source and receiver can be adjusted to vary the resolution (i.e. the thickness of the soil element being investigated).

Two boreholes were mapped using these techniques near St. James's: BH104TA about 600m to north; and BH115T on the embankment near the Thames and about 600m to the east. The results from BH115T for the tunnel depths between Westminster and Green Park are reproduced on Figure 3.10. It should be noted that elevations are in project datum which is +100m ordnance datum (newlyn). The log for BH104TA shows much more scatter in measured response for all instruments than seen in BH115T, and the results are relatively inconclusive.

At BH115T the calliper survey shows some undulations on the borehole wall in the upper few metres of London Clay which may reflect the weathered nature at this shallow depth. The trace becomes more smooth with only a few undulations to about 80mPD or 27.5mbgl. The borehole surface then becomes markedly more irregular to about 72mPD or 35.5mbgl, below which the log indicates a return to a smoother surface. In this roughened zone the core descriptions include references to silt and sand partings.

The change at 80mPD is paralleled roughly by a shift in the trend of bulk density with depth (determined from the Gamma backscatter), followed by a noticeable step (between -0.1 and -0.2g/cm³) at 77mPD which is recovered at about 73mPD (about 35mbgl). Above 80mPD and below 73mPD the density log shows an approximate linear increase in depth.

The Gamma backscatter log traces for both bed resolution density (BRD) and high resolution density (HRD) show an appreciable increase in response variability between 77 and 72mPD indicating a higher degree of heterogeneity in material density near to the borehole. The natural Gamma log shows what might be interpreted as a slight shift in the average response over the same depth range.

These geophysics results suggest the clay between 72mPD and 80 is a zone of more variable material, possibly reflecting the presence of silty sandy bands and/or silt and sand partings within the London Clay. The log descriptions provided by the drillers of this hole showed a slight increase in the observed frequency of partings and fissures over the same depth range, and an investigation of the cores summarised in Appendix A shows numerous silty bands and sand partings in the same interval. The depth range coincides very well with a 10m zone of clay observed by Burland and Hancock (1977) in New Palace Yard about 100m to the south and west of the borehole location which exhibited numerous closely spaced sand partings.

3.3 Hydrogeology of the site
Two aquifers exist in the London Basin: (1) a deep aquifer comprising the Thanet Sands, the Chalk and the Basal Sands (beneath the Chalk) topped by either the London Clay or the clays of the Lambeth Group (Water Resource Board, 1972): and (2) a perched water table in the Terrace Gravels on top of either the London Clay or the clays of the Lambeth Group. This upper aquifer is recharged from surface precipitation and locally from the Thames.
Abstraction from the deep aquifer in the early 1900s yielded significant drops in water pressures within the deep aquifer which under-drained the overlying clays. The water levels in 1967 were 60m below the observed levels in the 1850s when artesian wells were commonplace in many areas of outer London. In central London water pressures have risen about 48m between 1967 and 1997 (Thompson, 1997) and continue to rise by between 1m and 2m per year (Simpson et al., 1989).

During drilling for the JLE SI in 1990, the upper aquifer was observed at about 100mPD (-3mbgl at BH109, see Figure 3.2) in the Terrace Gravels for several boreholes between Green Park and Westminster Station. This aquifer is likely to be in direct communication with both the St. James’s Park lake and the river Thames.

In the London Clay, several borehole water strikes were recorded near the greenfield site: at 19mbgl immediately above a claystone band in borehole 106 (approximately 400m to the north of the instrumentation line); and at 24.5mbgl in borehole 109 (60m to the east of the site, see Figure 3.2) possibly associated with sand partings in the London Clay. No correlations of claystone layers or sandy partings were made between boreholes implying that lateral continuity of such layers or features may be limited. However, because of the borehole spacings larger water quantities and more continuous partings are not precluded from being present. JLE monitoring of the pore pressures in the clay gave somewhat scattered results. Nevertheless, the data generally showed a steady rise to about 100mPD after installation indicating a hydrostatic profile.

Figure 3.11 shows the observations from piezometers installed in London Clay at different depths at the St. James’s Park instrumented site (see Chapters 5 and 6). A near-hydrostatic piezometric pressure distribution is inferred from the top of the Terrace Gravels, but with some deviation at depth. The influence of underdrainage on the pore pressures in the London Clay is weakly apparent at the bottom of the data range. but pressures that are substantially below hydrostatic have been measured at deeper horizons (Simpson et al., 1989).
4 Instrumentation and monitoring details

This chapter describes the monitoring equipment used to characterise the ground response above the tunnels in St. James's Park; details of the instrumentation layout are given in Chapter 5. References are made in this chapter to monitoring periods which correspond to construction events at the instrumented site which are also described in Chapter 5. These periods are (chronologically) as follows: (1) prior to any tunnelling; (2) westbound tunnel construction; (3) rest period between construction events; (4) eastbound tunnel construction; and (5) long-term monitoring.

In parallel with the presentation of the results in Chapter 6, the discussion of instrumentation is by three categories reflecting different groupings of measured quantities:

- surface displacements
- subsurface displacements, and
- piezometric and total stress changes.

Under each category, the specific monitoring equipment used is described; the instrumentation and monitoring point installations are detailed; the specific survey features are discussed (i.e. how the measurements were obtained); and the precision and accuracies of the measured quantities are assessed. When addressing monitoring performance, specific errors and their relative magnitudes are described and the methods by which the errors are considered in the data processing are described. Evaluations of field performance through actual measurements are covered in Chapter 6.

4.1 Measurement uncertainty and sources of error

Uncertainty of measurement for site instrumentation has been considered in detail by Dunnicliff (1988), who defined eight groups of uncertainty as summarised below:

Conformance is a qualitative measure of the influence an instrument has on the desired quantity being investigated. Instrumentation that has little influence is said to have excellent conformance. It is a highly desirable attribute for high accuracy, but varies significantly between instrument types measuring the same quantity.

Accuracy is the degree of correctness of a measurement, or the nearness of a measurement to the true quantity. The accuracy of a measurement depends on the accuracy of each component of the monitoring system. It is generally evaluated during instrument calibration, where a known value and the measured value are compared. Accuracy is expressed as ±X units, meaning that the measurement is within X units of the true value.

Precision is the reproducibility and repeatability of measurement, or the nearness of each of a
number of measurements to the arithmetic mean. Precision is also expressed as ±X units, meaning that the measurement is within X millimetres of the mean measured value and is often assessed statistically with a degree of confidence associated with the statistical distribution; the number of significant digits reflects a higher precision.

*Resolution* is the smallest division readable or measurable on the instrument. Interpolation by eye between divisions generally does not improve the resolution, as the estimation is subjective and operator dependent.

*Sensitivity* refers to the amount of output response that an instrument produces when an input quantity is applied. Thus higher sensitivity means a stronger output for the same measured quantity, but it does not imply greater accuracy, precision or resolution.

*Linearity* is a measure of the proportionality of an instrument’s response to the measured quantity. Instruments for which the response is directly proportional are described as linear. The measure of linearity is the full-scale percentage divergence between indicated and actual response if a linear relationship is assumed.

*Hysteresis* is used to describe response which depends on whether the measurement is increasing or decreasing. Generally, instruments with large hysteresis are unsuitable for measuring rapidly changing quantities.

*Noise* describes the random measurement variation caused by external factors. Excessive noise results in lack of precision and accuracy, and may conceal small real changes in the measured parameter.

Combinations of the above measurement uncertainties manifest themselves as measurement error, which is the deviation between the measured quantity and the true value (and is mathematically equal to accuracy). Errors are classified as follows:

*Systematic errors* result from improper calibration, changes in calibration, hysteresis and non-linearity, but are generally errors whose magnitude and sign can be determined or estimated. Where appropriate, corrections may be applied to measured quantities to improve the accuracy.

*Gross errors* are caused by carelessness and inexperience. They include – but are not limited to – misreading, booking errors, misnumbering of monitoring points, use of non-standard monitoring techniques, and incorrect use of instrumentation. All measurements are suspect until gross mistakes are identified and eliminated. Measurements should be checked immediately (i.e. by repeating the measurement) to allow correction during the survey. This, however, is not always possible because of the transient nature of tunnel monitoring. Prior to analysis a
thorough review of all data should be performed to identify and correct gross errors.

**Conformance errors** are the result of poor installation procedures and poor selection of instrumentation and/or survey layout, but are minimised by careful supervision of instrument installation.

**Environmental errors** arise from the influence of heat, moisture, vibration, pressure, atmospheric conditions, lighting, etc. Errors arising from temperature changes, for example, can be quantified and corrected for some instruments, but generally environmental errors are only addressed in a qualitative manner.

**Observational errors** result from different observers using different monitoring techniques. The use of well-designed automatic monitoring systems can potentially minimise observational error, as can the use of standard recording forms and marking flags for instrument locations.

After identifying and correcting errors where possible, a variation in the readings will still exist as a result of random error, comprising the influence of the errors and uncertainties given above; this variation is representative of the precision of measurement. A summary of the monitoring systems and their estimated accuracies is given in Table 4.1.

### 4.2 Surface monitoring points

Measurements of surface displacements were made on surface monitoring points (SMPs) shown schematically on Figure 4.1. Each SMP comprises an extended socket embedded into a 1.5m deep concrete pillar (approximately 100mm in diameter) sleeved with stiff PVC tubing over the top 0.8m; the tubing was used to reduce the effect of near surface changes (i.e. temperature, seasonal movements). The holes were bored using a portable petrol-driven auger.

The socket is an elongated version of the standard BRE socket (BRE Digest 386, 1993) which facilitates repeatable positioning of a monitoring tool (e.g. a levelling plug) into the socket. The tool is pulled into place by a loose fit thread and its radial position is controlled by a precisely machined spigot. As the tool is tightened into the socket, mating machined faces are pulled together (see Figure 4.1) so that it is accurately located to nearly the same position each time. The quoted positioning precision for a standard socket and levelling plug is about 0.1mm using good monitoring practice.

A lockable cover was placed over each SMP and a thin ring of self-adhesive permeable foam (affixed around the top 0.1m of PVC) was used to prevent direct contact between the cover and the concrete pillar when the covers themselves were concreted into position. The foam also facilitates drainage of rain water which occasionally accumulates within the covers.

### 4.3 3-dimensional displacements measured using total station surveying

Total station surveying is a simple concept but uses a very complex instrument and requires a
significant amount of data processing. Its use is advantageous as target positions in three
dimensions relative to a local coordinate system are obtainable from a single set of observations
which can be carried out quickly. The monitoring relies heavily on the principles of
triangulation and trilateration using angle and distance measurements relative to reference targets
unaffected by construction activities.

Measurements at St. James’s Park were made using two high-precision coaxial electronic
theodolites with integrated electronic distance measurement (EDM) and digital data acquisition
capabilities (Leica model nos. TC1610 and TC2002) – often referred to as total stations. The
TC1610 was used for Period 1 and Period 2 monitoring, during which time all measurements
were recorded manually. The TC2002 was used for all subsequent monitoring: a schematic
figure of the instrument is given on Figure 4.2a.

Reference points affixed to structures outside the zone of influence of JLE construction
activities were established to provide a frame of reference for the angle and distance
measurements; these were assumed to be fixed during all site monitoring. Each point comprises
a retro-reflective prismatic target, affixed to newly cleaned surfaces (ideally smooth and flat) on
a building facade with silicone sealant. Details of the layout are given in Appendix B.

Measurements to the SMPs were made using two demountable retro-reflective prismatic
targets made at Imperial College as shown in Figure 4.3 which screw into the extended BRE
sockets. The thread system of the BRE monitoring plug described in Section 4.2 and shown
in Figure 4.1 was incorporated in the design. The top of each target rotates freely around the
vertical axis via a precision bearing (as indicated in the inset figure) enabling the prism
orientation to be adjusted to face the total station squarely. Target no.1 was used at St. James’s
Park during monitoring periods 1, 2 and 3, but was replaced with the larger and more robust
target no.2 near the end of Period 3 because the bearings in target no.1 were becoming loose.
Surveys were performed with both targets simultaneously to facilitate continuity of data analysis
after changing to target no.2.

Surveys were begun by carefully centring and levelling a total station over one of two
designated station locations (ST1 and ST2 marked by nails fixed into pathways, see Figure 5.2)
beyond either end of the line of SMPs orthogonal to the tunnel axes. At the second station
location, a tripod-mounted reflector was centred over the nail. This reflector is a ‘cube-corner
prism’ with cross-hairs for measuring horizontal and vertical angles: for distance measurement,
the prism returns transmitted waves from the EDM back precisely in the same direction as they
are received.

With the total station and the prism levelled, a sight was made to one of the reference
points and the horizontal angle set to zero. Measurements of vertical and horizontal angles and
inclined distances with both instrument faces were then made on this and all other visible
reference targets, on the tripod-mounted prism, and on the demountable target at each SMP.
Changing instrument faces involved rotating the optics 180° about both the standing axis and
the tilting axis (see Figure 4.2a). After surveying all SMPs, the measurements on the reference
targets were repeated. The instrument and prism were then removed from their tripods and
centred and re-levelled at the opposite station location. The survey was then repeated from the second monitoring location.

To speed up operations the surveys were often performed by measuring with both instrument faces for the reference points and the tripod-mounted prism, and only with a single face for the SMPs. Surveys performed at night excluded some measurements to distant reference points as they could not be seen clearly.

### 4.3.1 Total station measurements: uncertainty and sources of error

The TC2002 resolves to 0.1 second of arc for horizontal and vertical angles, and to 0.1mm for distances. The TC1610 resolves to a 1 arc second for angles and 1mm for distances. However, the accuracies quoted by the manufacturer are both about 1 arc second and 1mm.

A review of the collected data revealed some gross errors in point numbering and measurement booking; these were corrected where possible or the data omitted in analyses. Environmental errors such as those arising from gusting winds causing instrument vibration and ambient temperature giving rise to heat haze from warm ground are uncorrectable for individual sights, although the closing error may quantify to some extent any overall error resulting from vibration/wind.

Variations in atmospheric temperature and relative humidity affect the infra-red waves used to measure distances. The associated error may be estimated from Figure 4.4 (provided by the instrument manufacturer, Leica), which shows the atmospheric correction (in parts per million, ppm) as a function of ambient temperature for different relative humidities. The range in which the majority of monitoring was performed is also identified, indicating that a potential error up to +1ppm (in distance) could be realised for most surveys; for the extreme cases of very hot and humid weather the potential error on distance measurement could reach +2 to +3ppm. For the short distances measured - less than 150m for all points - the atmospheric corrections are almost certainly less than 0.5mm.

Within the instrument, the relative arrangement of the two axes of rotation and the line-of-sight may result in systematic errors in angle measurements. Ideally the line-of-sight should be exactly perpendicular to the tilting axis, and the tilting axis exactly perpendicular to the standing axis as indicated on Figure 4.2b. Deviations from the perpendiculars are manifested as horizontal collimation errors (or line-of-sight errors) and vertical collimation errors (tilting-axis errors) respectively. The line-of-sight error is, in most instances, quite small. It can be determined by measuring the horizontal angle reading on a clearly marked point (as close to the horizontal as possible), and then re-measuring the horizontal angle on the opposite face (i.e. rotating the instrument 180° about both the standing and tilting axes). Tilting-axis error is best determined by measuring the vertical angle reading on a point near a zenith angle of 45° or 135°, and then re-measuring the angle on the opposite face. In both cases, the difference in measurement for the two instrument faces represents the tilting-axis error.

By repeating measurements on both instrument faces, the two collimation errors can be compensated for by taking the average of the measured angles for each target; this also helps
to identify gross errors, and can reduce random errors in the measurements and provides a better statistical average. When the horizontal angles are calculated (as the difference between two horizontal angle readings) the line-of-sight error is minimised. The errors may also be affected by large ambient temperature fluctuations (e.g. direct sunlight) which can put the instrument out of level.

The demountable targets have a random error associated with returning the target to the same position; this is likely to be similar to the 0.1mm error given for the BRE sockets but may be accentuated in the horizontal directions for the longer target pedestal.

Differences (termed closing errors) at the beginning and end of the survey in measurements to the reference points were generally less than 5 arc seconds for both vertical and horizontal angles and less than 1mm for distances. These errors are considered to be representative of the general accuracy of measurement for the entire survey, as the source of the closing error and its occurrence within the survey is not easily determined. This angular error equates to approximately 0.5mm/20m distance perpendicular to the line of sight. This value agrees well with the variation in three-dimensional coordinates of between ±1mm and ±2mm shown from actual field measurements in Chapter 6.

4.3.2 Processing of the total station measurements

Obtaining the best results from theodolite measurements requires a considerable amount of processing. In this thesis, the angle and distance measurements collected were analysed using the well-established statistical concept of least squares adjustment. The principle uses redundant measurements taken during each survey to adjust the measured quantities to best satisfy geometric criteria (e.g. triangulation and trilateration) whilst minimising the adjustments made to the actual measurements.

The program used for analysis was written by Mr. S.K. Sharma (Department of Civil Engineering, Imperial College). Relative weightings were assumed for measured angles (1") and distances (1/50000) for calculation of the standard error of analysis (i.e. the overall value of the adjustments) to be minimised; these weightings reflect the precision and accuracy for each measurement. However, changing the relative weightings to 2" and 1/25000 resulted in only small differences in the calculated coordinates (<0.5mm), demonstrating that the analyses were not very sensitive to relative weighting for the survey layout. All analyses were run with a convergence value of 0.05% on adjustments, meaning that the solution was achieved if the coordinate adjustments for the last iteration were all less than 0.5mm.

Prior to processing the measurements on the SMPs, it was necessary to establish a local coordinate system and determine the position of all reference points relative to this local system using triangulation and trilateration. The positions were calculated for a number of surveys performed prior to construction activity beneath either site to establish the initial coordinates for each reference point. These were re-calculated for surveys performed later in the construction sequence to verify that the reference targets were stable relative to each other over longer time periods. The local coordinate system at St. James's Park is described in Appendix B.
With the reference system established, the measured data were then prepared for input into the program. Collimation errors were addressed by averaging the measured angles and distances for both faces. Measurements made to SMPs with only one instrument face were corrected by the average collimation correction for the survey determined from the control points and other SMPs for which both faces were observed. Known elevations from precision levelling could also be included in the program to give added vertical control in the analysis, but analyses performed using this additional information produced negligible changes in the calculated coordinates and therefore the levelling data were not included.

After the analyses were performed, calculated coordinates relative to the local system were transformed to the $x$, $y$ and $z$ coordinate system given in Figure 2.8 relative to the tunnel axes for presentational purposes.

4.4 Vertical displacements measured using precision levelling

Vertical displacements were measured on a monitoring plug screwed tightly into the extended BRE socket at each SMP as shown on Figure 4.1. Levels were also measured on the subsurface instrumentation reference heads using a separate levelling plug for correlation with deep extensometers (see Section 4.7).

Levelling was carried out using an electronic Wild NA3003 precise level with digital data collection capabilities, and a 2m invar bar-coded staff. The principle of measurement centres on the image processing of the coded measurement signal obtained from the staff as seen through the telescope. A microprocessor in the level calculates the staff reading from the received signal.

The level runs consisted of four change points and covered approximately 150m; the positions of the change points and of the instrument positions were maintained constant between surveys to minimise potential sources of error. The levelling run at St. James's Park started and closed on a local surface nail positioned north-east of the site near Horse Guards Road. This pin was replaced in Period 5 with a pin fixed into the concrete immediately adjacent to the Foreign Office across Horse Guards Road.

4.4.1 Levelling: uncertainty and sources of error

The NA3003 level resolves to 0.01mm, but digitally records to 0.1mm, perhaps reflecting a more realistic accuracy for a single measurement. The manufacturer quotes a standard deviation for measurement of 0.4mm for a 1km run using the NA3003 and an invar staff. The levelling plug has a re-positioning accuracy of approximately 0.1mm.

The instrument itself has a systematic collimation error associated with the line-of-sight not being perfectly horizontal (i.e. perpendicular to the standing axis). This was assessed and corrected by performing a 2-peg collimation test periodically and adjusting the level accordingly. By keeping the survey layout constant and maintaining similar backsight and foresight lengths on each change point, the potential influence of any collimation error was minimised.

Precision levelling measurements are subject to potential random environmental errors.
resulting from gusting wind, direct sunlight and other atmospheric conditions. To minimise potential atmospheric influence on readings, the individual sight distances were no greater than 25m. Gross errors such as those arising from using an unlevelled instrument and SMP misnumbering are minimised by using an automatic level which numbers automatically and is inoperative when not horizontal. Errors incurred by not having a vertical staff are minimised with good monitoring practice. Change points were always taken on SMPs so that potential accidental inaccuracies arising from using change plates were eliminated.

For nearly all levelling runs performed at the site the survey closures rarely exceeded ±0.3mm (except during active tunnelling periods, when sinking ground occasionally resulted in larger closing errors). This value is considered representative of the general accuracy of levels.

Longer-term stability of the datum was evaluated using an extensometer anchor positioned below the tunnelling works in the London Clay (see Section 4.8).

4.4.2 Processing of levelling measurements

From measurements taken prior to any construction beneath the site, an average base level, \( L_0 \), was established (removing any data judged to be affected by gross errors) for each SMP and reference head. Vertical displacements relative to the local datum were then calculated from the measured level, \( L \).

\[
w = L - L_0
\]

(4.1)

For the second tunnelling event, a new base level was calculated from surveys performed during the few days prior to the tunnel passing under the site.

4.5 Transverse horizontal displacements measured using a micrometer stick

Relative horizontal displacements between adjacent settlement points transverse to the tunnel axes were measured using a 3m long micrometer stick similar to that described by Burland and Moore (1974). The device, shown in Figure 4.5, was manufactured by BRE and comprises a long aluminum bar of hollow square section with a permanently mounted micrometer at one end, a machined slot, and a series of five machined holes located near the opposite end. A thermometer is affixed at the centre of the beam beneath a protective clear acrylic window.

Two extended posts with enlarged ball seatings at the top (~270mm in length) were screwed into adjacent SMPs: one of the ball-seatings was located into the middle of the five machined holes and the other rested in the slot (see Figure 4.5). The threading arrangement for the posts is identical to the BRE monitoring plug described in Section 4.2. The ball seat and post pairs were numbered and used in a set pattern during measurement; the micrometer stick was always oriented in the same way. A micrometer mounted at the slotted end is screwed in until contact is made between the metal micrometer tip and the ball seating. To facilitate repeatable readings between instrument operators, an electrical circuit is set up through the beam.
so that when contact is made the circuit is closed and a light (LED) is illuminated. Sets of three readings on the micrometer are taken for each span by raising and relocating the slotted end of the beam between each measurement. The temperature is also recorded at each span.

### 4.5.1 Micrometer stick measurements: uncertainty and sources of error

The micrometer is graduated with 0.005mm divisions. The micrometer manufacturer quotes an accuracy of measurement of 0.005mm. Repeated readings at each span are usually within ±0.05mm which is representative of the precision for each set, but trials to determine the potential variation for removing and replacing the extended posts into the SMPs showed a repeatability between measurement sets of about ±0.2mm.

The accuracy of measurement is generally dependent on the care with which the micrometer plunger is brought into contact with the ball; by using an electrical circuit to indicate contact this potential error is minimised.

A potential gross error committed by using the wrong numbered posts is possible and would show differing measurements for all points across the site. Observational errors arising from misreading the micrometer and/or temperature, and poor tightening of the posts into the SMPs may also contribute to random errors.

The bar-mounted thermometer is graduated to 1°C. Measuring the correct bar temperature at each span is difficult at times when there are rapid changes in temperature (i.e. direct sunlight); in such instances the effects on the bar and the thermometer are unlikely to be similar. Generally, the micrometer stick was removed from its protective box and exposed to the ambient conditions for at least half an hour prior to taking measurements to allow it to equilibrate. Not allowing sufficient time for the aluminum bar to acclimatise could also lead to significant errors. Temperature controlled tests performed to determine the bar's temperature sensitivity are summarised in Appendix C.

### 4.5.2 Processing micrometer stick measurements

The measurement observed on the micrometer, \( R \), is an arbitrary measure of the distance between two adjacent SMPs, but is not a true measure of the actual span length. However, the interest lies in the displacements between the adjacent SMPs and therefore in the change of \( R \).

A temperature correction is applied to each measurement using the measured temperature on the bar, \( T \), and the coefficient of thermal expansion, \( CoT \) (see Appendix C), to give a corrected reading \( R_c \) as follows:

\[
R_c = R + CoT \times (T - 20°C)
\]  

(4.2)

The average of the corrected readings prior to tunnelling and for each span were established as the base readings, \( R_0 \).

Change in micrometer reading from the base reading reflects the combination of real
change in the span distance between the two SMPs ($\Delta S$), and errors in re-positioning the post
and ball-seatings described above.

\[ \Delta S = R_c - R_e \]  

(4.3)

For small changes in span the measured change divided by the span length, $S$, gives the average
horizontal strain as

\[ \varepsilon_h = \frac{\Delta S}{S} \]  

(4.4)

with the convention of positive for extension, negative for compression. The transverse
horizontal displacements for each point can be assessed by assuming that one end of the SMP
line is stationary and summing the measured span changes. The displacements at the $i^{th}$ SMP
in the line from the ‘fixed’ SMP are calculated by

\[ \nu_i = \Sigma (\Delta S_1 + \Delta S_2 + \ldots + \Delta S_i) \]  

(4.5)

Alternatively, a profile of horizontal displacement may be estimated from the average horizontal
strains across each pair of SMPs by curve-fitting. Figure 4.6 shows a transverse horizontal
strain (or ‘point’ strain) profile calculated for a hypothetical tunnel using the empirical
framework discussed in Section 2.4.2. Equation 2.7. Superimposed on the plot are the average
strains determined from the corresponding displacement profile (not shown) at 2.5m spacings
(i.e. the SMP spacing at St. James’s Park) plotted at the middle of each span. Very close
agreement between the average strain and the point strain is observed and it is concluded that
the average strains measured for 2.5m spans reasonably represent the horizontal ‘point’ strain
for middle of the span.

Thus, a polynomial curve may be fitted to the measured strains between adjacent SMP
points and then integrated to determine a second polynomial equation for the horizontal
displacement profile. The displacement profile is forced to diminish to displacements at
distances of about 1.5 times the tunnel depth, $z_0$, from the tunnel centre-line (corresponding
approximately to the half-trough width of $3t_h$ described in Section 2.4.6. Different curve forms
for fitting the strain data were investigated (e.g. polynomial, Gaussian, etc.) but the resulting
displacement profiles were fairly insensitive to the curve form used.

4.6 Longitudinal horizontal displacements measured using a collimation
tool

Horizontal displacements longitudinal or parallel to the tunnel axis were measured using a high
precision total station (described in Section 4.3 above) and a collimation tool designed and
manufactured at Imperial College. The collimation tool consists of a digital vernier calliper
(200mm range) mounted on a three-pinned bracket which fits into an extended post as shown on Figure 4.7. The pins and the post socket are machined to fit together, and the post screws into the SMPs as described for the BRE monitoring plug. Trimmed retro-reflective surveying targets are affixed to either side of the sliding arm of the callipers.

The surveys were performed by centring and levelling a total station at one of the two designated instrument positions (ST1 and ST2, see Section 4.3). Distance and angle measurements were then made onto the visible reference targets. For the second (eastbound) tunnel, a line of site was set by sighting to a reflective target affixed to the top of a metal stake which was driven into the ground at least 60m away from the eastbound tunnel centre-line. For the westbound tunnel, the line of sight was set by placing the collimation device into the post on the furthest monitoring point; the pin number was preselected (no.2) and the callipers were extended to a fixed length (100mm) on the digital display. The collimator tool was rotated on the post so that the calliper arm and target are perpendicular to the line of sight. This was checked by placing a cross-beam perpendicular to the vernier arm on the calliper immediately above the target and aligning it with the sight line as directed by the total station operator (see inset drawing on Figure 4.7). The horizontal circle of the total station was then locked with the line of sight positioned on the centre of the target at the end of the calliper arm.

The collimator operator then moves the device to the next SMP. The total station operator rotates vertically the line of sight to see the next survey point without adjusting the horizontal angle, and directs the collimator operator to open or close the vernier callipers and to rotate the collimator tool so that the arm is perpendicular to the line of sight. When the target at the end of the calliper arm is aligned with the vertical cross-hair viewed from the total station, the reading on the vernier’s digital display is taken; angles and distances to the target from the total station are also recorded where possible. The exercise is repeated for all points in the line followed by repeating the measurement of the starting point. The reference point measurements are then repeated to complete the survey. The collimation tool, with targets on either side of the calliper arm, has two instrument ‘faces’; the location of the SMPs relative to the sight line determined which ‘face’ was used for measurement as shown schematically on Figure 4.8.

Collimation surveys were performed primarily during the two tunnelling events (Periods 2 and 4) and from only one station location during each period. The survey was not practicable at night because good contrast between the optical cross-hairs and the target face was required to direct the collimator tool operator.

4.6.1 Collimation tool measurements: uncertainty and sources of error

The accuracy of measurement depends predominantly on the accuracy of the total station which has already been assessed in Section 4.3.1. The same total station instrument face was used for all surveys to minimise potential line-of-sight and tilting-axis errors. Errors associated with atmospheric changes between and during surveys also apply here.

The vernier has a resolution of 0.01mm and an accuracy of about the same at 20°C as given by the manufacturer. Temperature related changes of the calliper arm and of the post are
likely to be much less than the errors associated with the total station instrument.

Different collimation tool operators may result in poorer precision at larger distances from the instrument location. The choice of alignment may be biased depending on which side of the sight line the target position is being adjusted from; the optical cross-hairs appear thicker than the cross-hairs on the target itself. The potential error approximates to about ±0.5mm (i.e. half the thickness of the target cross-hairs).

4.6.2 Processing of collimation tool measurements

The position of the total station for each survey was calculated relative to the control points, as described in Section 4.3.2, to verify that the station location did not differ greatly between surveys. The measurements recorded from the vernier, R, represent a measure of the relative positions of the SMPs off the sight line nearly orthogonal to the tunnel, but include variations arising from differences in the station position and the sight line between surveys. The readings for the surveys prior to each tunnelling event were averaged to establish base readings, R0.

The direction or sign of observed changes in reading relative to the datum measurement depends upon the collimation tool face used (see Figure 4.8), and on the total station position (ST1 or ST2); displacements perpendicular to the sight line (and parallel to the tunnel axis) are calculated as follows:

\[
\begin{align*}
ST1: \quad u &= (R-R_0) \text{ face right} \\
ST2: \quad u &= -(R-R_0) \text{ face left}
\end{align*}
\]

The resulting sign of u indicates horizontal movements in accordance with the direction of the axes shown on Figure 2.8.

4.7 Subsurface displacement monitoring

Monitoring of subsurface displacements was performed using a variety of state-of-the-art instrumentation installed above and adjacent to the tunnels. BRE carried out all of the drilling and instrumentation installation at the sites. Boreholes were drilled using a percussive cable-tool rig and were typically 150mm (6") in diameter and cased over the top 10m through water-bearing gravels and into the top of the saturated London Clay; the remainder of the drilling was performed without casing. Summaries of the drilling progress and comments on instrument installation are given in Appendix E.

All boreholes were grouted after the installation procedures described below using a premixed cement-bentonite grout prepared in a mechanical mixing tank. Grout was introduced from the base using a tremie pipe to obtain an even ground distribution and to displace any water at the hole bottom. Samples were taken from several of the grout mixes for unconfined compression testing to check that the strength and stiffness of the grout were similar to the London Clay. These indicated that the strengths were reasonably comparable to the London Clay, and gave more scattered values of stiffness.
4.8 Subsurface vertical displacement measurements using rod extensometers

Rod extensometers with integrated reference heads were used at St. James's Park to measure vertical displacements at several subsurface horizons as shown on Figure 4.9. Each extensometer consists of an anchor with three extendable prongs attached to the end of a stainless steel rod sleeved with thin plastic tubing over the entire length. The sleeved rod extends from the desired anchor position beneath ground level to the ground surface. The reference head at the surface comprises a guiding bracket which maintains each rod in place beneath a plate with machined reference holes aligned above each rod tip. The reference head elevation is determined from precision levelling on to a specially machined stainless steel plug which fits into the guides of the reference holes as shown on Figure 4.9.

During installation each rod and anchor pair was assembled at the surface where they were laid out to the required lengths. All anchors for a single hole were then gathered together and lowered into the open borehole: a temporary clamping arrangement allowed the weight of rods to rest on the top of the borehole casing. Up to eight (8) extensometers were placed in a single 150mm (6") diameter borehole at different elevations. Once in position, the anchor prongs were extended into the surrounding soil using a hand-operated hydraulic pump. The prongs facilitate anchor contact with the surrounding soil. The hole was then filled with grout to seal the instruments in place.

Each set of measurements along the extensometer line was done using a dial gauge placed in each reference hole with the plunger resting on the rod tip (see Figure 4.10b). Every measurement was repeated three times with the dial gauge removed from the reference hole each time. Each survey started and finished with a measurement on a calibration socket to facilitate comparisons between different sets of measurements and different dial gauges. Repeating the calibration at the end checks that the dial gauge setting has not been altered during the survey. The calibration piece comprises a stainless steel socket with a hole approximately 100mm deep into which the gauge is inserted, the plunger rests on a pin fixed at the socket bottom (see Figure 4.10a).

In some cases (e.g. for deep anchors) the reference head moved vertically downward while the rod anchor remained stable. This resulted in the rod tip moving upwards relative to the reference head, and occasionally the gauge could not be positioned correctly into the reference hole. To facilitate readings, specially machined extension pieces were placed on the reference head slot and the dial gauge then placed into the extension piece as shown in Figure 4.10b.

The measurement gauge converts linear movement of the spring-loaded plunger to more visible dial rotations on the gauge face. The quantity measured is the difference between the dial position with the gauge on the rod, and its null position (i.e. zero plunger displacement) as shown in Figure 4.10b; a larger dial gauge reading equates to a larger movement of the plunger.
4.8.1 Rod extensometer: uncertainty and sources of error

The resolution of the dial gauge is 0.005mm and each set of three measurements are usually within 0.05mm. When using the extension pieces (see Section 4.8.2) the repeatability of the three measurements reduces considerably to between 0.1 and 0.2mm because the contact of the extension piece with the reference slot is less reliable. Random operational errors such as misreading the dial gauge, drift in the zero reading during the survey, and dirt at the rod tip/plunger contact or on the reference slot are possible and in some cases are correctable (for example, misreading the dial gauge reading as 29.95 as opposed to 39.95). Some of the reference holes were found to be poorly machined; this resulted in increased variability of measurement.

The levelling precision on the reference head is about ±0.3mm (see Section 4.4.1) and is representative of the overall accuracy of the calculated anchor displacements.

4.8.2 Processing rod extensometer measurements

Rod displacements relative to the local datum (see Section 4.4) are determined by considering changes in calibration readings, dial gauge readings and reference head levels. These are described in detail below.

Calibration variation

The calibration measurement for any given extensometer survey, \( C \), and a base calibration value, \( C_0 \), allow extensometer surveys to be compared regardless of the initial null position of the dial gauge or the type of dial gauge. \( C_0 \) is the average of a number of different surveys performed prior to tunnelling beneath the site. The selected sets for the base calibration value must have accompanying extensometer survey readings so that the base values for the calibrations and the dial gauge readings are derived from the data. The change in calibration from the base reading is given by:

\[
\Delta C = C - C_0
\]  

(4.7)

Changes in calibration between surveys generally result from changes in the null reading on the dial gauge, or from the use of a different dial gauge. Where a different calibration socket has been used, the difference in dial gauge measurement for the two sockets must be determined. The calibration measurement of the second socket can then be adjusted to relate measurements to the calibration of the first.

At St. James’s Park, two calibration sockets (identified as Red and Yellow) were used during monitoring. The calibration readings differ by 1.0mm, the yellow socket depth being more shallow than the red and thus displacing the dial gauge plunger more.

Change in dial gauge reading

Movement of the rod anchor relative to the extensometer reference head is evaluated by
considering the measured dial gauge reading, \( R \), a base value derived from the same survey measurements used for base calibration value, \( R_0 \), and the change in calibration for the set of readings, \( \Delta C \), by:

\[
\Delta R = (R - R_0) - \Delta C
\]  

(4.8)

The dial gauge measurements increase in magnitude as the gap between the reference head and the rod tip becomes smaller (i.e. a larger plunger displacement). Therefore a positive value of \( \Delta R \) indicates an upward displacement of the anchor relative to the extensometer head or, equally, a downward displacement of the reference head relative to the rod anchor.

**Displacements of the reference head.**

The vertical displacement of the rod relative to a local datum (see Section 4.4) outside of the zone of influence of tunnelling may be calculated by incorporating the change in level of the reference head at the same moment in time. The change in level should be calculated from a base level derived from measurements taken during a similar time span as the base gauge reading and calibration.

For a measured displacement of the reference head, \( w \), the anchor displacement relative to the local datum is determined by

\[
w_{\text{anchor}} = \Delta R + w
\]  

(4.9)

where \( w \) is positive for heave and negative for settlement.

**Other corrections**

Two extension pieces were used during monitoring at St. James Park:

- No 2. 13.25mm
- No.3 17.90mm

These lengths are added to the measured dial gauge displacement for comparison with readings performed without the extension pieces.

**4.9 Horizontal displacement measurements using electrolevel inclinometers**

Uniaxial electrolevels which rotate within a single plane were installed by BRE at discrete positions within standard inclinometer tube. The tube comprises 3m sections of plastic tubing with two pairs of mutually perpendicular grooves broached internally down the entire segment length; the sections were connected by self-aligning rigid couplings and a quick-setting water-
tight glue.

During tubing installation the sections were joined, glued and lowered into the boreholes length by length, making certain that the slots were aligned and orthogonal with the tunnel axes. Prior to electrolevel installation, and again after their removal, the tubes were mapped using an inclinometer torpedo as described in Appendix D.

The electrolevels are small glass vials which contain an electrolytic fluid and three electrodes which are partially immersed in the fluid as shown on Figure 4.Ila. The instruments are energised with a small electric current and the voltage of the arrangement is measured and converted to a digital reading. When the electrolevel is tilted, the length of each electrode immersed in the fluid changes. This causes a change in the resistance of the circuit and a change in the measured voltage for a constant current through the circuit. The voltage change is calibrated against tilt over a 2-3° range of rotation in controlled laboratory conditions prior to installation, yielding a near linear calibration factor for each electrolevel under constant temperature conditions.

The electrolevels vials were mounted on short carriages as shown on Figure 4.Ilb, each with its own independent cable lead and socket running to the surface. On installation the carriages are placed into the tubing grooves and carefully pushed to the desired depth; readings were then taken to verify that the initial positions were aligned near the middle of the response range (i.e. at the null position).

During Periods 2 and 4, when the tunnel shields were passing beneath the site, the electrolevels were connected to multiplexer units for automatic computer logging. The multiplexers energised each electrolevel in a set sequence every 15 minutes and the responses were automatically logged using a portable computer. Manual readings were taken during the rest periods between tunnelling events using a Flyde hand-held read-out unit which measured the voltage response after five seconds of energisation. It should be noted that the measured voltages are different for each method of monitoring because of the difference in the resistance of the measured circuits (extra cable lengths, multiplexers, etc.) but the changes in voltage output for a similar tilt are equivalent.

4.9.1 Electrolevel inclinometer measurements: uncertainty and sources of error

The conversion of measured voltage to digital response (bits) is set such that 100 bits equates roughly to 100 arc seconds of rotation or a tilt of about 0.5mm/m. The electrolevels were checked by BRE for response stability after energisation and were generally rejected if response did not settle to within 2 arc seconds of the correct value within 5 seconds of being energised.

The calibration factors provided by BRE for tilt-to-voltage response showed linearity of between ±2% and ±8% over the full linear range. The potential error in measurement for a severe rotation (say 2°) is about 500 arc seconds or nearly 2.5mm/m. The largest measured rotation during the monitoring at St. James's Park was about 1500 arc seconds which yields a maximum potential error of 120 arc seconds or just over 0.5mm tilt.

Electrolevels are generally influenced by temperature changes (Barakat, 1996), but in
the inclinometer holes temperatures are generally constant. However, when using the auto-
logging system it was evident that diurnal changes in ambient temperature resulted in variations
in response of up to ±4 arc seconds.

When using the manual read-out unit, the electrolevel was first connected to the unit. The unit was then turned on (which energised the electrolevel immediately) and then the 'hold' button was depressed by the operator; this allowed for 5 seconds to pass before the measurement was taken. The time between initial energisation and the measurement being taken was entirely operator dependent. From repeated manual readings on a number of electrolevels it was found that the repeatability between measurements averaged about ±4 arc seconds but could vary up to ±8 arc seconds.

Because each electrolevel measures tilt at one discrete point, significant errors may arise in processing the measured data if the displacement gradients change rapidly between adjacent measurement depths. This is highlighted in Section 4.9.2 below.

Only uniaxial electrolevels were used in the inclinometer holes, and it is recognised that components of displacement – other than that which is intended to be assessed – may also affect the measured tilt.

4.9.2 Processing electrolevel inclinometer measurements

The rotation, $\alpha$, for each electrolevel position is calculated from the measured response in bits ($R$), the base response established prior to active tunnelling beneath the site ($R_0$) and the calibration factor, $CF$ (radians/bit). by:

$$\alpha = (R - R_0) \times CF$$

(4.10)

Horizontal displacements for each inclinometer hole were determined by estimating appropriate gauge lengths for each electrolevel and assuming that the measured rotation is valid over the entire gauge length (i.e. the inclinometer tube is effectively assumed to be ‘hinged’ at the ends of each length). The gauge length limits were assumed to coincide with the mid-point between each adjacent electrolevel. Over each discrete section of tube the displacements were calculated. Each section is connected with the adjacent length as shown in Figure 4.12 to form a cumulative displacement curve over the entire tube length. The interpretation requires movements at either tube end to be known or assumed. For deep inclinometers below the tunnel axes, the bottom of the tube was assumed stable. For shallow inclinometers (i.e. above the tunnel crown) movements at the top of the tubings were estimated from the surface displacement measurements described in Sections 4.3, 4.5 and 4.6.

Alternatively, the discrete measurements of slope can be fitted with a smooth curve and integrated to determine a profile of horizontal displacement with depth. This method of analysis was considered, but did not prove successful for movements nearest the extrados of the tunnels as the slope changes between adjacent electrolevel carriages were large and resulted in unlikely profile fits. Processing of the data near the tunnel was facilitated by using the borehole mapping
results obtained after construction of both tunnels that are presented in Appendix D.

4.10 Piezometric and stress change monitoring

Pneumatic piezometers and push-in spade cells were installed at St. James's Park to monitor the pore pressure changes above the deep westbound tunnel and both the pore pressure changes and the total horizontal stress changes around the eastbound tunnel. Drilling for the piezometer holes was performed using the same percussion-drilling equipment and methods described at the beginning of Section 4.7; these boreholes were also mapped using the temporary tubing and the inclinometer torpedo and the results are included in Appendix D. The holes were completed to about 0.3m beyond the desired piezometer depth, and sand and water were then tremied together down the hole to achieve a 0.2-0.3m long sand pocket at the borehole base. The piezometers were lowered down to the base of the hole and then more sand and water were tremied around and above the piezometers. The final sand pocket lengths varied between 0.5-0.6m. Bentonite pellets mixed with generous amounts of water were then poured down the hole and given time to swell; the boreholes were finally filled with grout.

For the spade cell holes, the top 8m of granular material was bored through and cased using a percussion rig. Drilling through the London Clay was performed with a lorry-mounted short auger; these holes were not mapped prior to instrument installation. The holes were bored to within 1m of the desired installation depths, and the spade cells were pushed in 0.7-1.0m beyond the end of the borehole. The reaction load was provided by the 5-tonne lorry with a rear-mounted hydraulic jack connected to a 50mm diameter pipe attached directly to the cell top. After pushing, the pipe was uncoupled from the spade and the hole was finally back-filled with a cement-bentonite grout.

4.11 Total stress and piezometric measurements using pneumatic piezometers and push-in spade cells with integrated piezometers

The pneumatic piezometers installed at St. James's Park comprise a pneumatic transducer and flexible diaphragm as shown schematically in Figure 4.13a: a porous stone filter is used to separate the diaphragm from the surrounding material.

A simplified figure of a push-in spade-shaped pressure cell is given in Figure 4.13b. Each spade cell consists of an oil-filled chamber formed by two steel plates (approximately 100mm x 200mm) welded around the edges for measuring lateral earth pressures, and a pneumatic piezometer integrated into the spade with a porous stone built into the cell body. The external thickness of the cell measures about 6mm although the gap containing the oil reservoir between the plates is very small. The gap is maintained by a pre-set pressure incorporated into the cell during manufacture. The oil-filled chamber is connected to a pneumatic transducer with a short length of tube which is also completely filled with oil; a second pneumatic transducer is linked to the porous stone for pore pressure measurement. The pre-set zero for the earth-pressure cell was established through repeated measurements.

The transducers for both instrument types were calibrated under laboratory conditions.
The porous stones in the piezometers are ‘high air entry’ filters with small pore diameters and low permeabilities. Prior to installation the stones were de-aired by forcing air-free water through at high pressures. The stones remained submersed in de-aired water until the last possible moment before installation. The connections and fittings were also checked for leakage.

Prior to installation of each spade cell, a zero reading was taken at the surface, followed closely by a reading once the cell had been lowered to the bottom of the borehole and it had equalised with the temperature of the surrounding soil. Measurements of both pore pressures and earth pressures were recorded immediately after push-in and at regular time intervals thereafter to monitor the excess pore pressure dissipation.

Measurements of both the piezometers and the spade cells were carried out by connecting the twin tubes (one set for each transducer) to a portable readout unit (see Figure 4.13). A gradually increasing pneumatic pressure (nitrogen was used to prevent moisture entering the transducers or tubes) was applied to the transducer through the inlet tube until it slightly exceeded the pressure at the transducer, thus causing the diaphragm to deflect away from the inlet and outlet ports. With the diaphragm open, nitrogen flows through the transducer and returns to the readout unit; this is shown schematically on the inset drawing on Figure 4.13b. Flow rate through the transducer is balanced at the readout unit by the operator using a floating-ball system (i.e. the flow rate is the same for each measurement) and the reading is then taken.

4.11.1 Spade cell and piezometer measurements: uncertainty and sources of error

The pneumatic read-out unit used for both instrument types has a resolution of 0.1 kPa. The pneumatic operating system is sensitive to gas flow through the tubing (i.e. for a given pressure at one end of the tubing, the pressure at the other end depends on the flow rate and the length of the tubing). Careful control of the gas pressure and the flow rate will minimise errors. Displacement of the diaphragm can result in volumetric changes which could potentially change the pressure response. These errors were minimised by using diaphragms for both instruments which have minimal displacements during measurement. The effect is further reduced as the measurements are taken only after the reading stabilises under a constant rate of gas flow (Dunnicliff, 1988).

The calibrations for all transducers gave near-linear responses varying between 95 and 105% of the applied pressure, and thus the measured pressures are generally accurate to ±5% (full scale). For the range of pressures measured at the greenfield site in St. James’s Park, the piezometer measurement accuracy is likely to lie between ±5 kPa and ±10 kPa; for the horizontal total stresses the potential error could be as high as ±50 kPa.

The earth pressure spade cell is temperature sensitive, with a typical drop of 10°C from calibration to ground temperature yielding changes in measured pressures exceeding 20 kPa. This change is addressed by using the down-hole measurement prior to spade push-in, with the cell close to equilibrium with the in-situ temperatures, as the atmospheric or zero reading.

The act of installing the spade cell is critically important to the success of the stress measurement and will influence the magnitude of measured pressures; push-in of the spade
invariably generates high pressures locally around the cell. Depending on the soil permeability, a significant period of time can be required to dissipate the pore water pressures induced. Stress relaxation may also occur over extended periods of time. Verticality of the cell is also important to be sure the measurements are representative of the desired stress orientation.

Tedd and Charles (1983) and Tedd et al. (1989) investigated the measurement of in-situ stress by comparing horizontal stress measurement for both spade cells and other in-situ test devices (self-boring pressuremeter, dilatometer, etc). For stiff clays, they concluded that reducing the measured total earth pressure by half the undrained shear strength \((0.5s_u)\) generally improves the accuracy of the measurement.

It was also noted by Tedd et al. (1989) and Dunnicliff (1988) that drift over a period of months in the pre-set stress reading can occur which will potentially affect the longer term measurements made after tunnel construction.

For both the single piezometers and the piezometers integrated within the spade cell, a potential time lag may arise as a result of the time required for water to flow into or out of the piezometer to equilibrate with the pore pressures in situ; the lag is a property of the type and size of the piezometer used and the permeability of the surrounding soil. A small lag could be significant in monitoring tunnelling induced pore pressure changes because of the transient nature of the response. It is estimated that the approximate time-lag for 90% response to be realised for the estimated operating conditions at St. James's Park is on the order of 1 hour.

### 4.11.2 Processing of total stress and piezometer measurements

Total horizontal stresses are determined by taking the difference between the measured pressure on the cell, \(R\), and the zero reading of the cell at ground temperature prior to push-in, \(R_0\), and multiplying by the calibration factor as given in Equation 4.11. Adjustment for pressure over-read (Tedd et al., 1989) is also made as described in Section 4.11.1 above.

\[
\sigma_h = (R-R_0)\times CF-0.5s_u
\]

Pore pressures are taken as direct readings from the readout unit.
5 St. James's Park site and JLE project works

This chapter gives an overview of the site by describing the surface, subsurface, and in-tunnel instrumentation arrays and the progress of Jubilee Line construction works beneath the instrumented section. Included are the following information:

- brief historical background
- overview of JLE project works influencing the site
- details of the monitoring goals and justification of instrumentation design
- review of the surface, subsurface and in-tunnel instrumentation locations relative to the tunnelling works
- information pertaining to shield characteristics and lining design
- definitions of the time periods in which the ground response occurred
- observations of excavation and tunnelling performance from JLE tunnelling engineers and inspectors.

The descriptions of the instrumentation details and the monitoring techniques used are presented in Chapter 4.

5.1 Introduction

St. James's Park is a broad area of trees, lawns and gardens with The Mall and Birdcage Walk as its northern and southern boundaries. To the west lies Buckingham Palace and the Queen Victoria Memorial, and Horse Guards Road forms its eastern boundary. The area was previously both common and cultivated swamp land; in the 1400s the area was used by the St. James's Hospital (now the site of St. James's Palace, Westminster) as grazing land for their livestock. Henry VIII enclosed the area with walls for use as parkland in the 1500s. Charles II extended the area of the park in the 1600s and built a small canal to join several small ponds. In 1826, during the reign of George IV, the canal was remodelled into a lake of curvilinear form which is still seen today. In 1855, the lake was dredged, concreted, and made a uniform 4 feet deep.

The instrumented site is located within the Park near the intersection of Horse Guards Road, Stow's Gate, Great George Street and Birdcage Walk as shown on Figure 5.1. The area around the park has an illustrious history, demonstrated by the presence of prestigious Georgian houses, private clubs, professional institutions, royal palaces, and government offices. Historical maps indicate that this corner of the park is free from previous man-made structures or building loads and is considered to be a greenfield site.

5.2 Site layout, instrumentation plan and tunnel positions

Twin 4.85m diameter running tunnels were constructed at separate times in very stiff London Clay using open-face shields and mechanical backhoes for excavation. Figures 5.2 and 5.3 show
their relative positions in plan and section along with the surface and subsurface instrumentation arrays used at the site. The local geology and site investigation results are summarised in Chapter 3.

The westbound tunnel advanced beneath the site at about $z=-31$ m (depth to axis level) in April 1995 while the eastbound tunnel was built at $z=-20.5$ m (axis level) in January 1996. At the line of SMPs on Figure 5.2, the two tunnels are 21.5 m apart in plan, diverging towards Green Park and rising slightly from a low point at the nearby Storey’s Gate escape shaft (see Figure 5.1). The westbound running tunnel intersects the instrumented line at an angle of approximately 80 degrees while the eastbound is essentially perpendicular.

5.3 Instrumentation layout

The plan of instrumentation on Figure 5.2 indicates the surface positions of the instruments on installation. Mapping of the boreholes summarised in Appendix D can be referred to for more accurate subsurface locations.

24 surface monitoring points (SMPs) are positioned at 2.5 m spacings and in a line nearly perpendicular to both tunnel axes: SMP no. 3 is located precisely on the westbound tunnel centre-line. Nine electrolevel inclinometer holes (Ai to Hi, Ji) were bored along a line approximately 4 m further towards Green Park and parallel to the SMP line at the spacings indicated on Figure 5.2. The extensometer array was arranged along two lines in order to avoid overhanging tree branches and the pavement during drilling: line 1 includes Ax, lx, Jx, and Kx; line 2 includes holes Bx to Hx. The position of the lines relative to the SMP line are seen on Figure 5.2. The piezometers are positioned along each tunnel centre-line, with 2 m spacing (in plan) between each adjacent instrument. The line for the spade cells runs perpendicular to the SMP line. The positions of the total station locations described in Section 4.3 are also shown on the figure.

Figure 5.4 shows the locations of the extensometer anchors and the electrolevel carriages and their identification numbers. The approximate coordinates of all the individual instruments are tabulated in Appendix E, with the centre of the coordinate system (0,0,0) being the position of SMP no. 3 on the westbound tunnel centre-line. The x,y,z convention follows the system given in Figure 2.8. The positions of the SMPs, piezometers and spade cells are also included in Appendix E.

Surface settlement monitoring lines were established by the contractor for the JLE at several locations across the length of St. James’s Park as shown on Figure 5.5. The coordinates of these monitoring points relative the westbound centre-line are also listed in Appendix E. Data from these points are included with the research results presented in Chapter 6.

5.4 Monitoring goals

Peck (1972) succinctly summarised the role of field observations as a tool in applied geotechnics by stating ‘We need to carry out a vast amount of observational work, but what we do should be done for a purpose and done well’. Thus each tool within a monitoring program should have
a purpose, and the monitoring techniques should be well conceived and repeatable.

The instrumentation at St. James's Park is selected primarily for the measurement of surface and subsurface movements above the twin tunnels. It was considered that the best results would be obtained using the state-of-the-art in traditional measurement tools (such as the electronic precision level and the electronic total station theodolite) and mechanical devices for measurement (micrometer stick, rod extensometer, and collimator tool). More sophisticated instrumentation such as the electrolevel inclinometers were used with a degree of scepticism as the instruments are delicate and (relatively) expensive. Additionally, the accuracy of the overall monitoring system (i.e. the inclinometer hole) is dependent on the performance of numerous individual components (the electrolevels) which, at the time of installation, had yet to achieve the high degree of reliability required for a 'primary' monitoring tool. Thus the inclinometers tubes were also monitored with a more traditional measurement tool (i.e. inclinometer torpedo) to provide a method of calibration and assessment of the electrolevel performance during and after tunnel construction.

Redundancy of measurements was deemed essential to confidently interpreting the recorded responses, and so the monitoring points and techniques at St. James's Park were devised to allow different systems to measure changes on the same points. Datum points are also critical to the success of monitoring systems. The bottom of inclinometer Di and the deepest extensometer (Dx) are intended as datums for vertical and horizontal movement; both are two diameters from the westbound tunnel extrados and nearly one and a half diameters below the invert. The surface monitoring systems use reference points affixed on buildings near to the site but outside the zone of tunnelling influence: these were tied in with the deeper datum to assess their stability, particularly in the long-term.

The measurements of pore pressures and total stress changes near the tunnels are directed at recording the total and effective stress changes around the tunnel during construction, and the longer-term pore pressure changes after tunnel construction. Very few measurements of this type have been made around tunnels in London Clay and the results will help to improve the understanding of the soil reaction to tunnel excavation. Pneumatic piezometers and spade cells were chosen primarily because of the wealth of experience that the Building Research Establishment had with these instruments.

### 5.5 Shield and lining details

The twin shields utilised for tunnelling are shown on Figure 5.6. They are 4.85m outer diameter (o.d.) and about 4.2m in length from cutting edge to shield tail. The maximum reach of the backhoe in advance of the shield cutting edge is about 1.9m.

Around the cutting edge, a ‘bead’ comprising a small thickness of metal is used to leave a small gap between the clay and the external shield surface; this gap, in conjunction with small amounts of wooden packing inserted selectively around the circumference between adjacent rings prior to shoving the shield forward, facilitates steering of the shield. The bead also reduces transfer of the soil load directly on to the shield so that the jacking pressures required to advance
the tunnel are kept minimal. The details of the bead given on Figure 5.7 show a maximum permanent welded bead of about 13mm at the shield crown reducing to 3mm at the invert. Additional bead thicknesses can be bolted to the cutting edge to provide extra clearance if ground closure on to the shield is too rapid.

Tunnel support is provided by a 200mm thick expanded concrete lining assembled from 10 segments as shown schematically on Figure 5.8 together with the two key segments. The lining is erected in the ring-build area by slotting each segment into its place around the extrados; each segment is held in place temporarily using the shield jacks. The keys are finally positioned at knee level on both the left-hand and right-hand sides (LHS, RHS) of the ring, and then shoved into place with jacks at about 100 bars of pressure to expand the lining ring to its full size. The standard assembly to achieve an external diameter of 4.85m utilises two different key sizes (A and B) pushed to a central position as indicated on Figure 5.9c. Other key combinations are also shown on this figure along with the calculated external diameters for each arrangement. The use of two small keys during lining erection yields a lined diameter up to 25mm smaller than the excavated diameter. This is potentially significant to the calculation of gap around the shield for volume loss calculations.

The lining is erected in the area between the tail of the shield and the last constructed ring. Trailing fingers formed from strips of metal attached to the rear of the shield (see Figure 5.6) were used to prevent blocks falling into the ring build area from the crown.

5.6 In-tunnel monitoring

Measurements of diametric changes after tunnel construction were made in both tunnels using a digital tape extensometer and a steel tape: temperatures were also recorded to correct for thermal variations on the tape. The tape extensometer is the same instrument used to measure average horizontal strains in structures as described by Standing (1998). In both tunnels the installation of the measurement points was feasible only after the shield ‘train’ (including the spoil removal conveyor and loading facilities, grouting equipment, temporary track laying, etc.) had cleared the rings (i.e. when the tunnel face was about 40m beyond the ring of interest). The results are presented in Chapter 6.

In the eastbound tunnel, measurements of lining loads were undertaken by TRL (Davies and Bowers, 1996) through two specially designed instrumented rings shown on Figure 5.10a and installed near to the instrumented site (ring nos.0995 and 0996). The first comprises vibrating-wire strain gauges fitted to the reinforcement cage of four segments as indicated on Figure 5.10b. The second and adjacent ring has four load cells (at both spring lines, and one each at the crown and invert) fitted as shown in Figure 5.10c. At each position, a gap is left between the adjacent segments so that all of the circumferential stresses are transferred to the load cells. Results from these instrumented linings are presented in Chapter 6.
5.7 Tunnel progress and monitoring periods

A typical level response for a monitoring point located between the two tunnels is shown on Figure 5.11 along with dates which mark five distinct ground response periods: Period 1 is prior to any tunnelling works during which time the ground is considered stable; Period 2 spans the construction of the westbound tunnel; Period 3 is a 'rest' period where no JLE construction works were affecting the site; Period 4 encompasses the construction of the eastbound tunnel drive; and Period 5 is the long-term monitoring period after completion of the tunnelling works beneath the site. The period timings are summarised in Table 5.1.

Period 2 (westbound tunnel construction) encompasses nearly 65m of tunnelling between expanded lining ring nos.0942 and 1007 (note: ring nos. increasing towards Green Park). The shield cutting edge crossed the SMP line during erection of ring no.0977. Period 4 (eastbound tunnel construction) includes about 45m of tunnelling (ring nos.0967-1012). The cutting edge passed immediately beneath the SMP line during erection of ring no.0987.

All of the monitoring was performed by the author, and staff from Imperial College and JLE involved in LINK Construction Maintenance and Refurbishment research project. During Periods 2 and 4, 24 hours-per-day monitoring was completed with the additional help of graduate students and staff from Imperial College, CIRIA. Throughout the monitoring, a careful note of time was kept to allow correlation of the measured response with the corresponding tunnel position.

5.8 Observations during tunnel excavation

5.8.1 Period 2 (westbound drive)

There was no formal pattern or sequence of tunnel face excavation beneath the instrumented section until the shield was about half-way through the instrumented area (at ring no.0992); however, it is likely that a systematic excavation method was adopted by each backhoe operator for their shift. From no.0992 onward a set pattern was used with excavation beginning at the left-hand side (LHS) knee and working around the face counter-clockwise. The length of excavation in advance of the cutting edge was estimated on JLE daily works reports to be at a maximum of 1.9m for all of Period 2 (and for all of the tunnelling from Westminster to the instrumented section). These reports also indicate that the linings on erection were very tight to the cut circumference, and that ring nos.0964 to 0991 had to be built with two small (B) keys as the push-in pressures for the larger (A) key were excessive and caused damage to the key itself. Packing (about 10mm thick) was being used between adjacent rings at the top segments as the shield advanced beneath instrumented area.

5.8.2 Period 4 (eastbound drive)

JLE daily works reports make no observations of the excavation patterns during construction beneath the instrumented section. When advancing beneath the nearby Treasury Building both breasting plates and forepoling were used during excavation and shield advance, but it is unlikely
that the same methods were employed beneath the monitoring line. It is also noted that tunnelling was stopped and the face boarded-up for the Christmas break when the shield was less than 50m from the section. Subsequently, the shield passed beneath the instrumented line within the first two days after tunnelling resumed. Packing of between 10 and 20mm thickness was being placed at the tunnel crown and right-hand side (RHS) axis level respectively for ring nos.0964-0975.

5.8.3 Other information
A survey of the lining rings between Westminster Station and the step-plate junction at Green Park after completion of both tunnels was attempted by the author to investigate the number of rings for which two small keys were used for lining erection, and their positions along the tunnel drives. The necessity of using two small keys may indicate rapid closure of the gap left by the bead around the tunnel and imply something about the ground response during tunnelling.

It was concluded from records of ring-build details that only a few rings had been erected using two small keys on the westbound tunnel (Harris, 1998). The combination was used with greater frequency on the eastbound tunnel but primarily along Great George Street where compensation grouting was being performed in parallel with tunnelling works.

5.9 Discussions
The shield advance was rapid for both tunnels (i.e. more than 0.9m h), which is sufficiently fast for undrained response of a low-permeability clay during construction. The tunnel depths of 31m and 20.5m lead to C/D ratios (depth of cover to the tunnel crown over tunnel diameter) of 6.3 (westbound) and 4.2 (eastbound). The combination of maximum backhoe reach, shield length and ring build area yields a potential $P/D$ ratio (length of unsupported excavation over tunnel diameter) of at least 1.5 (assuming the shield lends no support to the excavation). Limiting the backhoe reach in front of the shield cutting edge to 1m yields a reduction in $P/D$ to 1.3. Figure 2.3 in Chapter 2 shows the influence of $P/D$ and $C/D$ on the critical stability number, $N_{1c}$. Although Mair (1979) only investigated fairly shallow tunnel depths, it is inferred from the graph that even for deeper tunnels ($C/D$ greater than 4), $P/D$ plays an important role in stability. The change in $P/D$ described above (which could realistically be achieved during construction) can yield a change in $N_{1c}$ of about 0.7 to 1.

The stability ratio, $N (\sigma_\ell/s_u)$, for each tunnel can be calculated using the undrained strengths ($s_u$) estimated at the tunnel crowns from the design line given on Figure 3.5 ($s_u = 215$ kPa westbound, =130kPa eastbound). For both tunnels the stability ratios are about 2.8 which, based on case history data for tunnels in clay on Figure 2.14, indicate potential volume losses of between 2 and 3%. However, using an average value for $s_u$ at the westbound crown from the SI results on Figure 3.5 ($s_u = 150$ kPa) gives a stability ratio of nearly 3.9, which might be expected to give larger volume losses.
6 Monitoring results from St. James’s Park

This chapter presents the results of: surface displacement monitoring; subsurface displacement measurements: total stress and piezometric observations; and in-tunnel monitoring. The monitoring is identified in terms of five ground response periods identified in Table 5.1. Schedules of monitoring surveys for all periods are given in Appendix F. Surface levelling results from the JLE contractor’s data obtained at different sections through the park (see Figure 5.5) for each tunnelling event are also included for comparison.

Measurements obtained during Period 1 were used to establish base readings for all measured quantities, and to assess the performance of the monitoring techniques and instruments used. The displacement measurements were reviewed utilising two statistical measures of precision. The first is the standard deviation for measurement which applies to the calculated normal deviation about the mean value for a given measurement (e.g. SMP level, span measurement, x, y and z coordinates, etc.). The second is the standard deviation for survey which considers the normal deviation about the mean measured difference for all monitoring points within the survey (omitting obvious anomalous values). The former gives a measure of the precision for a single measured quantity while the latter gives an indication of the overall survey performance, particularly for total station measurements.

Figure 6.1 shows the conventions used to describe the 3-dimensional movements relative to the x, y and z coordinate system around the progressing tunnels (see Figure 2.8). At the top of many plots a key diagram is given which shows the distance between the shield cutting edge and the monitoring line of interest, or x. A positive x offset means the leading shield edge is approaching the instrument line, while a negative x implies that the shield edge has moved past.

Horizontal displacements parallel to the tunnel axis, u, are positive if they are in the positive x direction (i.e. in the direction of tunnel progress). At St. James’s Park, positive u displacements are west and towards Green Park, while negative u movements are eastward towards Westminster.

The sign convention for horizontal displacements transverse to the tunnel axis, v, are positive in the positive y direction (see Figure 6.1). Positive y direction at the instrumented section is referred to in this thesis as north (although it is closer to north-east). Positive v displacements for SMPs south of the tunnel centre-line are northward and towards the tunnel centre-line. However, for SMPs north of the tunnel, v displacements towards the tunnel centre-line are negative.

For vertical displacements, w, positive movements are upward (positive z) implying ground heave. Negative w displacements are downwards, and are often referred to as settlement. All results presented in this chapter are consistent with this framework.
6.1 Surface monitoring for Period 1

6.1.1 Vertical displacements for precision levelling

Sixteen precision levelling runs (nos. 1-16) were made prior to construction of the westbound running tunnel beneath the site. Figure 6.2a shows for all SMPs the variation with time in measured levels from the average level. The magnitude of difference for all points and for all surveys rarely exceeds ±0.3mm. For a given survey, the magnitudes and sense of difference are similar for all SMPs; the standard deviations for survey are all less than 0.2mm and are generally about 0.1mm.

Figure 6.2b shows the levelling data as profiles of difference from the average levels across the SMP line; the standard deviations for measured levels are also given for each SMP. The variability in the measured level increases with distance from the local datum, but the standard deviations for measurement are still less than 0.2mm for all points.

These 16 surveys demonstrate that confidence in the measured levels to about ±0.2mm is justified. This range is within the ±0.3mm range concluded from general considerations of precision for the monitoring system in Chapter 4.

Base levels for westbound tunnel construction were chosen as the average of all 16 surveys in Period 1 excluding any obvious anomalous values (identified by visual inspection only).

6.1.2 Transverse horizontal ground strains and displacements by micrometer stick

Twelve measurement sets (nos. 1-12) were obtained prior to tunnelling, ten of which are represented on Figure 6.3 (set nos. 1 and 2 are excluded because the monitoring techniques were not standard). All measurements are temperature corrected as discussed in Section 4.5.2.

Figure 6.3a shows for all spans the variations with time in the measured span from the mean measurement for Period 1, and the standard deviation for each survey. Differences from the mean values for all spans and all surveys are generally less than ±0.25mm (equal to approximately ±0.01% strain for the 2.5m spans being measured). The standard deviation for a given survey seldom exceeds 0.15mm.

Figure 6.3b presents profiles of differences from the average measured spans across the SMP line and the associated standard deviations for measurement at each span. Differences from the mean values are generally less than ±0.25mm except for span 18-19 for which larger variations (up to ±1.1mm) were observed; these resulted from slight wobble in the post when screwed into SMP no.19. Because of the survey technique used, which tended to pull the post in SMP no.19 to a repeatable position for each measurement, a similar variation is not realised for span 19-20. The standard deviations for span measurements are less than 0.2mm except for span 18-19. Generally, the accuracy of ±0.25mm agrees well with that anticipated from the instrument characteristics in Section 4.5.1.

The averages of survey nos. 3-12 were taken as the base span readings for westbound
tunnel construction omitting any obvious anomalous values (identified by visual inspection only).

The apparent displacement profiles (base values as the averages measured during Period 1) are shown on Figure 6.4. Changes are summed across the line, assuming SMP no.24 at the end of the line is stable. The variability for measurement at span 18-19 is removed by taking the average change in measurement for the spans to either side. The cumulative outcome of small and systematic differences for each span is a linear transverse displacement profile of apparent horizontal displacements, with apparent movements at the westbound tunnel centre-line ranging between ±1.5mm. It is inferred from the data that there are systematic errors for the micrometer stick measurements which are compounded by summing the changes. The errors are probably the result of changes in ambient conditions (i.e. temperature) which put the aluminum bar out-of-equilibrium with the recorded temperature. Thus, the precision for determining the relative displacements by summing the span changes across the SMP line is ±1mm.

6.1.3 Parallel horizontal displacements by collimation tool
Prior to the westbound tunnel affecting the site, five collimation surveys were made (survey nos. 1-5). The measurements are presented in Figure 6.5a as the difference of the calliper measurements from the mean values varying with time; standard deviations for survey are also shown. Because of poor monitoring procedures, Survey no.3 is not included on Figure 6.3a and is omitted from further analysis. The differences from the mean values for all readings and for all surveys is generally less than ±1.5mm (±1mm if survey no.2 is excluded). For a given survey the observed magnitude and direction of differences are similar for all SMPs. The calculated standard deviation for survey of each set is around 0.5mm, with the exception of survey no.2 which is about 1mm.

Figure 6.5b shows the same data as profiles across the SMP line of differences from the mean measurements along with the standard deviation for measurement at each SMP. The variation is again ±1mm (if survey no.2 is excluded) and the standard deviation for measurement varies between 0.2 and 1mm with a slight increase for SMPs closer to the instrument location and further from the point on which the line of sight for measurement is set.

The large standard deviation observed in survey no.2 is the result of a sighting error onto the reference points; the error is reflected in the profile shown on Figure 6.5b as open circles but is excluded from further analysis. The averages of calliper measurements from surveys 1, 4 and 5 were taken as the base readings for westbound tunnel construction.

6.1.4 3-dimensional displacements by total station
During Period 1 there were twelve total station surveys; eight of these were made from total station position ST1 only (i.e. at the north end of the SMP line, see Figure 5.2), and four were from both ST1 and ST2 station positions (nos. 6, 8, 11, 12). During this period the monitoring techniques were evolving to an appropriate standardised practice. Of these four surveys, the techniques used from ST2 only allowed the collimation or face errors to be corrected on one of
Several of the surveys made only from ST1 did not have sufficient readings to allow for face corrections. Survey details are summarised in Appendix F.

Plots showing the variation with time of calculated coordinates from Period 1 means for surveys performed from ST1 and the calculated standard deviation for survey are given on Figure 6.6. The same data are given on Figure 6.7 as profiles across the SMP line. Calculated standard deviation for measurements are also given for each SMP. Figures 6.8 and 6.9 show the complementary results for surveys from ST2. The mean values for each coordinate exclude surveys for which the standard deviation for survey or overall difference from the mean values show inconsistencies with the general trend. The x, y, and z coordinates follow the convention given on Figure 2.8 relative to the tunnel axis.

Figures 6.6 and 6.7 (ST1)

x coordinate.
The variation in the x coordinates (parallel to tunnel) for all SMPs and all surveys are generally within ±2mm of the mean value; survey no.2 shows between 2mm and 10mm difference from the mean value. For any one survey, the differences for all SMPs are in the same direction and broadly similar in magnitude. Standard deviation for surveys are all less than 1mm except for survey no.2 which is closer to 2.5mm.

Profiles of the differences from the mean coordinate values given on Figure 6.7 show linear variations across the site with SMPs further from the total station position showing larger differences than those closer to the instrument. Standard deviations for measurement at each SMP location vary between 0.7 and 1.9mm, increasing with distance from the total station location.

The average x coordinates for survey nos.1 and 3-12 were taken as the base measurement for westbound tunnel construction. Survey no.2 was excluded from analyses because of its large standard deviation for survey.

y coordinate.
The calculated y coordinates (transverse to tunnels) for all surveys are – with few exceptions – within ±2mm of the mean values. For a single survey, the magnitudes and sense of difference are similar for all SMPs. Standard deviations for survey are less than 0.5mm for most sets.

Shown as profiles, the differences are seen to be consistent across the SMP line. Standard deviations of measurement for all points range between 0.6 and 1mm; no clear trend across the monitoring line in measurement deviation is apparent.

The base y coordinates for westbound tunnel construction were taken as the average of all 12 sets in Period 1.

z coordinate.
Surveys corrected for collimation errors plotted at the bottom of Figure 6.6 show variations from the mean value of ±1mm and standard deviations for survey of less than 0.5mm.
Measurements which are uncorrected (survey nos. 4, 8, and 10) show larger deviations for survey (about 1mm).

Differences from the mean plotted as profiles on Figure 6.7 show strong divergence between the collimation-corrected surveys and the other surveys. Differences are exaggerated for points closest to the station location, for which the vertical collimation correction is most crucial. Standard deviations for measurement at each SMP (for the collimation-corrected surveys only) vary between 0.5 and 1.1mm with no obvious trend across the monitoring line.

The base level for the \( z \) coordinate was taken as the average of all collimation corrected surveys. Survey nos. 4, 8, and 10 were omitted because of their larger standard deviations for survey and absence of collimation corrections.

*Figures 6.8 and 6.9 (ST2)*

\( x \) coordinate.

Differences from the mean values of the \( x \) coordinates for all SMPs are seen to be less than ±1mm with the exception of survey no.11 which shows larger overall differences (±2mm). The magnitudes of the observed differences from the mean value for a given survey have a similar variation for most SMPs. The standard deviations for survey are less than 0.5mm, except for the 1mm deviation calculated for survey no.11.

As a profile of difference from the mean values, survey no.11 is nearly linear across the monitoring line, increasing with nearness to the total station location. Profiles for the other surveys show differences which are broadly similar in magnitude for all SMPs. The standard deviations of measurement show a range of 1 to 1.5mm, increasing with distance from the total station location; some individual points have deviations of measurement as low as 0.3mm.

The average measured \( x \) coordinates for survey nos. 6, 8, and 12 were taken as the base values for westbound tunnel construction. Survey 11 was omitted because of its larger standard deviation for survey.

\( y \) coordinate.

The differences in calculated \( y \) coordinates for all surveys and all SMPs are within ±1.5mm. The magnitude and sense of difference is similar for all SMPs in a single survey, and the standard deviations for survey are between 0.5 and 1mm. Differences for survey no.11 are much larger (~10mm) but the deviation is similar to the others calculated.

Profiles of differences from the mean \( y \) coordinates (Figure 6.9) show similar movement direction and magnitude of all SMPs for a single survey. The standard deviation for measurement lies between 0.5 and 1.5mm, decreasing with distance from the station location.

The base measurements for the \( y \) coordinate were taken as the average of survey nos.6, 8, and 12. Survey 11 was omitted because of its large differences from the mean values.

\( z \) coordinate.

Survey no.6 was the only survey corrected for collimation errors and was taken as the mean
value for comparison. At the bottom of Figure 6.8 the surveys show differences up to 6mm from no.6, but are consistent among themselves to about ±2mm. The standard deviations for survey are all greater than 1.5mm.

The profiles of displacement relative to survey no.6 given in Figure 6.9 show larger differences for the SMPs closest to the station location, because of the stronger influence the vertical collimation error has on the z coordinate for steep sights.

Survey no.6 was taken as the z coordinate base measurements for westbound tunnel construction. Surveys 8, 11, and 12 were omitted because of the absence of face corrections.

6.1.5 Assessment of surface displacement monitoring
The suite of surface monitoring employed at the instrumented site has several redundant measurements which allow for comparisons between different techniques and tools for determining ground displacements. The precision for measurement of each displacement coordinate and technique (as standard deviations for measurement) are summarised in Table 6.1.

For measurement of the x-coordinate displacements by the collimation tool (movements horizontal and parallel to the tunnel axes) has sub-millimetre precision. This is the first time that measurements of this component of movement have been achieved to such a high degree of precision. The precision for the same horizontal direction from the total station is somewhat larger.

For measurement of the y-coordinate displacements by micrometer stick (movements horizontal and transverse to the tunnel axes), the standard deviation for measurement of a span change is less than 0.2mm. Cumulative errors in determining a displacement profile from the span measurements can result in nearly linear profiles of apparent displacement, with a maximum movement ranging from ±2.5mm (see Section 4.5.2 on processing micrometer stick measurements). The total station surveying has a maximum deviation of 1.5mm for measurement of y-coordinate movements: this is clearly an improvement over the micrometer stick measurements.

For vertical movements (z coordinate) the levelling has a standard deviation for measurement of less than 0.2mm, while the total station surveying gives a poorer precision, with a maximum deviation of about 1.1mm (for collimation-corrected surveys).

For total station and collimation tool surveys, the differences from the mean values are often similar in magnitude for all SMPs in a given survey and the standard deviations for survey are usually small. These two observations suggest that the standard deviations for measurement likely reflect errors in positioning the instruments, movements of the local datums and reference point, and random measurement errors onto the datums and the reference point positions.

The accuracy using the total station and collimation tool may be improved during the tunnel monitoring by reviewing the precision levelling and the micrometer stick measurements to determine which SMPs are stable and unaffected by tunnelling. The total station and collimation measurements to all SMPs can then be adjusted by the apparent movements determined for these stable SMPs (such that the stable ones give negligible movements): this
should minimise the problems associated with total station positioning errors.

The values for precision given in Table 6.1 compare very favourably with the expected precision and accuracies described in Chapter 4.

6.2 Surface monitoring in Period 2

Period 2 monitoring events are summarised on Figure 6.10 which shows the position of the shield edge relative to the SMP line with time, and the timings of levelling runs, micrometer stick traverses, total station surveys and collimation measurements. During this period the tunnel was advancing at an average rate of 45m/day (~1.9m/h or 2 rings/h). The results for each monitoring type are described below.

6.2.1 Vertical displacements for precision levelling

Twelve sets of levels were taken (nos. 17-28) during Period 2. The development of settlements with time are given on Figure 6.11 for SMP nos.1-13 (nearest to the westbound centre-line) and SMP nos.21 and 24 at the far end of the instrument line (see instrumentation plan, Figure 5.2). The effect of tunnel construction was first observed at survey no.20; later surveys showed increased displacement magnitudes for SMP nos.1-13 until survey no.27 after which the movements stabilised. SMP nos.21 and 24 show very little movement during tunnel construction.

Transverse settlement profiles are given on Figure 6.12. The key diagram at the top shows the approximate position of the tunnel face in relation to the SMP line for each survey. The instrumentation line enters the influence zone of the tunnel when the face is between 13m (survey no.19) and 6m (survey no.20) from the SMP line. The later surveys show a deepening of the settlement trough until the shield face is about 30m past the SMP line. A maximum vertical displacement of -20.4mm at the tunnel centre-line is observed for survey no.29 when the tunnel is over 40m past the SMP line. The volume loss for this profile, $V_v$, represented as the volume of the perpendicular surface settlement trough, $V_s$, divided by the theoretical volume of soil excavated at the tunnel face (i.e. $\pi D^2/4$ per metre advance), is 3.3%, assuming symmetry about the westbound centre-line.

SMPs at offsets greater than 45m (nos. 21-24) are essentially unaffected by tunnel construction. This is demonstrated by the statistical assessment of the calculated displacements on SMP nos.21 to 24 tabulated on Figure 6.11. The standard deviations for measurement on these points range from 0.2 to 0.3mm, which are agreeable with the results during Period 1 prior to tunnelling.

The contractor’s levelling data for monitoring lines A-G are shown on Figures 6.13 and 6.14; the data are split into profiles north of the St. James’s Park lake and profiles south of the lake (see Figure 5.5). Line A lies about 5m towards Green Park from the SMP line at the instrumented section.

Lines A, B and C south of the lake all show maximum displacement magnitudes at the tunnel centre-line of between -18 and -20mm; the last survey on each plot (5-May-95 10:00) was
taken several days after the tunnel had passed. The volume losses at these sections are estimated
to be about 3.5%. The data from Line A shows that the final construction profile above the
westbound tunnel is fairly symmetrical about the centre-line.

North of the lake (Figure 6.14) the centre-line settlement magnitudes at lines D-G of
between 9 and 11.4mm were recorded, with estimated volume losses of between 1.5% and 2.0%.

6.2.2 Transverse horizontal ground strains and displacements by micrometer
stick
Twelve measurement sets (nos. 13-23) taken during westbound tunnel construction are shown
in relation to tunnel progress in Figure 6.15 as profiles of average strain transverse to the tunnel
axis measured between adjacent SMPs: a key diagram showing the approximate position of the
tunnel face at the start of the survey is also given. As with the precision levelling, movements
are first observed when the tunnel face is between +13 and +2.5m from the SMP line; the strain
magnitudes increase until survey no.23 when the tunnel is over 30m beyond the instrument
plane. The maximum average tensile ground strain measured is about 0.0004 or 0.04%, while
the maximum average compressive strain is about -0.001 or -0.10%. Spans at offsets greater
than 45m appear unaffected by tunnel construction, showing strain ranges (see Figure 6.15)
which are only slightly greater than those observed prior to any tunnelling near the site during
Period 1.

The final calculated horizontal displacement profile due to tunnel construction (taken as
the average of survey nos. 21-24) is shown in Figure 6.16. The displacements are determined
from integrating a curve fitted to the measured strains as described in Chapter 4, assuming zero
average strains beyond the end of the SMP line (i.e. offsets greater than 52m). A maximum
movement of about -6mm (towards the tunnel centre-line) is calculated occurring at an offset
of about 14m. The offset corresponding to zero horizontal displacement is seen at approximately
+3m from the westbound centre-line.

The calculated displacements at negative offsets from the tunnel centre-line appear larger
than as similar positive offsets, but these differences probably arise from the curve-fitting.

6.2.3 Parallel horizontal displacements by collimator tool
Five collimation surveys were performed as indicated on Figure 6.10: survey nos.3-5 were all
performed when tunnel face was more than 15m away from the SMP line, and have instead been
included in the Period 1 measurements (Section 6.1.3) to improve the quality of the base
readings. Survey nos.6 and 7 were made with the tunnel face 25m and 40m beyond the line and
essentially after tunnel construction beneath the site. Survey nos.8 and 9 were performed within
12 days after tunnel completion (i.e. in Period 3).

Profiles of calculated displacements, u, parallel to the tunnel centre-line relative to the
mean readings are given on Figure 6.17 along with the standard deviations for measurement at
each SMP. An error in survey no.8 appears to have occurred, marked by a jump in the
calculated profile: the data shown as open symbols are not included in the mean value or
deviation calculations. Generally the calculated displacements are negligible over the entire SMP line, except possibly for some small displacements near the tunnel centre-line (<1.5mm) towards Westminster. The standard deviations for measurement are generally less than 0.5mm. The deviations are much smaller than the 1mm calculated for Period 1 as a result of survey adjustments performed to account for apparent displacements observed on SMP nos.21-24 (beyond 45m offset). These points were shown from precision levelling and ground strain measurements to be essentially stable during westbound construction.

6.2.4 3-dimensional displacements by total station
Five total station surveys were made during Period 2. Survey nos.13 and 15 from ST1 and survey nos.13, 14, and 15 from ST2 were not corrected for collimation errors. The calculated displacements for all surveys are presented on Figures 6.18 and 6.19 for ST1 and ST2 data respectively. Key diagrams at the top of each figure indicate the approximate distance from the shield edge to the SMP line at the beginning of each survey. The calculated vertical displacements for surveys not corrected for collimation errors are shown on the plots as open data symbols; these are not used for further data analysis.

Figure 6.18 (ST1)
The top plot shows transverse profiles of parallel horizontal displacement, \( u \). Movements are first observed when the tunnel face is about 7m from the SMP line (survey no.13). The profile at this time shows movement of -5mm near the westbound centre-line (towards the tunnel face and Westminster), decreasing to nearly zero at points beyond 30m offset. With the tunnel face approximately 2.5m past the SMP line a maximum displacement towards Westminster of about -8mm is recorded at the tunnel centre-line with a similar displacement distribution with offset. With the shield moving past the SMP line the overall magnitude of displacements begin to decrease. A maximum of -3mm is seen at the centre-line when the tunnel face is 10m past the monitoring line (survey no.15). The final displacement profile is observed for survey nos.16, 17 and 19 with the tunnel over 30m past the instrument plane; the profile shows some small displacements (up to +3mm) towards Green Park at offsets near the tunnel centre-line. The profile for survey no. 18 shows apparent displacements at SMPs offset over 20m from the tunnel centre-line and magnitudes of up to +7mm (towards Green Park). This is not consistent with the displacements of survey nos. 16, 17 and 19 and the survey has therefore been omitted from further analysis.

The middle plot shows transverse profiles of horizontal displacements, \( v \). Small movements (±3mm) are seen on either side of the tunnel and towards the centre-line when the tunnel face is about 7m from the SMP line (survey no. 13). The profile shape becomes clearer when the tunnel face is 2.5m past the line (survey no. 14); nearly 4mm of movement (towards the centre-line) is recorded at +10m offset. The final profile (survey nos.16-19) shows a maximum movement towards the tunnel centre-line of -7mm at an offset (\( y \)) of between +10 and +12.5m. The offset coinciding with zero displacement is found at approximately +2.5m from
the centre-line.

The bottom plot shows the vertical displacement profiles. w. The data symbols for survey nos. 13 and 15 are open to indicate no face corrections were made. When the tunnel face was 2.5m past the SMP line (survey no.14) -12mm of vertical movement (settlement) is recorded at the centre-line. The final profile (seen for survey nos. 16-19 performed when the tunnel was over 30m past the SMP line) shows a maximum displacement of -21mm (settlement) with a near normal distribution of displacements about the centre-line.

**Figure 6.19 (ST2)**
Transverse displacement profiles of horizontal movements, u. are given in the top plot. When the tunnel face is approximately 4m from the SMP line (survey no.13) a maximum displacement of -5mm (towards Westminster) is observed at the tunnel centre-line; the profile shows decreasing displacement with increased SMP offset from the tunnel as was observed from ST1. Surveys nos. 14 and 15, made when the tunnel face was 7 and 12.5m past the monitoring line, show little change in the profile shape or displacement magnitudes. The final profile (determined from survey no.18) when the tunnel is over 40m past shows nearly zero displacement across the SMP line except for some small movements (~2mm) towards Green Park at offsets near the tunnel centre-line.

Horizontal movements transverse to the tunnel axis, v. are given in the middle plot. The profiles show similar trends of displacement growth as seen from ST1. Movements of -4mm (towards the tunnel axis) are seen at about +15m offset when the tunnel face is 4m from the SMP line. The displacement magnitudes increase as the tunnel moves past the instrument plane (survey nos. 14 and 15). Survey no. 18, performed with the tunnel face over 40m past the line, shows a final profile with -7mm maximum displacement (towards the tunnel) occurring at approximately +12.5m offset. The offset coinciding with zero displacement is approximately +1.5m.

Vertical displacement profiles, w. are given in the bottom plot. Data from survey nos. 13, 14 and 15 (shown as open symbols) were not corrected for vertical collimation errors. The final profile calculated for survey no. 18 resembles an inverted normal curve with a maximum displacement near the tunnel centre-line of about -21mm, and decreasing displacements with offset.

### 6.2.5 Comparison of Period 2 measurements
The final measured displacement profiles determined from the four different measurement tools and techniques are compared in Figure 6.20 where the final profiles are shown for the three directions of displacement, u., v., and w. With few exceptions, the similarity between the different methods for all three dimensions and for all SMPs is better than ±1mm. Comparisons with the contractor's data are made in Chapter 7.

### 6.3 Surface monitoring for Period 3
Period 3 monitoring surveys are summarised on Figure 6.21 which shows the timings of the levelling runs, micrometer stick traverses, total station surveys and collimation measurements. The eight-month rest period between construction of the two running tunnels spanned a hot and dry summer season. At the instrumented site the ground surface was noticeably affected during this period, with local desiccation cracking appearing in areas of poor grass coverage.

6.3.1 Vertical displacements for precision levelling

Figure 6.22 shows selected settlement profiles from the 33 levelling runs (nos. 29-62) performed during Period 3; the base levels were taken as the average of surveys nos.27-29 at the end of Period 2. Time plots of settlement are not included as the response is influenced greatly by seasonal changes.

Survey nos.29, 32, 35 and 37 performed within 20 days of westbound tunnel completion show less than -1.5mm (settlement) of additional movement at offsets within +30m of the westbound centre-line. Profiles for surveys performed during the summer months of June, July and August (survey nos.41, 43, and 44) illustrate the development of time-dependent settlement plus the seasonal movements which yield localised peaks of settlement at offsets far from the tunnel centre-line. These are thought to be caused by the large trees in the vicinity of the north end of the SMP line. For survey no.44 a maximum displacement magnitude of -12.5mm is seen at +47.5m offset. At the tunnel centre-line only -5mm of movement is observed.

Survey nos.47 and 51 were in October and December of 1995 respectively. The shapes of these two profiles change little from that seen for survey no.44, but an overall increase in displacement magnitude of between 2 and 3mm is observed for all SMPs. The last survey presented in Period 3 (no.62) shows a small amount of heave relative to survey no.51 at points far from the tunnel centre-line. For survey no.62, a maximum measured displacement of -15mm (settlement) is observed at SMPs offset far from the westbound tunnel (+32.5m, +37.5m and +40m offset), with approximately -8mm of movement noted at the tunnel centre-line.

6.3.2 Transverse horizontal ground strains and displacements by micrometer stick

Twenty-two sets of strain measurements (nos. 24-45) were taken during Period 3 as indicated on Figure 6.21. Selected surveys are presented in Figure 6.23 as profiles of horizontal ground strain relative to the base span measurements at the end of Period 2 (taken as the averages of survey nos.21-24). Within the first two months after tunnel completion (to survey no.29) changes in strain are generally less than ±0.0002 (i.e. changes in span lengths of ±0.5mm). Survey nos.30 and 33 show strains developing during the summer months; peaks of both compressive and tensile strains are observed at offsets near to and far from the tunnel centre-line. The positions of the peaks compare well with those observed in the vertical displacement profiles over the same period on Figure 6.22. Survey nos.34 and 36 performed in October and December of 1995 show only small increments of strain from survey no.33. The final survey profile for Period 3 (survey no.43, completed in January 1996) shows a reduction in strain...
magnitudes for some spans.

Relative profiles of transverse horizontal displacement determined from the changes in measured spans are given on Figure 6.24. SMP no.24 located at +52.5m offset from the westbound tunnel centre-line was assumed to be stable and the changes in span lengths were summed across the SMP line (see Section 4.5.2). Survey nos.24, 28, and 29 show variations of less than ±1.5mm from the base value. These variations result from the cumulative errors for each span across the SMP line (see the monitoring results from Period 1). Survey nos.30, 31 and 32 extend over June, July and August of 1995 and show a developing displacement profile for which nearly all points are displacing northwards (positive displacement) relative to SMP no.24. The remaining surveys (nos.35, 36, 38 and 44) show varying magnitudes of displacement between surveys (±2mm at the westbound tunnel centre-line) but with only small changes in the overall profile shape. Again these variations are likely to be attributable to the cumulative effect of small changes or errors in span measurements. The final profile shows differential movements between SMP no.24 and the SMP no.3 at the tunnel centre-line of between 4 and 7mm. The distribution of movements between the two points is broadly linear, with some peaks at SMPs far from the tunnel centre-line corresponding to the local strain peaks shown in Figure 6.23.

6.3.3 3-dimensional displacements by total station

Nineteen total station surveys were made in Period 3 (nos. 18-36): details are summarised Appendix F. The timings of these surveys are shown on Figure 6.21. Survey nos. 32-36 completed in the 2 days immediately prior to construction of the eastbound tunnel are included in the Period 4 results. During Period 3 a new survey target was brought into use (see Section 4.3). Survey nos.18-25 and 29 were completed using target no.1 while survey nos. 26 and 29-36 used target no.2. Survey nos. 25/29 and 26 30 were made at effectively the same times to provide an overlap of data measurement and continuity of data analysis. The coordinates determined from survey no.17 were used as the base values for surveys during Period 3 from ST1. For surveys from ST2, the base values were taken as the coordinates determined for survey no.18: note that no vertical collimation corrections were made for this survey because only one face was used for the survey.

Displacements in three dimensions for all SMPs with time are presented on Figures 6.25 and 6.26. These data show for both station locations a general increase in the magnitude of differential displacements calculated across the SMP line from the end of Period 2 to September 1995. This is demonstrated by the increased range (or band width) of movements for a given survey which reaches 6mm by survey no.23. After September the range recorded movements becomes static, and in some cases decreases, to December. This is indicative of the seasonal influence on the ground in the summer months of June, July and August.

The data also show that the differences between each sequential survey (i.e. sense and approximate magnitude of relative change) are similar for nearly all SMPs. This behaviour may be related to larger-scale seasonal effects, or the influence of seasonal changes on the control page 80
points for the survey. It is demonstrated in Appendix B how small amounts of control point movement can result in apparent movements calculated at the SMP line.

Transverse profiles of calculated displacements across the SMP line are given as Figures 6.27 and 6.28. For simplicity, the presented data are limited to survey nos.18 and 19 completed within 12 days of tunnel completion beneath the site (ST1 only), and survey nos.25/29, and 31 performed between 222 and 235 days after westbound tunnel construction near the end of Period 3 for both ST1 and ST2. The intermediate surveys (not shown) follow the development of displacements between these two extremes.

Survey nos. 18 and 19 from ST1 show small movements in all three directions on Figure 6.27. Horizontal \( u \) displacements range between -1 and -2mm across the SMP line for survey no.19 while survey no. 18 gives a profile of much larger movements which is probably erroneous. Values of horizontal movement, \( v \), are between +2.5mm and -2.5mm across the SMP line, with somewhat poor agreement between survey nos. 18 and 19 of about ±2mm. Vertical displacements, \( w \), are less than -2.5mm and are largest near the tunnel centre-line.

Survey nos.25/29 and 31 show peaked displacement profiles for surveys from both ST1 and ST2; the peaked profiles are the result of localised seasonal effects. \( u \) displacements show movements of between -3 and -7mm at the tunnel centre-line (towards Westminster), reducing steadily to between about -3 and 0mm at +40m offset; a peak of between -4 and -8mm is seen at about +45m offset. The displacement magnitudes for profiles of \( v \) displacements from ST1 and ST2 are markedly different. However, from both station locations the profiles shapes are alike, with differential displacement of about 5mm between either end of the SMP line. Vertical \( w \) displacement profiles show peaks of settlement. The profiles from the two station locations give significantly different magnitudes of movement near the tunnel centre-line. These differences probably result from a lack of collimation correction to the base survey for ST2 (survey no.18) which manifests itself in the vertical displacement calculations for points closest to ST2 (and near the tunnel centre-line). Generally the agreement between the two surveys at each station and for all three displacement components is within ±1.5mm.

6.3.4 Comparison of Period 3 measurements

A comparison of the different methods of measurement is given on Figure 6.29 which shows the calculated displacements for the different monitoring methods at the end of Period 3. In the top plot horizontal \( u \) displacement profiles parallel to the tunnel centre-lines are given for total station surveying from ST1 and ST2: they both show similar profile forms and agree to within ±1.5mm for most SMPs across the monitoring line.

The middle plot shows the horizontal \( v \) displacements transverse to the tunnel centre-lines for the total station surveying and the micrometer stick measurements. These three independent measurements profiles which agree to within ±2.5mm for most SMPs, but are noticeably similar in shape across the SMP line. The profiles from the ST2 surveys and from the micrometer stick agree generally to within ±0.5mm.

The vertical \( w \) displacement profiles from the total station surveying and precision
levelling are given in the bottom plot. These profiles show agreement within about ±2mm at the tunnel centre-line, reducing to ±0.5mm at the far north end of the SMP line: the distribution of displacements for each independent measurement is very similar.

Generally, the variations between the different monitoring results are larger than those seen for the Period 2 monitoring. This is thought to be an effect of the seasonal changes on the structures on which the reference points are fixed. The forms of mismatch between the total station surveying and the other methods of monitoring are characteristic of such seasonal effects (see Appendix B).

6.4 Surface monitoring for Period 4
Period 4 monitoring is summarised on Figure 6.30 which shows the approximate position of the eastbound tunnel face relative to the SMP line and the timings of levelling runs, micrometer stick traverses, total station surveys and collimation measurements. During this period the tunnel advanced at an average rate of about 21m/day (~0.9m/h or nearly 1 ring/h); isolated periods of advance exceeded 1.2m/h. The results for each monitoring type are described below.

6.4.1 Vertical displacements for precision levelling
Twenty-five levelling runs were performed (survey nos. 57-81) in Period 4. Survey nos. 57-62 made on 8-Jan-96 (immediately prior to eastbound tunnel construction beneath the site) are presented separately on Figure 6.31 which shows (a) variations of measured levels from mean values of these six surveys for all SMPs and the standard deviation for survey for each level run, and (b) profiles of difference across the SMP line and the standard deviation for measurement at each SMP. The variation in level for all six surveys is less than ±0.3mm. Both the deviations for survey and the deviations for measurement are less than 0.15mm in all cases showing a slightly better precision than observed during Period 1. The averages of these six surveys were taken as the base levels for Period 4 displacements.

The development of settlements with time during eastbound tunnel construction are presented on Figure 6.32 for SMP nos.11-24 (i.e. north of the eastbound centre-line and away from the already constructed westbound tunnel). The effects of tunnel construction were first observed at survey no.68. Later surveys show increasing settlement for SMP nos.11-22; an increase in settlement rate is seen after survey no.74. Similar patterns (not shown) were also observed for SMP nos. 1-10. SMP nos.23 and 24 show negligible movement during the tunnelling: standard deviations for measurement at these points (less than 0.2mm) are similar to those observed during the stable period immediately before tunnel construction (Figure 6.31b).

Transverse profiles of vertical displacement are shown on Figure 6.33 along with a key diagram indicating the location of the shield edge at the start of each survey. The first measured displacements (survey no.68) occur with the shield edge about 10m from the SMP line. As the tunnel progresses beneath the line, the settlement trough grows in magnitude but appears to change very little in overall shape, except perhaps with some widening at the southern end of the profile. At survey no.81, when the shield is over 20m past the instrument line, a maximum
vertical displacement of -23.4mm is seen at SMP no.12 (approximately +1m offset from the eastbound tunnel centre-line). The maximum volume loss, $V_1$, represented as the volume of the perpendicular surface settlement trough, $V_s$, divided by the theoretical volume of soil excavated at the tunnel face (i.e. $\pi D^2/4$ per metre advance), is 2.9%.

Levelling data from the JLE contractor's for the monitoring lines at the north end of the park (lines F and G, see Figure 5.5) and for Imperial College monitoring on the same points are shown on Figure 6.34. The centre-line settlement magnitude at line G is -10mm, and the volume losses are estimated for both monitoring lines to be between 1.2 and 1.4%.

6.4.2 Transverse horizontal ground strains and displacements by micrometer stick

Survey nos. 40-56 were completed during the construction of the eastbound tunnel as seen on Figure 6.30. Six measurement sets (nos.40-45) performed immediately prior to the eastbound tunnel affecting the site (5-Jan-96 to 8-Jan-96) are shown separately on Figure 6.35. 6.35a shows the variation with time for all span measurements from the mean value, and the standard deviations for survey. With few exceptions the variation in measured spans between surveys is less than ±0.25mm; the standard deviations for survey are less than 0.2mm. Figure 6.35b shows the variations from the mean value as profiles of span change across the SMP line; the standard deviations for measurement at each span are also shown. Generally the deviation for measurement lies between 0.1 and 0.2mm, except for span no.18-19. The average of these six sets were taken as the base span measurements for Period 4 monitoring.

Profiles of average horizontal strain measured between adjacent spans during eastbound tunnel construction are given in Figure 6.36, along with a key diagram indicating the distance from the shield cutting edge to the SMP line for each survey. A significant amount of asymmetry about the eastbound centre-line is apparent for all profiles. A maximum average tensile ground strain orthogonal to the tunnel of 0.0009 or 0.09% is seen at +15m offset from the eastbound tunnel centre-line. The maximum average compressive ground strain measured is -0.0023 or -0.23% at +3m offset. The last two spans at the north end of the SMP line (between SMP nos.22, 23 and 24) are unaffected by eastbound tunnel construction; the variations in span measurement during tunnelling are within the range seen during the stable period immediately prior to tunnelling.

The final horizontal displacement profile due to eastbound tunnel construction, derived from the average of survey nos.55-57, is shown in Figure 6.37. The displacements were determined from integrating polynomial curves fitted independently to the measured strains on either side of the eastbound tunnel centre-line (see Section 4.5.2) and assuming zero average strains (and displacements) beyond the north end of the SMP line. A maximum displacement towards the eastbound centre-line of 11mm was determined from the final strain measurements for the south half-trough at -10m offset; a slightly smaller movement of -8mm towards the tunnel centre-line was calculated north of the eastbound tunnel at approximately +8m offset.
6.4.3 Parallel horizontal displacements by collimator tool

Three collimation surveys were made during Period 4; survey no. 10 was completed just prior to tunnel arrival at the site, survey no. 11 was when the shield was almost immediately beneath the SMP line, and survey no. 12 when the shield was more than 20m past the SMP line. Measurements from survey no. 10 were taken as the base values for Period 4 displacements.

Profiles of horizontal displacements, \( u \), parallel to the tunnel centre-line are given on Figure 6.38. For survey no. 11, a maximum displacement of approximately -5.5mm (towards Westminster) was observed when the shield cutting edge was about 2m from the SMP line. The profile appears asymmetrical with generally larger displacements magnitudes measured at SMPs south of the eastbound tunnel (i.e. nearer the previously constructed westbound tunnel). Due to a minor (but consistent) error in setting the line of sight, measurements on SMPs at offsets greater than \(+15m\) were not possible for all surveys. Survey no. 12 made after the tunnel had passed beyond the site shows a final displacement profile with calculated movements generally less than \( \pm 1mm \).

6.4.4 3-dimensional displacements by total station

Survey nos. 33-37 being in the week prior to the tunnelling affecting the site were used to establish base coordinates of each SMP; the measurements for these are shown on Figures 6.39 and 6.40 for the surveys from ST1 and ST2 respectively.

Figure 6.39(ST1)

In the top plot, variations in the \( x \) coordinates from the mean values are shown as profiles across the SMP line for surveys from ST1. The maximum variation in calculated coordinates for survey nos. 33, 35 and 36 is about \( \pm 2mm \). The standard deviations of measurement range between 0.5mm to slightly over 2mm, and tend to increase linearly with distance from the total station location. Survey no. 34 shows a large difference from the other three surveys and is omitted from the mean value and deviation calculations. The average \( x \) coordinates for survey nos. 33, 35, and 36 are taken as the base value for Period 4 from ST1.

The middle plot shows profiles of differences from the calculated mean \( y \) coordinates. The differences for nearly all SMPs are less than \( \pm 0.5mm \). The standard deviations for measurement are less than 0.4mm, except for SMP no. 24 at the end of the monitoring line. The base values for the \( y \) coordinates were taken as the average for survey nos. 33, 34, 35, and 36.

The bottom plot presents the differences from the mean \( z \) coordinates. For all SMPs the differences between surveys are less than \( \pm 1mm \). With few exceptions the standard deviations for measurement for all SMPs are less than 0.5mm. The average \( z \) coordinates for survey nos. 33, 34, 35, and 36 were taken as the base value for Period 4 monitoring.

It is worth noting here for clarity that the \( x \) coordinate is more sensitive than the \( y \) coordinate to errors sighting onto the reference points. Thus, a survey which shows errors in the \( y \) profile does not infer that the \( y \) profile is also in error (and vice versa).
The top plot profiles of the differences from the mean values of \( x \) coordinates are shown for survey nos. 34, 35, and 37. The profiles show differences of between \( \pm 3 \) and \( \pm 8\text{mm} \), increasing linearly with distance from the instrument location. Calculated standard deviations for measurement also vary between 3 and 8\text{mm}. Survey no. 33 shows a large jump in calculated differences from the mean values (over 12\text{mm}) between two adjacent SMPs and is omitted from the mean value and deviation calculations for this reason. The average \( x \) coordinates for survey nos. 34, 35, and 36 were taken as the base values for surveys from ST2.

Profile of differences from the mean value for the calculated \( y \) coordinates are given in the middle plot. A maximum range of about \( \pm 3\text{mm} \) is seen for survey nos. 34, 35, and 37. The standard deviation for measurement ranges between 2 and 3\text{mm}. The standard deviations for survey (not shown) are all about 0.5\text{mm}. Survey no. 33 shows a jump in measurement of about 4\text{mm} between adjacent SMPs (at the same position as seen in the top plot) and is therefore excluded from the mean value and deviation calculations. The average \( y \) coordinates for survey nos. 34, 35, and 36 were taken as the base values.

The bottom plot presents profiles of differences in calculated \( z \) coordinates from the mean values. Survey nos. 33, 34, and 35 agree to within \( \pm 1\text{mm} \) for all SMPs; the calculated standard deviations for measurement are generally less than 0.7\text{mm}. Survey no. 37 has a much larger difference from the mean value (up to 3\text{mm}) and is therefore excluded from the mean value and deviation calculations. The average \( z \) coordinates for survey nos. 33, 34, and 35 were taken as the base values.

The \( x \) and \( y \) coordinates show poor precision in comparison with the surveys of Period 1. Most of the Period 4 surveys were performed at night, when errors for sighting on to the control points are more likely; the weather was also extremely unfavourable. The profiles of variation from the mean values calculated are typical of what would be expected for such errors (see Appendix B).

Survey nos. 38-48 were completed during eastbound tunnelling construction and are presented on Figures 6.41 and 6.42 as transverse displacement profiles in three dimensions for surveys from ST1 and ST2 respectively. At the top of each figure is a key diagram indicating the relative position of the SMP line from the shield cutting edge for each survey. Profiles are adjusted by the average calculated displacements at SMP nos. 22-24 (see Section 6.1.5) which are shown by the levelling and horizontal strain measurements to have moved very little.

**Figure 6.41 (ST1)**

Horizontal displacements parallel to the tunnel axis (\( u \)) presented in the top plot were not observed until the tunnel face was approximately 7.5\text{m} from the SMP line (survey no. 40). Prior to this the profiles for survey nos. 38 and 39 show apparent movements of up to 2.5\text{mm} at the SMPs furthest from the instrument position, but with almost no displacements at the tunnel centre-line. With the tunnel face between 0 and 2.5\text{m} past the section (survey nos. 42 and 43), movements of between -6.5 and -8\text{mm} (towards Westminster) are observed at the eastbound...
centre-line. Displacement magnitudes then begin to reduce from survey no.43. The profiles calculated for survey nos.47 and 48 when the tunnel face is over 20m past the SMP line show approximately linear displacement profiles across the site, increasing in magnitude to between ±4mm at the SMPs furthest from the instrument position.

Horizontal displacements orthogonal to the tunnel axis (v) are given in the middle plot. Survey nos.36, 38 and 39 show movements of between 0 and -2mm at the SMPs furthest from the station location, decreasing to zero northwards to the instrument position. Small construction-induced displacements are first seen for survey no.40 (x=+7.5m). The profiles grow in magnitude during survey nos.42-46, but little change in movement distribution is seen. The final construction profiles for survey nos.47 and 48 (when the tunnel was over 23m past the section) show maximum displacements of -8mm (towards the tunnel) at about +9m offset for the north-half of the site. A larger displacement of +13mm (towards the tunnel) was observed at -11m offset (south of the eastbound tunnel).

Vertical displacements presented in the bottom plot show a growing subsidence trough with tunnel progress. Survey no.42 performed when the shield cutting edge is immediately beneath the instrument line shows about -7.5mm of displacement (settlement) near the tunnel centre-line, and a roughly normal profile of displacements about the centre-line. The final profile has changed very little in shape and shows a maximum displacement of approximately -23mm (settlement) near the tunnel centre-line.

Figure 6.42 (ST2)

Horizontal movements parallel to the tunnel drive (u) are first observed when the tunnel face is about 6m from the SMP line (survey no. 40) and continue to grow in magnitude until the tunnel is 1m past the monitoring section (survey no.43). At this position a maximum displacement of -6mm (towards Westminster) is observed at the tunnel centre-line. The movement trend then reverses for survey no.45 with the maximum displacement reducing to about -2.5mm. Profiles for surveys 46, 47, and 48 made when the tunnel was between 17 and 32m past the SMP line show displacements up to +3mm (towards Green Park) at the tunnel centre-line, and a range between the three surveys of ±1.5mm.

Displacements horizontal transverse to the tunnel centre-line. v. are first observed for survey no.40. On the middle plot, the profile shape is seen more clearly at survey no.43 when the tunnel face was only just past the instrument line. The overall displacement magnitudes increase across the SMP line for surveys 44-46. Survey nos.47 and 48 performed with the shield over 23m past the section show a maximum displacement of -9mm (towards the tunnel centre-line) at approximately +9m offset (north of the tunnel); a slightly larger movement of +11mm (towards the tunnel centre-line) is noted at -11m offset.

Vertical displacements, w, are shown in the bottom plot. Survey no.40, performed with the shield edge only 6m from the SMP line, shows movements of up to -2mm (settlement). By survey no.43 the tunnel face is just past the instrument line and about -12mm of displacement is seen at the centre-line; the movement profile is roughly a normal distribution, and slightly
asymmetrical on either side of the tunnel axis. The displacements at nearly all SMPs grow for
survey nos.44-46 with little change in profile form. The final profile due to eastbound
collection shows a maximum displacement of -24.5mm (settlement) at the tunnel centre-line.

6.4.5 Comparison of Period 4 surface measurements
A comparison of the different methods of monitoring is made on Figure 6.43 which shows the
final transverse displacement profiles in three dimensions from measurements presented in
Sections 6.4.1 to 6.4.4. Profiles of \( u \) displacements horizontal and parallel to the tunnel centre-
line are presented in the top plot for the average of total station survey nos.47 and 48 from both
ST1 and ST2, and for the collimation tool survey no.12; all three profiles show similar
displacement forms and agreement to within ±1.5mm for most SMPs across the monitoring line.

The middle plot shows the \( v \) displacements horizontal and transverse to the tunnel centre-
line for the total station results from ST1 and ST2, along with the final displacement profile
from the micrometer stick measurements. These three independent results show differences less
than ±0.5mm for most SMPs.

The vertical \( (w) \) displacement profiles determined from the total station measurements
from ST1 and ST2, and from the precision levelling, are given in the bottom plot. The three
profiles show agreement to within ±1mm.

6.5 Surface monitoring for Period 5
Monitoring events after eastbound construction are summarised on Figure 6.44 which shows the
timing of levelling runs, micrometer stick traverses, and total station surveys against date and
days from completion of the eastbound tunnel construction beneath the instrumented section.
The collimation tool was not used after Period 4. The presented data are limited to the first year
after tunnel completion; measurements are on-going at the site but are not included in this thesis.

6.5.1 Vertical displacements for precision levelling
Sixteen levelling runs (survey nos.85-100) were made in the first year after construction of the
eastbound tunnel beneath the site. The development of settlement with time is illustrated in
Figure 6.45 for three monitoring points: SMP no.3 on the westbound centre-line, SMP no.12
near the eastbound centre-line, and SMP no.22 at the northern end of the instrument line.
During eastbound construction SMP no.22 settled less than 1mm. SMP nos.3 and 12 show
similar forms of displacement growth with time. Larger settlements are observed at SMP no.12
at the eastbound tunnel centre-line. SMP no.22 shows a slower rate for the first 100 days after
construction. Between survey nos.95 and 97 (spanning the summer months of June to
September 1996) the settlement at SMP no.22 jumps from -7.5mm to -25mm; SMP nos.3 and
12 show a smaller but noticeable increase in displacement rate over the same period; this appears
to be a seasonal effect. From Oct-96 to Jan-97 (survey nos.97-100) SMP nos.3 and 12 continue
to settle (at a slower rate than observed during the previous summer) while SMP no.22 show
some small heaving movement.
Selected transverse profiles of vertical displacements are presented on Figure 6.46 with a key diagram indicating the number of days after tunnel completion for each survey. The profiles are quite smooth across the SMP line showing a maximum movement midway between the eastbound and westbound tunnels (about -8m offset) until survey no.96, where peaks of settlement are seen towards the northern end of the SMP line at offsets greater than +10m. The settlement profile for about 1 year after eastbound construction (survey no.100) shows movements larger than -16mm (settlement) for all SMPs. -24mm of settlement at the eastbound tunnel centre-line, and a maximum displacement of -27mm at an offset of +11.5m. The patterns of peaks and troughs broadly correspond to those that developed during Period 3 (see Figure 6.22 during the summer months).

6.5.2 Transverse horizontal ground strains and displacements by micrometer stick
Twelve sets of micrometer stick readings (survey nos. 57-68) were performed during the 500 days after completion of the eastbound tunnel. Figure 6.47 shows measured horizontal strains with time for selected spans across the SMP line: Survey nos. 57 to 61 performed in the first 20 days show changes in strain of ±0.0001. With time the strain magnitudes oscillate at all spans, a result which is consistent with seasonal variations.

Transverse profiles of horizontal strain for selected surveys are given on Figure 6.48. Apart from the isolated peaks, the measured strains are generally compressive with magnitudes between -0.0003 and -0.0005 (-0.03 and 0.05%). The localised peaks develop between Apr-96 and Jul-96 (survey nos.62 and 64) during the summer months and again correspond well to the peaks seen from levelling.

Relative profiles of transverse horizontal displacement are given on Figure 6.49. SMP no.24 located at +31m offset from the eastbound centre-line is assumed to be stable during the whole period and the displacement profiles were determined from summing the measured changes at each span along the SMP line. Profiles for survey nos. 57-60 are broadly similar in form to those observed during eastbound tunnel construction with up to 2mm of movement on either side of the tunnel (towards the centre-line). The pattern suggests some small but continued horizontal movement after tunnel construction.

Survey no.62 shows the cumulative effect of the small compressive strains measured; the calculated movements relative to SMP no.24 are directed predominantly northwards and up to 5mm of movement is observed near the westbound centre-line. Survey nos. 64-67 performed between Jul-96 and Jan-97 all show roughly linear displacement profiles across the site. Differential movements range between 10.5mm and 12.5mm across the SMP line. The increase at survey 68 (May-97) to 14mm of differential displacement may represent early seasonal influences for the summer of 1997.

6.5.3 3-dimensional displacements using total station
Seven total station surveys (nos.48-54) were made to November of 1996. Figures 6.50 and
6.51 show the measured displacement profiles in three dimensions for surveys done from ST1 and ST2 respectively. The averages of coordinates determined from survey nos.46 and 48 were taken as base coordinates for Period 5.

**Figure 6.50 (ST1)**

The *u* displacement profiles given in the top plot for survey nos.48 and 49 (completed within 10 days after tunnel completion) show movements of +2mm at the eastbound centre-line reducing to near zero at either end of the SMP. Profiles for survey nos.50 to 54 are all similar in shape but show increasing magnitudes with time. At survey no.54 (nearly 1 year after eastbound tunnel construction) the profile shows movements increasing linearly from near zero at the northern end of the SMP line to about +6mm at the eastbound centre-line. South of the eastbound tunnel, the displacements vary between +3 and +6mm to the end of the SMP line.

Horizontal displacements transverse to the tunnel (*v*) given in the middle plot show movements of 2mm on either side of the tunnel towards the centre-line for survey nos.48 and 49. Survey no.50 shows +5mm of movement (towards the centre-line) at -21m offset and about -2.5mm movement (towards the tunnel) at 16.5m offset. Survey nos.51-54 all show linear distributions of displacements across the SMP line with about +8mm of movement at the south end of the SMP line, and -3mm of displacement at the north end.

The vertical displacement profiles, *w*, show increasing movements across the site with time; some peaks of settlement develop during survey nos.51-54 in the north half of the SMP line. For survey 54 a maximum movement of -22m is seen at 11.5m offset and settlement at the eastbound tunnel centre-line is about -17mm.

**Figure 6.51 (ST2)**

For survey no.48 the profile of *u* displacement given on the top plot shows movements of ±1mm across the SMP line. The profile for survey no.49 shows a linear trend from +1mm at the southern end of the SMP line to +5mm at the north end furthest from ST2. Survey nos.50-54 show varying displacement magnitudes but with similar profile shapes. The displacement profile for survey no.54 is essentially bi-linear, increasing from +1mm at -26.5m offset to +5mm at the eastbound centre-line, and then decreasing to about +3mm at the northern end of the SMP line.

Transverse horizontal displacement profiles, *v*, show movements of ±2mm from the base coordinates for survey nos.48 and 49 broadly similar to those seen from ST1. At survey no.50 a linear displacement profile is observed with +5mm of displacement seen at the westbound centre-line, and -5mm of displacement at +28.5m. The profiles for survey nos.51,53 and 54 show slightly larger movements but are similar in shape. Magnitudes agree to within ±1.5mm for these surveys at most SMPs.

Vertical displacements (*w*) presented on the bottom plot show some additional settlement in the first days after eastbound construction (survey nos.48 and 49); -3mm of movement (settlement) at the eastbound tunnel centre-line, decreasing with offset from the centre line. Survey 50 shows an overall increase in settlement across the SMP line, and marks the beginning
of localised peaks of settlement in the northern half of the site. These become more pronounced for survey nos. 51, 53, and 54. Survey no.54 shows a maximum settlement of -25mm at offsets of +11.5m and +26.5m. About -18mm movement is seen at the westbound centre-line (-21.5m offset) and about -21mm of displacement is noted at the eastbound centre-line (0m offset).

6.5.4 Comparison of Period 5 surface measurements
The results for different monitoring methods are compared in Figure 6.52 for all three displacement coordinates. The top plot shows the total station survey profiles of $u$ displacements in the $x$ direction determined from survey no.54 from both ST1 and ST2. The two profiles agree to within ±1.5mm for all SMPs.

The middle plot shows the transverse displacement profiles, $v$, for survey no.54 from ST1 and ST2 and the displacement profile calculated from micrometer stick survey no.66 (taken on the same day). The two total station profiles show agreement of ±1mm for most SMPs. The micrometer stick profile has a very similar shape, but is shifted about +4mm from the average of the total station profiles. These differences almost certainly arise because (a) SMP no.24, at the north end of the SMP line, which is assumed to be stable or fixed in calculating the micrometer stick profile, has itself moved; and (b) the reference points on the nearby buildings have moved as a result of seasonal variations.

The vertical displacement, $w$, profiles for survey no.54 from ST1 and ST2, and the precision levelling survey performed on the same day (no.98) are given in the bottom plot. The precision levelling is calculated relative to the deep extensometer, Dx7, thus removing potential seasonal effects on the local levelling datum. The data from ST2 shows close agreement (±1mm) with the levelling data, but the data from ST1, although similar in profile shape, differ in magnitudes by about +5mm (i.e. less measured settlement). This implies settlement of structures on which the total station reference targets are affixed, which would give a profile of apparent heave (or less settlement) at the SMP line.

6.6 Subsurface displacement monitoring results in Period 1
Measurements taken during Period 1 are used to evaluate the precision and the performance of both the rod extensometer measurements and the electrolevel inclinometer readings. These measurements are used as base readings for all measured quantities, and to assess the performance of the monitoring techniques and instruments used.

The measurements are reviewed here utilising two statistical measures of precision: the standard deviation for measurement and the standard deviation for survey as described at the opening to Chapter 6.

6.6.1 Rod extensometers
Figure 6.53 illustrates a typical variation in measurement observed for the twelve sets of extensometer readings made during Period 1. by presenting the differences in calculated anchor
levels from the mean values for the eight anchors installed in extensometer Ax. The anchor level calculations incorporate the precision levelling measurement onto the reference head as described in Section 4.8.2.

**Figure 6.53a** shows the differences observed with time and demonstrates that the variations in calculated levels are well within the ±0.3mm band observed for levelling measurements in Period 1 (Section 6.1.1). Standard deviation for survey is typically less than 0.05mm for a single borehole, reflecting the performance of the extremely precise dial gauge. **Figure 6.53b** presents the differences as vertical profiles with depth below ground level. Variation magnitudes between surveys for all anchors is similar and the standard deviations for measurement are all about 0.1mm.

From these plots, it is inferred that the measurement of displacements relative to the reference head (i.e. using the dial gauge only) are very precise, but it is the levelling on the reference head that controls the overall precision of the calculated anchor displacements.

### 6.6.2 Electrolevel inclinometers

Period I monitoring of the inclinometers comprised nine sets of manual readings, followed immediately by computer controlled auto-logging at the end of the period in preparation for westbound tunnel arrival. The change in measured response with time varied between instruments and depends on the monitoring techniques used, and therefore each electrolevel must be addressed individually in terms of its response for both monitoring systems.

**Figure 6.54a** presents the manually measured responses from the 16 electrolevels in inclinometer hole Ai during Period 1. The plot demonstrates that there are apparent rotations for many of the electrolevels (assuming that the ground is stable during this period). The apparent rotation rate for each electrolevel was calculated using a linear regression of the last four measurements and are tabulated on the figure. The sense or sign of the rotations are coincidently similar for almost all of the electrolevels in this hole.

Auto-logging began five days prior to tunnelling beneath the site (22 April 1995) with measurements taken at 15-minute intervals. Apparent rotations with time for the electrolevels in inclinometer hole Ai are presented on **Figure 6.54b** (note the different scale when comparing with **Figure 6.54a**). Again the rates of apparent rotation for each electrolevel were calculated using linear regression over the range indicated and are tabulated on the figure.

The calculated instrument drifts for the two monitoring systems yield apparent rotation rates which are different in magnitude, and often different in direction of rotation. There is no consistent correlation between the two monitoring techniques. The rates of rotation are small, but the method of analysis for determining displacements is a summation of changes along the inclinometer length (see Section 4.9.2). This can produce profiles of apparent movements with large magnitudes, particularly over long-term horizons.

In order to minimise any apparent displacements due to drift being included within the measured response during tunnel construction, the base readings for auto-logging were taken
as near as possible to the time when the tunnel construction first began to influence the control site.

6.7 Subsurface displacement monitoring for Period 2

The approximate timing for all extensometer surveys performed during Period 2 are presented on Figure 6.55 along with the relative position of the shield cutting face to the instrument lines perpendicular to the tunnel centre-line. Two lines of tunnel progress are shown which correspond to the two extensometer lines shown on the site plan on Figure 5.2. The electrolevel inclinometers were auto-logged using a computer at 15-minute intervals; vertical profiles of horizontal displacement presented in this section are selected to coincide with the extensometer surveys by matching the tunnel face position relative to the instrument lines.

6.7.1 Rod extensometers

The development of anchor displacements are summarised on Figures 6.56 to 6.66 which show the variations in calculated vertical movements, $w_{anchor}$ with advancing tunnel face position for each extensometer hole.

The response at extensometers Ax-Cx (nearest the westbound tunnel centre-line, Figures 6.56-6.58) indicate that the tunnel influence reaches between 9 and 12m in front of the shield cutting edge. For each of these extensometers, the anchors positioned at levels above the tunnel shoulder level initially move downward together with very little differential movement. As the shield edge moves very near to, then beneath and past the instrument line, the response of the anchors nearest to the tunnel diverge from the anchors at upper levels by showing much larger and faster movements vertically downward. Once the shield is past the instrument lines, the displacement trends for all anchors slow markedly. At shallow depths the rates of settlement are slightly larger than at anchors nearer the tunnel. The maximum movement on the centre-line at the ground surface (Figure 6.57) is about -18mm (settlement); this is about 2mm less than observed for SMP no.3 also on the centre-line, and might indicate varying conformance between the monitoring methods. The settlement near the crown (Bx8) is -43mm, giving a ratio of maximum surface to maximum crown settlement ($w_{max}/w_c$) of about 0.4. This agrees well with the field data for tunnels in London Clay presented on Figure 2.12.

Extensometers Dx and Ex (Figures 6.59 and 6.60) further from the tunnel centre-line show similar movement patterns with tunnel face position. However at Dx the reference head and all anchors to -17.2m move in unison for the entire tunnelling episode. The deeper anchors (-22.5m and -27.5m) are seen to move slightly less. At Ex the reference head and anchors above -12.9m move together while the deeper anchors (between -15.1m and -27.4m) move noticeably less. Movements at extensometers Fx to Hx (Figures 6.61 to 6.63) are similar for all anchors (within ±0.3mm) but generally show a slight decrease in displacement magnitude with depth from the reference head. Extensometers Ix, Jx and Kx (Figures 6.64 to 6.66) show only very small displacements (less than ±0.5mm) during the westbound tunnel construction.

Deep anchor Cx8 shows a small amount of heave (+2mm) during westbound tunnel
construction on Figure 6.58. The other two deep anchors (Ax4, Figure 6.56 and Dx8, Figure 6.59) were more stable during the excavation and oscillated between 0 to +0.7mm.

Vertical profiles of vertical displacement in each extensometer hole are shown on Figure 6.67. For each hole, the displacement development can be traced in sequence, increasing in magnitude as the tunnel face moved beneath the instrumented lines. Plotted in this form, the uniformity of displacements with depth prior to the tunnel reaching the instrument line at extensometers Ax - Cx is seen (solid and open triangles), along with the development of differential anchor displacements near the tunnel extrados thereafter.

Horizontal profiles of vertical displacement at different subsurface horizons are given on Figure 6.68. The key diagram indicates the position of the tunnel face relative to the instrument line containing extensometers Bx-Hx. The data for Ax (at -4m offset), which is 4m closer to the tunnel face, tends to show more displacement during trough development at all levels considered, until the tunnel face is well past the monitoring lines. Again, the development of settlement follows an ordered sequence, increasing with tunnel progression beneath the instrument line. The figure visually demonstrates both the increasing narrowness of the transverse trough shape and increasing displacement magnitudes with nearness to the tunnel crown.

6.7.2 Electrolevel inclinometers
The measured electrolevel responses with time during westbound tunnel construction are given on Figures 6.69-6.79 for inclinometers Ai to Hi; the distance between the instrument line and the tunnel face is also shown with time on each plot. A key diagram also indicates the sense of rotation relative to the coordinate system given on Figure 2.8.

At inclinometers Ai and Ci (Figures 6.69 and 6.72) the electrolevels located near the tunnel axis and crown levels (electrolevel nos. Ai3-Ai5 and Ci3-Ci5 as indicated on key diagrams) typically show a two-stepped response. Rotations begin when the shield cutting edge lies between 7 and 10m from the inclinometer line. These increase rapidly at first, but slow down when the shield edge is approximately 5m from the line.

A second phase of response begins about 1-2m before the cutting edge passes the instrument line and is marked by increased and steady rate of rotation for nearly all electrolevels in the inclinometer tube, until the shield edge is about 6m past (at which point a ring at the instrumentation plane is erected and expanded). The response of the electrolevels beyond this point reach a pseudo-plateau where rotations appear to stop at most positions. The other electrolevels in boreholes Ai and Ci located above and below show less pronounced steps and smaller rotation magnitudes. It is noted that the response of electrolevel no.Ai4 changes direction in the second phase of response and does not compare well with the response observed for electrolevel no. Ci4 at the same level on the other side of the tunnel. Comparing adjacent electrolevels in inclinometer Ci at the level of the tunnel crown (electrolevel nos.Ci4 and Ci3 on Figure 6.72 which span about 3m) shows the variability in measured rotations in this area; +5mm/m rotation (~1000 bits) is seen at electrolevel no.Ci4 while at no.Ci3, -5mm/m rotation (-
1000 bits) is recorded. Clearly the magnitude and sign of measured responses are closely controlled by the exact positions of each electrolevel in the tubing relative to the tunnel.

At inclinometers Di-Gi (Figures 6.73 to 6.77), a two-stepped response is observed for many electrolevels, particularly those at shallow depths: some of the deeper electrolevels in Ei, Fi and Gi are below the zone of influence for westbound construction and show virtually no rotation during tunnel construction. Electrolevels in inclinometer Hi are oriented askew to the parallel and transverse directions, but show similar patterns of response to those observed in the other boreholes.

At inclinometer Bi the electrolevels are oriented both transverse (B1i) and parallel (B2i) to the tunnel axis. The transverse electrolevels (Figure 6.70) all measured small rotations (less than about 0.2mm/m) in the same direction; several of these exhibit a two-stepped response but do not appear to level off as sharply as other electrolevels when the shield has passed the instrument line. Electrolevels in inclinometer B2i (oriented parallel to the tunnel drive) show varied forms of response as presented in Figure 6.71. Near the ground surface the response for electrolevel no. Bi13 is initially positive with rotation counter-clockwise implying tilt of the inclinometer toward the advancing tunnel face. The sense of rotation reverses as the tunnel face moves past the section.

At the bottom of the inclinometer B2i, electrolevel no Bi-1 (positioned less than 2m above the tunnel crown) shows a large and rapid rotation when the tunnel is between 10m and 5m from the instrument line. Rotations continue to grow at a slower rate as the tunnel passes the instrument, until it is about 20m past the section.

Vertical profiles of relative horizontal displacement calculated from the measured rotations are given on Figures 6.80: the bottom of each inclinometer borehole is assumed stable (i.e. no displacement at the bottom, see Section 4.9.2 on interpreting inclinometer measurements). The development of large displacements towards the excavation at and near the westbound axis level (z=31m) are clearly seen at inclinometers Ai, Ci and Di. The spiked profiles are representative of the variable rotation magnitudes and directions measured near the tunnel, and the problems associated with extrapolating measured tilts over large gauge lengths.

Boreholes Ei-Hi show movements in the upper 7.5m of inclinometer (i.e. in the overlying gravels and alluvium) but only very small movements at horizons in the London Clay. B2i shows large relative displacements focused at the tunnel crown, reflecting the large measured rotations of electrolevel no. Bi-1 shown in Figure 6.71.

6.8 Subsurface displacement monitoring in Period 3

Figure 6.81 shows the timings of the subsurface displacement surveys with time along with a typical response for an SMP near the westbound tunnel. 24 extensometer traverses (survey nos. 26-49) and nine manual electrolevel inclinometer reading sets (survey nos. 11-19) were performed during Period 3. For the extensometers, the average dial gauge readings and calibrations measurements for survey nos. 22-24 completed at the end of Period 2 were chosen.
as the datum for Period 3. All vertical changes are made relative to extensometer anchor Dx7, which is assumed stable for Periods 3-5.

6.8.1 Rod extensometers

The change in calculated anchor levels from the end of westbound tunnel construction are shown on Figures 6.82-6.92 with time for all extensometers. Generally the rates of response are seen to slow with time at all anchor positions reflecting the equilibration of pore pressure and stress changes near the tunnel. Some comments on specific extensometers are given below.

- At extensometers Ax and Cx to either side of the tunnel (Figures 6.82 and 6.84), all anchors above tunnel shoulder level (i.e. excluding the deep anchors Ax4 and Cx8) initially move in identical patterns. With time some small differential movements between anchors begin to develop. The deep anchors positioned below the tunnel axis show small amounts of heave (between +1.5 and +2mm).

- At Bx (Figure 6.83) the anchors within one tunnel diameter above the tunnel crown (nos. Bx6-Bx8) show a more rapid response for the first 110 days than other anchors. However, between survey no.33 (August 1995) and the end of Period 3 the rates of response are similar over the entire extensometer depth.

- Anchors in Dx-Hx (Figures 6.85-6.89) show varying magnitudes of response but follow similar patterns of movement with time as for Cx.

- Extensometers lx, Jx and Kx (offset between 40m and 52m from the westbound centre-line, Figures 6.90-6.92) highlight the localised seasonal effect on the ground at shallow depths. Each of these extensometers comprises a single anchor positioned at about -5m. The traces of displacement with time for all three anchors are similar in form and magnitude and show only small movements during Period 3. In contrast, the traces of the reference head displacements range from -5 to -15mm (settlement) showing a high degree of variability between positions.

Profiles of vertical displacements with depth at each extensometer hole are seen on Figure 6.93 for selected surveys during Period 3. The movements are nearly constant with depth at Ax-Dx for anchors to above -17.5m: the reference heads show seasonal influence. Compared with the displacements at lesser depths, larger movements are observed near to and above the tunnel crown at Bx.

Transverse profiles of vertical displacements presented on Figure 6.94 show the lateral distribution of movements at different subsurface horizons. The limit of the seasonal influence is clearly demonstrated by the absence of peaks at z = -5m when comparing the profiles at z = 0m (reference head). Again the surface profiles agree broadly with the surface monitoring results presented in Figure 6.29. 250 days after tunnel completion, the profiles show about -6mm of displacement at the tunnel centre-line and -8mm at about +10m offset from the centre-line, for depths between -5 and -17m. At z = -22.5m the maximum is seen at +4m offset, and
at -27.5m (only 1m above the tunnel crown) the maximum movement (exceeding -10mm) is seen at the tunnel centre-line.

6.8.2 Electrolevel inclinometers

It was concluded through analysis of the auto-logged results at the end of Period 2 and the manual measurements made during Period 3 that a shift in the instrument drifts occurred on changing the monitoring systems. This change between systems was marked by changes in the time-dependent rotations, and were also accompanied by changes in the apparent rotation direction for some electrolevels. It was concluded that time-dependent horizontal displacements after tunnel construction could not confidently be estimated because of these effects.

However, comparison between the measured rotations immediately after tunnel construction and rotations estimated at each electrolevel position from the borehole mapping results (performed more than two years after westbound tunnel construction and described in Appendix D) give a strong indication that very little differential horizontal movement occurred after tunnel construction near the tunnel extrados (see Figure 7.26 in Chapter 7).

6.9 Subsurface displacement monitoring for Period 4

Figure 6.95 presents the timing of extensometer surveys during Period 4 in relation to the eastbound tunnel progress. The two lines of tunnel position shown correspond to the two extensometer lines shown on Figure 5.2. The electrolevels were auto-logged during this period and profiles of horizontal displacement presented in this section are selected to coincide with the extensometer surveys by matching the tunnel face position relative to the instrument lines.

6.9.1 Rod extensometers

The development of subsurface vertical displacements with eastbound tunnel progress are summarised on Figures 6.96 to 6.106 which show the changes in anchor level with tunnel face position for each extensometer. The average levels and rod measurements for survey nos.47-49 (at the end of Period 3) performed 1 day before tunnelling beneath the site were chosen as the base values for Period 4 displacements.

Data from extensometers Ex (-5.5m offset), Fx (centre-line of eastbound tunnel) and Gx (+4.5m offset) indicate that the zone of influence in advance of the shield extends between 9m and 11m (Figures 6.100-6.102). From this position to where the shield is about 5m from the section (survey no.54) the displacements magnitudes at Ex, Fx and Gx do not exceed -2mm (settlement) and are similar at all levels above the eastbound tunnel crown (within ±0.3mm); slightly more movements are consistently seen at the shallower anchor depths. At survey no.55 the displacement magnitudes increase rapidly near the tunnel crown (anchors Fx4-Fx6) as the shield approaches the instrument line. The same is seen for anchors near the tunnel crown level at Ex and Gx at survey no.56 as the shield edge moves through the section. Large differential movements develop between anchors, with generally larger movements observed nearer the tunnel for levels above the crown. At survey no.56 the observed movement at anchor Fx6

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(closest to the tunnel crown) is over twice that recorded at the reference head. Survey nos. 57 and 58 show continued growth in both the overall movement magnitudes and the differential movement between anchors. Once the shield is past the extensometer line and the ring built and expanded at the section the rates of displacement are seen to reduce. Movement attributable to construction activities is considered to stop by the time the shield is 20m past the line; some additional movements occur (between 0.1 and 1.3mm, depending on the location of the anchor) during the 2 days after tunnel construction. Anchors Gx6, Ex6 and Ex7 positioned below the tunnel axis show heaves of between +1 and +2mm during tunnelling.

The remaining extensometers (Ax-Dx and Hx-Kx) show broadly similar movement patterns with tunnel face position to those of Ex and Gx. The maximum displacements, however, are generally seen at anchors nearest the ground surface, with movement magnitudes generally reducing with depth. Data from extensometer Kx show only very small displacements (less than ±0.3mm) during construction of the eastbound tunnel.

Vertical profiles of settlement are presented for extensometers Ax-Hx on Figure 6.107. Nearest the tunnel, the profiles at Ex, Fx and Gx (Figure 6.100 to 6.102) show uniform displacements with depth prior to the tunnel reaching the instrument line (to survey no. 55). As the tunnel passes the instrument line the anchors positioned near the tunnel crown profiles in these three holes show larger and more rapidly accumulating displacements. On the centre-line (Fx) the maximum surface displacement is about -23.5mm while the maximum movement near the tunnel crown is about -41mm, giving a ratio of \( w_{\text{max}}/w_c \) of about 0.57. This value agrees well with the case history data presented on Figure 2.12 (cover to diameter ratio of about 3.7).

Anchors near the previously constructed westbound tunnel and below the eastbound tunnel axis (boreholes Ax, Bx and Cx, Figures 6.96 to 6.98) show movements of between -1 and -2mm (settlement). Interestingly, the profile with depth at Bx shows a minimum displacement at about -17m, with larger settlements both above and below this level although the displacements magnitudes are small (less than 2mm).

Transverse profiles of vertical displacement at different subsurface horizons are given on Figure 6.108. The key diagram indicates the position of the tunnel face relative to the instrument line containing extensometers Bx-Hx. Extensometers Ax and Ix-Kx are about 4m closer to the tunnel face than the line containing Bx-Hx, and may appear to have proportionately larger settlements for profiles shown during tunnel advance. The figure visually demonstrates a narrowing of the transverse subsidence trough shape with depth, an increase in displacement magnitudes with depth, and asymmetry of the settlement profiles at all levels above the tunnel crown.

6.9.2 Electrolevel inclinometers

During westbound tunnel construction, the automatic logging set-up utilised the ‘slow’ multiplexers which gave electrical isolation between each electrolevel circuit. The arrangement for the eastbound tunnel monitoring included ‘fast’ multiplexers which power all electrolevels on a common circuit. Prior to eastbound tunnel arrival, it became apparent that the electrolevels
in inclinometers Ai, Bi, and Ci were drifting substantially. Temporary disengagement of the problem instruments from the system resulted in response change at all other electrolevels logged on the same multiplexer, implying a current leak between the electrolevel cabling. The fast multiplexers were replaced by the slow type used for westbound construction just before tunnel arrival. The effects of this are discussed below.

Inclinometer responses with time during westbound tunnel construction are presented on Figures 6.109-6.120 which show the measured output (in bits) for all electrolevels (boreholes Ai to Hi and Ji); the distance between the instrument line and the tunnel face is also shown with time on each plot. A key diagram on each figure indicates the sense of rotation relative to the coordinate system given on Figure 2.8.

At inclinometers Ei and Gi (Figures 6.114 and 6.117) positioned on either side of the eastbound tunnel the electrolevels typically show a smooth response during construction. Rotations are first observed for the more shallow instruments when the shield cutting edge is about 5m from the inclinometer line; the first responses for deeper electrolevels are seen slightly later (shield about 3m from instrument line). The rotations slowly increase in magnitude until the shield is beneath at the instrument line at which time rotations begin to increase rapidly, particularly for instruments nearest the tunnel extrados (electrolevel nos. Gi2-Gi4, Ei2-Ei3). The responses slow when the shield edge is about 5m past the inclinometer line (at which point a lining ring is erected and expanded at the instrumentation plane) but rotations continue in smaller amounts until the shield face is about 15m past.

At inclinometers Fi (Figures 6.109-6.111) were adversely affected by the fast multiplexer used at the beginning of Period 4: continued monitoring after the system was changed recorded significant rotations well before the first settlement response observed at the centre-line. They do, however, show response trends with time which are similar to Di and Ei (for example) reflecting some construction influence. The rotations in these holes are generally positive which is consistent with ‘leaning’ towards the eastbound tunnel. Electrolevels in inclinometer Hi (Figures 6.118 and 6.119) are oriented askew to the parallel and transverse directions, but show similar patterns of response relative to the construction events as are observed in other boreholes.

At inclinometer Fi the electrolevels are oriented both transverse (F1i) and parallel (F2i) to the tunnel axis. The transverse electrolevels (Figure 6.115) all measured small rotations (up to 70 bits or about 0.4mm m); several of these instruments exhibit trend changes after the tunnel face moved past the instrument line. Electrolevels oriented parallel to the tunnel drive (F2i, Figure 6.116) show varied forms of response (note the different plotted scales between this figure and Figure 6.115). Near the ground surface electrolevel no.F2i-5 first begins to respond when the tunnel is between 10 and 15m from the inclinometer line. Rotation is negative (counter-clockwise) indicating tilt of the top of the inclinometer towards the advancing tunnel face. The direction of rotation reverses once the shield edge is about 5m past the instrument line. At the bottom of the same inclinometer hole, electrolevel no.F2i-1 (positioned less than 1.5m above the tunnel crown)
shows a rapid positive rotation when the tunnel moves to within 5m of the instrument line. The passing of the cutting edge is marked by a reversal in rotation direction, followed by a dramatic jump of 400 bits (or about 2mm/m) when the shield is about 5m past the instrument line and a lining ring is expanded at the section. Smaller rotations continue with tunnel progression until the shield is about 20m past the section.

Vertical profiles of relative horizontal displacement calculated from the measured rotations presented above are given on Figures 6.121; the bottom of each inclinometer borehole is assumed to be stable. The development of displacements towards the excavation at the eastbound axis level (-20.5m) is seen for inclinometers Ei and Gi. Inclinometers Ai, Bi, Ci, and Di show overall tilting of the boreholes towards the eastbound tunnel with the maximum displacement at the ground surface. At inclinometer F2i large relative displacements are focused at the tunnel crown, reflecting the large measured rotations of electrolevel no.F2i-1 shown in Figure 6.116. The profiles at lesser depths show some relative movement initially towards the tunnel (negative) but ends with very little differential displacements with depth.

Final profiles at Ai, Bi, Ci, and Di were determined by assuming the rate of drift immediately prior to the tunnel affecting the instrument as the baseline; these are shown on Figure 6.121 as bold lines. Compared with the unprocessed data these show much smaller magnitudes of movement at all inclinometers, and slightly different profile shapes at Bi, Ci, and Di, but the overall forms are not dissimilar.

6.10 Subsurface displacement monitoring in Period 5

6.10.1 Rod extensometers
Seventeen extensometer surveys (nos.61-77) were performed during the first year after eastbound tunnel completion (Period 5) as indicated on Figure 6.122. The changes in calculated anchor levels with time from the end of tunnelling are shown on Figures 6.123-6.133 for all extensometers. Generally, the response appears similar in form to that of the rest period after westbound tunnel construction (Period 3). Rates of response slow with time at all anchor positions reflecting the equilibration of pore pressures and stress changes induced during construction.

Vertical profiles of subsurface vertical displacements at each extensometer hole are seen on Figures 6.134 for a selected number of surveys during Period 5. Key diagrams indicate the time in days after eastbound construction. For a given extensometer hole the movement magnitudes at all levels above -17.5m are similar; the reference heads show some seasonal influences. The profiles immediately above the eastbound tunnel (Fx) tend to show maximum movements near the crown reducing with distance from the excavation.

Transverse profiles of vertical displacements presented on Figures 6.135 show the distribution of movements across the site at different subsurface horizons. The profiles at the ground surface exhibit some localised seasonal variations: the magnitudes and patterns broadly agree with the surface monitoring results presented in Figure 6.29. At depths between -5 and
-17m the profiles are much smoother.

At $z=-5m$ on Figure 6.135, about 18mm of settlement is seen at the eastbound centre-line: magnitudes of movement are pretty similar with depth. Slightly larger displacements are seen at offsets towards the westbound tunnel centre-line ($y<0$). Continued consolidation of the ground above the westbound tunnel is evident from the subsurface profiles at -27m (i.e. well below the eastbound tunnel invert level). One year after eastbound construction the ground has settled a further -8mm at the westbound centre line.

6.10.2 Electrolevel inclinometers

Evaluation of the auto-logged response after the eastbound tunnel was completed and the manual measurements performed thereafter showed that a shift in the instrument response with time occurred when the monitoring systems were changed between Periods 4 and 5. This change has obscured the real time-dependent response due to tunnelling and therefore the potential movements can not be confidently evaluated. These results are therefore not presented.

Again, comparison of the rotations obtained from borehole mapping using an inclinometer torpedo (performed over two years after westbound tunnel construction and described in Appendix D) with the measured rotations immediately after tunnel construction gives a strong indication that very little differential horizontal movement occurred near the eastbound tunnel extrados after tunnel construction (see Figure 7.26 in Chapter 7).

6.11 Pore pressure and total stress measurements in Period 1

6.11.1 Piezometers

The piezometers were installed at the site about 80 days before westbound tunnel construction; between 12 to 15 measurements were completed at each instrument prior to the tunnel drive. These are presented on Figures 6.136 and 6.37 with time for the westbound and eastbound centre-lines respectively. On installation it was expected that the piezometers would show a rise in measured pressures as the bentonite used to cap the sand pocket finished swelling, and negative pore pressures that have been generated near the hole from boring dissipated. However, nearly all of the piezometers show only small changes after installation ($\pm 5kPa$). Piezometer BP2 located immediately above the westbound tunnel crown showed a rise (~25-30kPa) over a period of about 30 days. These differences between piezometers may reflect variations in permeability through the clay. It is noted that the sand pocket at BP1 is likely to be in direct contact with a sand parting observed in a U4 sample taken at the borehole base (see Appendix A).

Best-fit linear regression performed on the last 6 measurements for each instrument yield rates of response between 0.01kPa day and -0.02kPa/day which seem to indicate the instrument response is reasonably stable. The pressure magnitudes at the end of Period 1 just before westbound tunnel construction are tabulated on each figure: BP1 (-24.3m) = 191kPa; BP2 (-27.2m) = 212kPa; FP1 (-10.5m) = 58kPa; FP2 (-14.6m) = 98kPa; FP3 (-17.9m) = 124.5kPa.
The values plot close to a hydrostatic pressure profile for a water table level of about -5m, as shown on Figure 6.139.

6.11.2 Spade cells with integrated piezometers

The spade cells were installed about 50 days prior to the westbound tunnel arrival; during this period about 12 sets of readings were made. Figure 6.138 shows both the measured piezometric pressures and horizontal total stresses for each spade after installation during Period 1. These data are adjusted for over-read typically found around spade cells in the stiff blocky clay, by the recommended correction factor of $-0.5\sigma_0$ (Tedd et al., 1989).

A period of rapidly changing response spanning 14 days after installations is seen for all four spades. During this period the measured total stresses at each cell drop as the excess stresses generated by push-in redistribute and the excess pore pressures equilibrate. The change for each stress cell is about -400kPa and the final measured total horizontal stresses range between 500 and 775kPa as tabulated on Figure 6.138. During the same time span the measured pore pressures all show a steady rise with time from zero to about 150kPa. These ‘equilibrium’ pore pressures also plot on the hydrostatic profile depicted on Figure 6.139.

Considering that the instruments are all installed at the same depth, the variation in measured total stresses is substantial. However, a similar range (between about 700 and 900kPa) was seen at the Heathrow Express trial tunnel (New and Bowers, 1994) where the same type of push-in pressure cells were inserted at a similar depth into London Clay.

At St. James’s Park, the relative stress magnitudes between the instruments can be correlated with the relative magnitudes of the push-in pressures observed immediately after instrument installation. There are no records of siltstone or claystone bands in the eastbound tunnel face at this depth and so the variations appear to be attributable only to installation effects/differences, although no noticeable change in installation method was observed.

Linear regressions performed on the last four measurements in Period 1 for each spade cell (both piezometric pressures and earth stresses) are summarised on the Figure 6.138. The piezometers all show rates of response between -0.05 and -0.11kPa/day; the total horizontal stress cells show between -0.09 and -0.34kPa/day rate of change. The final pressures at the end of Period 1 are summarised on the figure.

The in-situ coefficient of earth pressure at rest, $K_0 (=\sigma_0/\sigma_v)$, determined from the spade cell measurements ranges from 1.4 to 2.6, with an average of 2.0. This range compares with between 1 and 2.3 determined during the JLE site investigation work (see Figure 3.8).

6.12 Pore pressure and total stress measurements in Period 2

6.12.1 Piezometers

The piezometric responses measured at the westbound tunnel centre-line (BP1 and BP2) and at 21.5m offset (eastbound centre-line: FP1, FP2, and FP3) are shown with westbound shield position on Figures 6.140 and 6.141.
On the westbound centre-line and above the advancing tunnel crown the pore pressures rise by about +20kPa (or about 10%) when the tunnel face is about 20m from the instruments. The pressures then reduce rapidly as the shield moves closer to and finally past the instruments. Pressures at BP1 reach a minimum recorded value of 35kPa (a change of -155kPa) with the shield cutting edge just past the instrument. Recovery of pore pressure begins immediately to 45kPa with the shield edge about 8m past the instrument, and to 110kPa with the shield about 40m past. The response at BP2 is similar to BP1 but with a minimum pressure of 2kPa when the shield is 6m past the instrument; pressure recovery also begins immediately but at slower rate than at BP1. With the shield about 12m past the instrument, pressures reach 35kPa; a further increase to 40kPa is observed when the shield is 40m away. 2kPa is considered the minimum measurable pressure through a completely open pneumatic system and therefore it is likely that the soil at BP2 had zero or possibly negative pore pressures for a short time. The gap in measurements between \( x=+20m \) and \( x=0m \) occurred because of essential servicing of the pneumatic read-out unit.

At the eastbound centre-line (Figure 6.141) the piezometers all show a small rise of between 10 and 15kPa as the tunnel moved past each instrument plane.

### 6.12.2 Spade cells with integrated piezometers

The piezometric and total stress changes at the spade cells measured during westbound tunnel construction are shown on Figure 6.142 along with tunnel face position and a key diagram indicating the relative locations of each spade.

The pore pressures show a gradual increase starting from when the shield is about 20m from the spade cell line. The rise continues steadily for nearly six days after tunnel construction. Magnitudes of pore pressure change vary from +18 to +35kPa, increasing with nearness to the tunnelling works. About 12 days after tunnel construction, the pore pressures show signs of slowly dissipating.

The total horizontal stresses measured show a positive increase in pressures response during tunnel construction, reaching a pseudo-stable plateau immediately after the tunnel is clear of the site. The measured changes in total stress range from +6 to +23kPa, again increasing with nearness to the tunnel. The pressures begin slowly to reduce about 6 days after tunnel construction.

The responses of the spade cells during westbound tunnel construction give some indication of the relative response rates for stress and pore pressure measurements and or ground response. The total stress measurements are closely tied to the construction events and are stable for the first few days after tunnel the westbound tunnel passed. The pore pressure responses appear to be lagging slightly, reaching their minimum values several days after the tunnel had cleared the site.
6.13 Pore pressure and total stress measurements in Period 3

6.13.1 Piezometers
At the westbound centre-line, the rapid drops in pore pressures observed during tunnel construction are shown on Figure 6.143 to recover slowly with time. The rate of pressure rise at BP2 (immediately above the tunnel crown) is initially more rapid than BP1, but after about 150 days the response rates are more alike and are significantly slower. After 230 days the pressures changes are very small; the pore pressure at both instruments is about 165kPa (or a net change from initial conditions of about -30kPa at BP1 and about -55kPa at BP2).

On the eastbound centre-line (Figure 6.144) the small rises in pore pressures observed during construction at all three piezometers are seen to dissipate slowly with time. FP1 (the shallowest piezometer and the one which showed the smallest change in Period 2) shows the most rapid dissipation of excess pore pressures, reaching its pre-tunnelling pressure after 75 days. Later measurements show some further reduction, but at the end of Period 3 the net change from Period 1 is zero. At FP2 and FP3 similar trends are observed to FP1. At both of these positions the pressures reduce from their peak immediately after westbound tunnel construction; changes of about +10kPa from the initial pressures are seen at the end of Period 3.

6.13.2 Spade cells with integrated piezometers
The total stress and piezometric changes measured during and after westbound tunnel construction at the spade cells are shown on Figures 6.145 and 6.146 with time; key diagrams indicate the relative locations of each spade.

The total horizontal stresses reduce with time from the observed increases seen after tunnel completion. SP1 shows a very gradual reduction in stress; after about 100 days the stress has returned to the initial value but by the end of Period 3 a net change of -10kPa is seen. Stresses at SP3 and SP2 return to initial levels within the first 25 to 50 days after tunnel completion, but continue to drop with time. Net changes of between -20 and -30kPa are noted at the end of Period 3 (about 230 days after tunnel completion). SP4 which showed the greatest positive change in stress during westbound tunnel construction and the most rapid reduction with time, but reaches its pre-tunnel stress later than the others. The net stress change at the end of Period 3 is about -10kPa. Generally, the rate of response is quicker with proximity to the tunnel excavation for the spade cell measurements.

From the peak pore pressure changes of between +15 and +30kPa seen five days after tunnel completion the pressures at all spades show a steady reduction with time. The rate of response is slightly quicker for the spade closest to the excavated tunnel (SP4). After nearly 250 days the piezometric pressures at these locations are approaching the original pressures measured prior to tunnelling.
6.14 Pore pressure and total stress measurements in Period 4

6.14.1 Piezometers

The piezometric responses measured at -21.5m offset (westbound centre-line: BP1, BP2) and on the eastbound tunnel centre-line (FP1, FP2 and FP3) are shown in relation to shield position on Figures 6.147 and 6.148.

At the westbound tunnel centre-line (Figure 6.147) the two piezometers show a gradual increase in pressures starting when the tunnel is between 17 and 25m from the instrument lines. At BP2 the rise continues gradually, reaching about 167kPa (a net change of +17kPa from the end of Period 3) when the shield is over 40m past the section. A sharper step in response is observed at BP1 during eastbound construction with pore pressures reaching about 190kPa (an increase of +28kPa from Period 3).

Above the eastbound tunnel (Figure 6.148), pore pressures are observed to increase when the tunnel face is about 20m from the instruments; maximum pressure increases of between +20 and +30kPa occur when the tunnel is about 7m from the instrument line. The pressures at all three piezometers then reduce as the tunnel progresses closer to and then finally past the instrument positions. Pressures at FP1 reach a minimum value of about 2kPa (a change of -60kPa from Period 3) with the shield cutting edge is at the instrument. FP2 registers a minimum pressure of about 30kPa (a change of about 70kPa from Period 3) when the shield is about 5m past the instrument. The sand pocket of FP3 at the tunnel crown was clipped by the shield crown, and consequently pressures dropped to 2kPa when the shield passed immediately below. It is likely that FP1 was measuring zero or possibly negative pore pressures for a short time.

At FP1 the pore pressure remains at 2kPa until the tunnel face is about 12m past the instrument, after which the pressure begins to rise slowly: a recovery of about +20kPa is seen when the shield is over 40m past the instrument. The response at FP2 shows a +10kPa pressure rise to 40kPa when the tunnel is about 20m past the instrument, but drops to about 30kPa when the shield is over 40m past. No pressure recovery is seen at FP3.

6.14.2 Spade cells with integrated piezometers

The piezometric and total stress changes recorded during eastbound tunnel construction are shown on Figures 6.149 as measured pressures and stresses with tunnel face position, and on 6.150 as changes in response from the end of Period 3 with tunnel face position.

The total horizontal stresses measured show a positive change as the tunnel approaches the instrument line. With the shield 5m from the section, stresses at SP1 and SP2 begin to reduce rapidly and continue until the shield is about 3m past. At SP3 and SP4 the stresses also reduce, starting when the shield is about 2m from the instrument line. When the shield is between 5m and 7m past the section the magnitudes of total stress changes are generally at their maximum (Figure 6.150): SP1 -425kPa, SP2 -330kPa, SP3 -180kPa. At SP4 the maximum stress change (-90kPa) is seen slightly later when the shield is about 10m past the instrument.
As the tunnel moves away from the instrumentation line the stresses begin to increase; recovery rates vary, but are initially most rapid for SP1 and SP2 nearest the tunnel.

Pore water pressure increases of about 5kPa are observed at SP2, SP3 and SP4 as the shield approaches the instrument line. When the tunnel face is about 2m from the line, the pore water pressures begin to reduce gradually, stabilising when the tunnel face is about 15m past. The largest changes are observed at SP1 (-52kPa) and SP2 (-35kPa) closest to the tunnel (Figure 6.150). At SP4 pressure the changes are small (-5kPa) and are seen when the shield is over 5m past the instrument line. With the face over 15m past the section the pressures at SP4 show an increase to a net change of +10kPa.

6.15 Pore pressure and total stress measurements during Period 5

6.15.1 Piezometers
The post-construction response on the westbound and eastbound tunnel centre-lines are shown on Figures 6.151 and 6.152. Above the westbound tunnel the small increases in pressure observed during eastbound construction are observed to dissipate with time. At BP1 pressures returned to the level immediately prior to eastbound tunnel construction after about 200 days; pressures continue to drop through the remainder of Period 5 monitoring, reaching a net change of -25kPa after 600 days. At BP2 the 15kPa pore pressure increase remains reasonably static for about 200 days after eastbound construction, beyond which the pressures reduce at a similar rate to BP1.

Above the eastbound tunnel (Figure 6.152) the recovery of pore pressures is most rapid at FP1, reaching a steady value of about 55kPa after about 100 days. FP2 shows a more prolonged trend of pressure recovery; a stable value of 90kPa after more than 500 days. FP3 continues to show no pressure recovery. At piezometers FP1 and FP2 the net change in pore pressure (from the end of Period 3) after 600 days is less than -10kPa.

6.15.2 Spade cells with integrated piezometers
The magnitudes and changes at the spades of both total stresses and pore pressures are shown on Figures 6.153 and 6.154 against time after eastbound tunnel construction. The total stress measurements on Figure 6.153 show a continued rise in total stress for the first 10-15 days after eastbound tunnel construction. During the 600 days that followed, the stress changes vary with instrument position: at SP1 and SP2 the stresses show a slow increase while at SP3 and SP4 the stresses decrease with time after completion.

For all four instruments the pore water pressures increase during the first 20 days after tunnel construction. Changes range between 20kPa (at SP1 and SP2 nearest the tunnel) to about 50kPa (at SP4) from the end of tunnel construction (Figure 6.154). After 20 days a gradual reduction in pore pressures with time is seen, with similar rates of change for all instruments. After about 600 days the measured pressures are reasonably stable. The net changes in pressure from the end of Period 3 are about -90kPa at SP1 and SP2 nearest the tunnel, -60kPa at SP3 and
about -40kPa at SP4.

6.16 In-tunnel measurements
Measurements of diametric change in both the eastbound and westbound tunnels were taken periodically to assess lining performance. Lining loads were monitored by the Transport Research Laboratory (TRL) in the eastbound tunnel only (see Chapter 5 for a description of the instrumented rings).

6.16.1 Measured diametric changes in tunnel linings
Figure 6.155 shows the changes in horizontal diameter with time for five rings (nos. 0984-0992) in the westbound tunnel. Eyebolts were installed in the crown, invert, and spring line segments of the five rings about 210 days after westbound construction: the first set of measurements were taken shortly thereafter. These were used as the base set for assessing further diametric changes, and therefore zero changes are shown on the Figure 6.155 at this time. The two measurement sets which were performed within 30 days after the initial set show only very small changes in the horizontal span (less than +0.1mm). The fourth measurement survey, completed 8 days after eastbound tunnel construction (18 Jan 1996), shows a horizontal diametric lengthening of about +1.2mm. This movement was complemented by an equal vertical diametric shortening (not shown). Survey no.5 shows additional horizontal lengthening reaching between +4.8 and +5.6mm from the base set. The vertical diametric changes were not measurable because the eyebolts were destroyed after survey no.4.

Horizontal diametric changes in the eastbound tunnel are presented in Figure 6.156 against both date and days after eastbound tunnel construction. Measurements were made across the two instrumented rings which are described in Section 5.6. The rings were built in the early morning hours of 10 Jan 1996 and the base span readings vertically and horizontally were obtained 24 hours later on 11 Jan 1996. Within the first 12 days after the base set, the horizontal diameter increased by about +4.2mm (consistent with squatting deformation of the tunnel). This elongation magnitude increases to +11mm after about 120 days, and to between +17.2 and +20.4mm after just over 1 year. These values are much larger than the +2mm horizontal diameter lengthening recorded at Regent’s Park after about 110 days (Barratt and Tyler, 1976). The equivalent radial straining for the eastbound tunnel after nearly 400 days of 0.4% is 1.5 to 2 times greater than Peck’s (1969) data for tunnels in stiff clays.

6.16.2 Lining loads in eastbound tunnel
The lining thrust measured from load cells in the eastbound tunnel are presented on Figure 6.157 against log-time. The lining expansion results in an initial jump in the load for all four cells. Thereafter, the measured loads steadily increase with time for all cells, reaching between 40°o and 50% of the total vertical overburden stress after 100 days. The consistently higher loads measured at the NE side axis (-10°o to +15°o) are attributed by Bowers and Redgers (1996) to localised effects. The measurements from the other cells imply slightly larger vertical
loads (measured at the spring lines) than horizontal loads (recorded at the crown and invert).

After 1 year, the vertical loads (measured at the spring lines) are seen to increase to about 60% of the total overburden stress. The horizontal loads at the crown and invert cells show only small increments, reaching about 45% of the total overburden stress after the same time period. Similar patterns of difference in vertical and horizontal lining loads were recorded by Barratt et al. (1994).
7 Data Analysis

In this chapter the data presented in Chapter 6 are analysed to illuminate the modes of response and to characterise the displacement patterns around and above the two tunnels. Comparisons are made where possible with field measurements obtained from other tunnelling projects in London and with the measurements made by the JLE and their contractors.

7.1 Displacement development and characterisation above both tunnels

7.1.1 Surface settlement, horizontal movement and vectors of displacement

Across St. James's Park

Volume losses ($V_j$) and normalised trough width parameters ($i_j/z_0$) determined from the contractor’s surface monitoring above the westbound tunnel between Westminster Station and the existing JLE tunnels at Green Park are shown on Figure 7.1. At the top of the plot, a simplified picture of the geology is given along with elevation of the westbound tunnel crown relative to ordnance datum (OD). The length of tunnelling south of the lake from Westminster to St. James’s Park shows volume losses exceeding 3% and normalised trough width parameters of about 0.4. North of the lake, the normalised trough width parameters increase slightly to between 0.4 and 0.5, and the volume losses drop to less than 2%. This decrease in volume loss along the route is paralleled by changes in construction method as marked along the chainage axis (x axis) of the plot. South of the lake, the method of excavation was described in the daily shift reports as 'a maximum muck of 1900mm ahead of shield' with no forepoling or breasting plates used (see Figure 5.6 for shield details). The term ‘muck’ refers to the excavation and the length given is the excavation length in front of the shield cutting edge. The full tunnel diameter was probably removed with the backhoe leaving very little soil around the tunnel circumference to be cut by the cutting edge when the shield was jacked forward.

North of the lake, three other methods of excavation were used along the route as shown on the Figure 7.1, the details of which are as follows:

Method 1 muck 1200mm ahead and 500mm tight around circumference
Method 2 mucking ahead 800mm and 500mm tight for top half of face, extend forepoling and move breasting plates to active position (again see Figure 5.6), then muck lower half of face; retract forepoling and breasting plates, repeat excavation sequence, and then shove shield forward.
Method 3 as for Method 2, but with a half-shove of the shield every 500 to 800mm of digging.

The term ‘tight’ in these descriptions refers to the act of leaving soil around the extrados for the shield to cut when the shield is jacked forward. The length given refers to the thickness of soil...
It is not intended to investigate the different techniques here, but to record that the smaller ground disturbances observed appear to coincide with the closely supervised and more rigorous excavation methods during the westbound tunnel drive.

Figure 7.2 shows the volume losses determined from levelling north and south of the St. James's Park lake during eastbound tunnel construction, and the distance between the two tunnel centre-lines through the park. Comparative data through the Westminster area are not presented because compensation grouting was being performed above the advancing tunnel shield to protect overlying structures.

For the eastbound tunnel, the volume losses immediately south of the lake is nearly 3%. North of the lake the estimated values are less than 1.5%. The JLE shift reports give no indication of the tunnelling methods used or changes in the excavation method along the drive. It is noted that the speed of the eastbound tunnel progress is about half that observed for the westbound drive, which might indicate closer supervision during the eastbound tunnel excavation through instrumented section and through the park. The reducing volume loss is accompanied by increasing distance between the two tunnels, which implies that any interaction effects are probably reducing between the two tunnels with progress northward.

At the instrumented section

The transverse profiles of settlement during tunnel construction measured at the instrumented section are considered as three separate profiles: the trough measured above the westbound tunnel; the south half-trough above the eastbound tunnel (closest to the previously constructed westbound tunnel); and the north half-trough above the eastbound tunnel. The response above the eastbound tunnel is asymmetrical, and so for clarity and investigative ease the two halves are assessed separately.

Profiles of normalised settlement, \( w_y/w_0 \), for the construction of both (a) westbound and (b) eastbound tunnels are plotted on Figure 7.3 against normalised offset from the centre-line \( y/z_0 \). Each plot incorporates the precision levelling results presented in Chapter 6 during both Period 2 (7.3a) and Period 4 (7.3b). Note that the south half-trough above the eastbound tunnel has been mirrored onto the north half-trough in Figure 7.3b for comparison.

Also given on Figures 7.3a and b are the range of settlement profiles obtained by the JLE contractors at their monitoring lines north of the lake (where the volume losses were estimated to be between 1.5% and 2.2%), and the range of profiles seen during construction of the Heathrow Express \( K' \) of between 0.7 to 1.3% where the geology, geometry and tunnelling methods are very similar to St. James's Park (Barakat, 1996). It is worth noting here that the accuracy of measurement at the north end of the park was estimated from the contractor's measurements prior to tunnel construction at about ±1mm, and the maximum vertical displacements observed were between -9 and -11mm.

The westbound profiles during construction on Figure 7.3a are tightly grouped near the centre of the Heathrow data range. The range of profiles observed at the north end of the park
are slightly wider on average. On Figure 7.3b (i.e. the eastbound drive) two groupings are seen; the north half-trough data plot on the narrow side of the Heathrow range while the south half-trough data (nearest the previously constructed westbound tunnel) plot as wider troughs. The profile shapes for the westbound drive and the 'less disturbed' north half-trough above the eastbound drive are within the observed range at Heathrow, but are generally narrower than the profiles at the north end of the park (indicated by the profiles from monitoring lines D-G).

For the large range of volume losses observed (0.7% at Heathrow to 3.3% at St. James's Park) these figures appear to demonstrate that the transverse trough shape at the ground surface is reasonably independent of the magnitude of ground disturbance for similar ground conditions. The differences at larger offsets between the recorded data and the contractor's monitoring is probably attributable, in part, to the differences in measurement accuracy.

The southern 'disturbed' half of the eastbound troughs tend to be wider than the other field data which indicates proportionately larger movements further from the tunnel relative to the maximum settlement near the centre-line. The wider trough is likely to be the result of westbound tunnel construction which would have left a slightly softened soil south of the eastbound tunnel. The stress changes induced during eastbound construction cause larger strains in this less stiff soil and therefore larger displacements occur.

Displacement development during construction for both tunnels is illustrated by plotting normalised centre-line settlements, \( w/w_{0, \text{max}} \), and surface trough volumes, \( V/V_{s, \text{max}} \), against normalised distance from the shield edge to the SMP line, \( x/z_0 \), as shown in Figure 7.4. Two thick lines indicate the range of profiles observed by the JLE contractors north of the St. James's lake; good agreement is seen between these and the data recorded at the instrument section. The two dashed lines are best-fit curves to the westbound and eastbound profiles. They are obtained from least squares analysis by assuming that the form of the profile is a cumulative probability function as described in Chapter 2 and by varying both \( i_x \) and the position of 50% displacement \( (x_{50}) \); the curve-fit results are summarised in Table 7.1. The trough length parameter at the ground surface varies between 9.8m for the westbound tunnel to 6.9m for the eastbound tunnel.

The centre-line settlements and surface trough volumes develop similarly for both tunnels. This verifies an important assumption of the empirical prediction framework, and supports results from 2-D numerical and model testing which show direct relationships between volume loss and centre-line settlement (Addenbrooke, 1996; Mair, 1979). Just over 50% of both the maximum centre-line displacement and maximum trough volume is seen when the westbound shield face is positioned directly beneath the monitoring line as compared with 40% observed above the eastbound tunnel. This indicates that a larger proportion of vertical displacements above the westbound tunnel were occurring in front of the shield edge. In advance of the eastbound shield \( (x/z_0 \text{ between } 0.5 \text{ and } 1) \) small amounts of heaving volume (i.e. negative \( V/V_{s, \text{max}} \)) were recorded; these are equivalent to apparent heave at all SMP's of about +0.3mm. The trend is not reflected in the centre-line settlements.

The best-fit curve for the westbound data on Figure 7.4 overestimates the initial movements due to construction as the tunnel approaches the SMP line but generally appears to
match the maximum slopes and curvatures as the tunnel progresses beneath it. With the shield past the section, the observed displacement growth is slower than for the cumulative probability line (i.e. the best-fit line reaches $w_0/w_{0,max} = 1$ more rapidly). For the eastbound tunnel the best-fit line agrees very closely at all tunnel positions (generally to within ±0.02 $w_0/w_{0,max}$ or about ±0.4mm) after the initial heave.

Final transverse settlement profiles measured immediately after each completed tunnel are plotted on Figures 7.5a and b to determine the trough width parameter, $i_\gamma$ (see Figure 2.16). For both tunnels the distribution of settlement transverse to the tunnels is not exactly Gaussian. For the eastbound tunnel there are two clear trends seen (corresponding to the north half-trough and south half-trough) indicating a significant amount of asymmetry in the profile.

For an idealised settlement trough, the point of inflection also coincides with the point of maximum slope (see Chapter 2). Hence an estimate of $i_\gamma$ can be determined from plots of average slope between settlement monitoring points. Figure 7.6a and b shows the calculated slopes for the final displacement profiles for both tunnels. Best-fit curves derived from least squares analysis are superimposed on each plot; the north and south half-troughs above the eastbound tunnel were assessed separately. The calculated offsets to maximum slope are also included in Table 7.1.

The values of $i_\gamma$ calculated for each tunnel using different methods summarised in Table 7.1 range between 10.4 and 13.3m for the westbound tunnel, 9.0 and 10.8m for the south half-trough above the eastbound tunnel, and 6.6 and 8.2m for the north half-trough above the eastbound tunnel. For comparison, values of 12.8m (westbound tunnel) and 8.3m (eastbound tunnel) are estimated using Equation 2.20 for layered ground (assuming $K_{clay} = 0.45$ and $K_{gravel} = 0.3$). For both tunnels the trough widths estimated from the Gaussian fit are larger than the offsets to the point of maximum slope: the differences range from 2.5m for the westbound tunnel to 1.5m for either side of the eastbound tunnel.

Asymmetry above the eastbound tunnel is also seen by comparing the observed surface trough volume, $V_s$, for the two half-troughs after tunnel completion (see Table 7.1). The volume for the less disturbed north half-trough accounts for only 38% of the entire trough volume, and gives an equivalent $V_1$ (assuming symmetry about the eastbound centre-line) of 2.2% — only 75% of that observed.

By repeating the curve-fitting described above for the transverse settlement profiles observed during shield advance beneath the section, the change in $i_\gamma$ relative to the moving tunnel face can be investigated. Figure 7.7 presents the variation in calculated inflection offset with normalised tunnel shield position. As the tunnel face moves closer to the SMP line the magnitudes of $i_\gamma$ calculated for both drives are seen to reduce by between 2 and 4m. Once the shield edge moves past the SMP line, the estimated values of $i_\gamma$ become more or less stable.

For both tunnels the differences between the Gaussian fit estimates of $i_\gamma$ and the offsets to the point of maximum slope generally differ between 1.5 and 4m for all tunnel face positions. Offsets to the point of maximum slope for the north half-trough of the eastbound tunnel increase
slightly with tunnel advance.

Transverse horizontal displacement development is evaluated by considering vectors of surface displacements in the transverse (or y-z) plane for each tunnel as shown on Figures 7.8 and 7.9. These were determined by combining micrometer stick measurements and precision levelling results. Vectors less than 1mm in length are not shown. For simplicity only two tunnel face positions are considered: (a) shield immediately beneath the SMP line, and (b) shield well past the SMP line.

With the tunnel face immediately beneath the SMP line, the vectors of displacement for the westbound tunnel on Figure 7.8a are focused at a depth of about -23m and at about +2m offset. The eastbound vectors for a similar shield position are directed at about -12m depth on the eastbound centre-line for the south half-trough. The north half-trough vectors above the eastbound drive (Figure 7.8b) are less well-conditioned without a strong centre of focus.

On westbound tunnel completion the surface displacement vectors on Figure 7.9a are focused at a depth of -20m and about +3m offset; vectors above the eastbound tunnel continue to show more scatter than for the westbound tunnel (Figure 7.9b) but are generally focused between -9m and -12m near the centre-line over the whole trough. Similar patterns are found from the total station surveys results (not shown) above both tunnels.

The observed surface displacement vector directions for both tunnels are significantly different from the widely adopted assumption that displacement vectors are aimed at the tunnel axis or invert (O’Reilly and New, 1982; New and Bowers, 1994). Mair (1979) recorded vectors of near surface movements above model tunnel tests in overconsolidated kaolin which were aimed at a point above the tunnel crown, and more recently by Grant (1998) made similar observations for model tunnel centrifuge tests in layered ground (sand on top of overconsolidated kaolin). This shallow depth of focus implies larger horizontal components of movement at ground surface and is probably related to the low confining stresses near the surface and the correspondingly low material stiffness (particularly for the granular material subjected to tensile straining). Differences in vector directions between tunnel face positions immediately below and well past the SMP line imply that ratios of horizontal to vertical displacements increase slightly with tunnel progress.

Figure 7.10a-c shows: (a) parallel horizontal displacement, $u$, at the tunnel centre-line; (b) transverse horizontal displacement ($v$, the maximum in the profile for each survey); and (c) vertical displacement ($w$) at the westbound tunnel centre-line above the westbound tunnel with respect to normalised distance from the shield edge to the SMP line ($x/z_0$) of. Each plot represents changes in the largest magnitude of movement (positive or negative) for each dimension ($x, y, z$) with advancing westbound tunnel face positions. The data presented includes the results from all of the surface monitoring types presented in Chapter 6.

The profile of $u$ displacements on the centre-line is nearly Gaussian in form, peaking when the shield is slightly past the instrument line; this agrees well with the idealised form presented on Figure 2.10. The maximum movement recorded ($u_{max}$, dashed line) is about -7mm (towards the advancing tunnel face and Westminster).
The largest transverse horizontal displacement \((v)\) grows as the shield advances toward and beneath the SMP line. The maximum movement \((v_{\text{max}})\) of -7mm (towards the centre-line) is recorded when the shield is well past the SMP line \((x/z_0>1.5)\).

The bottom plot shows the centre-line vertical displacements, \(w\), measured from total station surveying and levelling onto SMP no.3, and levelling onto extensometer reference head Bx; these show a maximum vertical movement \((w_{\text{max}})\) of about -20mm.

The equivalent data for the eastbound drive are presented on Figure 7.11a-c. The upper plot shows a profile of \(u\) at the eastbound centre-line similar to that observed above the westbound tunnel; a peak-movement \((u_{\text{max}})\) of about -7mm is seen at \(x/z_0=0\) when the tunnel face is directly beneath the SMP line.

The largest transverse horizontal displacements above the eastbound show two trends, one for each side of the tunnel centre-line. The maximum displacement magnitudes \((v_{\text{max}})\) are 12mm for the south half-trough and about -8.5mm for the north half-trough, both towards the tunnel centre-line.

The vertical displacements from total station surveying and levelling onto SMP no.11, and levelling on extensometer Fx agree favourably with each other, and show a maximum vertical movement, \(w_{\text{max}}\) of about -24mm.

### 7.1.2 Subsurface settlement profiles

Subsurface settlements are investigated on Figures 7.12a-c and 7.13a-c which present for each tunnel the following data:

- \((a)\) transverse settlement profiles for each subsurface level after tunnel completion, and the calculated volume losses for each (by determining the area of the settlement profiles, \(V_s\));
- \((b)\) plots of normalised settlement against offset squared for determination of the trough width parameter, \(i_s\);
- \((c)\) normalised centre-line settlement \((w/w_{\text{max}})\) and normalised trough volume \((V_s/V_{s,\text{max}})\) against distance from the shield face \((x)\), along with the best-fit cumulative probability function as described in Chapter 2.

Figures 7.12a and 7.13a show that above both tunnels, the vertical displacement magnitudes increase with depth and the troughs clearly become more narrow. The calculated volume losses remain nearly constant with depth, except possibly for a small increase close to the tunnel crown in the case of the westbound tunnel. This constant volume trend seems to vindicate the assumption of an undrained response for tunnels in London Clay, but also indicates that any volume changes which may be anticipated from the dilatant behaviour of the London Clay on shearing are small or reasonably constant with proximity to the tunnel excavation. The transverse subsurface profiles above the eastbound tunnel exhibit asymmetry similar to the surface profiles presented in Section 7.1.1.
The trough width parameters \((i_y)\) calculated on Figures 7.12b and 7.13b are seen to reduce with depth for both tunnel, a result which reflects the narrowing trough shapes described in Sections 6.7 and 6.9. Above the eastbound tunnel on Figure 7.13b there are only small differences in \(i_y\) between the subsurface half-troughs. In contrast, the surface profiles presented on Figure 7.5b show significantly wider south half-troughs than for the north half-trough. Having only two extensometers on the north side of the eastbound tunnel at depths greater than -5m probably prevents a clear definition of the potential asymmetry of \(i_y\). The linearity of the data on Figures 7.12b and 7.13b is a measure of nearness to the Gaussian shape. The profiles at all depths, particularly within a half diameter of the tunnel crown, are not exactly Gaussian in form.

Subsurface settlement development on the centre-lines above both tunnels are presented on Figures 7.12c and 7.13c. Similar patterns of response for both the centre-line displacement, \(w_0\), and the estimated trough volume at each depth demonstrate the direct association between the two quantities above the excavation to levels within 1m of the tunnel crown. The trough lengths \((i_l)\) at different subsurface levels show a reduction with depth which reflects the shortening of the overall profile length nearer the tunnel. Details of the subsurface profiles and the calculated trough width and trough length parameters are summarised in Table 7.2.

Changes in transverse trough shape during construction are investigated for each tunnel drive by calculating \(i_y\) (in the same manner as before) for profiles measured at different tunnel face positions at each subsurface level. Figure 7.14 shows the variations against normalised offset from the shield face \((x/z_0)\) and a key diagram indicates the subsurface levels considered. Above the westbound tunnel, a small reduction in \(i_y\) is seen as the shield moves towards the instrument line. Only a few surveys were completed between initiation of settlement and the westbound tunnel face arriving at the instrument line, and so the trends are not well defined. Above the eastbound tunnel crown, a clear reduction in \(i_y\) is recorded as the tunnel face approaches the extensometer line for all subsurface levels. The initially large values of \(i_y\) for the eastbound drive on Figure 7.14 \((x/z_0>0.5)\) are for early profiles which are further from the Gaussian form. Once the shield face moves past the section the values at all depths become stable. Presenting the same data as \(K = i_y/[z-z_0]\) on Figure 7.15 highlights a difference between the data at a level within 1m of the tunnel crowns and the levels further from the tunnel extrados. This is, in part, the result of the settlement data being further from the Gaussian form at the horizon nearest the tunnel crown.

### 7.1.3 Discussions on surface and subsurface displacement patterns

The trough width parameters calculated on completion of construction at different subsurface horizons, and the trough length parameters determined on Figures 7.12c and 7.13c at the same horizons are normalised by axis depth \((i_y/z_0, i_l/z_0)\) for each tunnel and plotted against normalised depth \((z/z_0)\) in Figures 7.16 and 7.17; superimposed on each plot is the equation given by Mair et al. (1993) for variation of the trough width parameter \((i_y)\) with depth for tunnels in clay (Equation 2.27). Data from levels above the gravel-clay interface for each tunnel are plotted as
open symbols \((z/z_0 \text{ less than 0.3 for the westbound tunnel, less than 0.4 for the eastbound tunnel})\).

For the normalised trough width parameters on Figure 7.16, close agreement is seen between the westbound data for levels in the clay and Mair et al.'s equation to normalised depths of about 0.8; the values above the eastbound tunnel are slightly larger than the equation over the same normalised depths, suggesting a wider profile near the eastbound tunnel crown. The trough widths above the eastbound tunnel are probably larger because the values are derived mainly from data in the south half-trough where the soil is more disturbed by the previous westbound tunnel. However, the westbound tunnel is constructed in a stiffer material than the eastbound tunnel (even in the absence of recent stress history effects) and therefore the result supports Peck's (1969) suggestion that tunnels in more deformable material give larger values \(i_y\) (see inset diagram, Figure 7.16).

Normalised trough length parameters \((i_x/z_0)\) shown on Figure 7.17 are smaller than the subsurface variation of the trough width parameter \(i_y\) proposed by Mair et al. (1993) at the same normalised depth. This result is emphasised in the plot on the right side of Figure 7.17 which presents the ratio of \(i_x/i_y\) for each depth. Above both tunnels, the ratios are between 0.78 and 0.8 at shallow depths (i.e. in the granular material and at the top of the clay) and between 0.7 to 0.75 for horizons near the clay.

In Table 7.3 a comparison is made between the longitudinal (x direction) and transverse (y direction) profiles within the empirical framework. Tabulated are the ratios of maximum horizontal displacement, maximum slope, maximum curvatures, and maximum deflection ratio in hogging (see Chapter 2) calculated for prescribed differences from unity in the ratio of trough length to trough width parameters \(i_x/i_y\). Reducing \(i_x/i_y\) increases the ratios of longitudinal to transverse maximum slope, maximum curvature and deflection ratio. This correspondingly implies that the longitudinal profile is becoming potentially more damaging to structures relative to the transverse profile (see Section 2.8). Since the observed ratio of \(i_x/i_y\) at St. James's Park is less than unity, assuming \(i_x = i_y\) for assessing the potential damage to structures for the parallel profile above these tunnels is potentially unconservative.

In contrast to the other criteria, the ratio of longitudinal to transverse maximum horizontal displacement, \(u_{\text{max}}/v_{\text{max}}\), decreases with decreasing \(i_x/i_y\). The horizontal displacements observed at the ground surface and presented in Figures 7.10 and 7.11 give ratios of \(u_{\text{max}}/v_{\text{max}}\) of between 0.8 (eastbound) and 1.0 (westbound). These are larger than the ratio of 0.66 given in Table 7.3 for \(i_x = i_y\). Thus, it is inferred from these data that not only are the transverse horizontal displacements larger than expected relative to the settlements observed (Figure 7.9), but the horizontal displacements parallel the tunnel axis at the ground surface are much larger than expected relative to the transverse horizontal displacements.

### 7.1.4 Near-tunnel displacements

The displacements near the tunnels at St. James's Park can be considered in the framework of an idealised collapsing cylinder discussed in Section 2.5 and compared with other tunnels in
London Clay. Existing case history data presented Mair and Taylor (1992) are reproduced again on Figure 7.18 which shows normalised vertical displacements measured at different subsurface horizons on tunnel centre-lines ($w/R$), plotted against normalised distance from the tunnel centre, $R/(z_0-2)$. Figure 7.19 presents normalised horizontal movements ($v/R$) measured at axis level in the direction transverse to centre-line against offset from the tunnel centre-line, $R/y$. The data on both plots approach linear trends when plotted in this manner. It is noted that the volume losses measured or estimated for most of these case histories are between 1 and 1.5%.

Some new information has been added to the data of Mair and Taylor (1992) on Figures 7.18 and 7.19 as follows.

Heathrow Express trial tunnel (New and Bowers, 1994)
The vertical movements at Heathrow agree favourably with the patterns for other tunnels, but the normalised horizontal displacements on Figure 7.19 are markedly less than the case histories. The latter may be the result of the staged excavation process, where the presence of a completed tunnel drift shields the ground (and the instruments installed there) from the following enlargement phase. This was highlighted in Section 2.6 (Figure 2.30) where the right hand side-drift of the Heathrow Express trial was shown to have little effect on the ground to the left of the tunnel which was shielded by the already constructed left hand side-drift.

Field data from St. James's Park
Superimposed on both Figures 7.18 and 7.19 are the movements recorded at St. James's Park after the construction of each tunnel. These data show much larger normalised displacement magnitudes (both vertically and horizontally) than for the case history data, but the trends for both are broadly similar. On both figures the best-fit lines to the data for both tunnels at St. James Park are similar or slightly steeper than for the case history data (particularly for normalised horizontal displacements), implying potentially larger stability ratio, $N$ (i.e. less stable tunnel), for Mair and Taylor's idealised framework (see Figure 2.34). Even steeper trends are estimated if the data from the westbound tunnel is considered separately. On Figure 7.18 the empirical relationships given by Equation 2.28 plotted for the geometries and volume losses observed at St. James's Park bound the field data, with the more shallow eastbound tunnel predicted to give larger normalised values of settlement.

On Figure 7.19 measurable horizontal movements at St. James’s Park are shown beyond a radius of $5R$ (i.e. for $R/y$ less than 0.2) but the majority of these data come from the more disturbed south half-trough of the eastbound tunnel.

FEM modelling of the tunnelling at St. James's Park (Addenbrooke et al., 1997)
Normalised vertical displacements from 2-dimensional (plane strain) FEM modelling of the westbound tunnel (using a non-linear isotropic pre-yield soil model [34] and a modelled volume loss of 3.2%, see Addenbrooke et al.) shown on Figure 7.18 plot on the low side of the data for St. James’s Park. This result is consistent with the FEM surface settlement predictions which
underestimate the displacement magnitudes above the tunnel. In contrast, the horizontal displacements predicting at the tunnel axis are larger than the field data, and plot above the range of field data on Figure 7.19. In both instances, the trends for the data are very similar to the observed field data at the St. James's Park site.

Since the displacement trends around the JLE tunnels are broadly similar to the case histories, it must be asked why there are such large differences in the normalised magnitudes between the observations at St. James's Park and the data represented on Figures 7.18 and 7.19. These data imply that the simple normalisation of displacements by tunnel radius is not generally applicable.

The answer to the differences seems to lie in the fact that the volume losses for the case histories are between 1 and 1.5% while those at St. James's Park are much greater, and that the normalisation presented on Figures 7.18 and 7.19 takes no account of the variability in work quality, shield performance, etc. which each contribute to the overall magnitudes of ground response.

Based on the data from St. James's Park, it is proposed that comparing the ground response around tunnels in London Clay could be improved by normalising with respect to the volume loss, $V_t$. This takes account of the differences in ground disturbance on construction, even though the ratios of $p\sigma_u$ derived from the different site investigations are similar, and makes the normalisation consistent with the empirical form for normalised centre-line settlements with depth given by Equation 2.28.

Figure 7.20 shows the vertical displacement data from Figure 7.18 normalised by the measured volume losses for each tunnel. Some case histories were omitted because there were insufficient data to calculate or estimate a volume loss. In this representation, all of the field data for tunnels of similar diameter — Regent's Park, Green Park, and St. James's Park — fall within a thin band; a similar improvement is seen on Figure 7.21 for the horizontal displacements. Good agreement is also seen for the normalised field measurements and the empirical relationship (Equation 2.28) as presented on Figure 7.20.

Vertical displacement data above the large diameter SCL tunnel at the Heathrow Express trial ($D_{eq} = 8.66\text{m}$, New and Bowers, 1994; Deane and Bassett, 1996) plot above the other case history data on Figure 7.20. The empirical relationship for the geometry at Heathrow gives reasonable agreement to the field data. This simply implies that a unique relationship is not expected for tunnels of different depth and radius when plotting field data, although more field data and parametric studies are needed to evaluate this more thoroughly.

The conclusion from these comparisons is that normalisation of movements within the framework of a collapsing cylinder model must be used with of caution. It seems that application to movements above and adjacent to tunnels is improved if the ground disturbance (i.e. the volume loss) is incorporated into the normalisation. However, the applicability for using the method for comparisons between tunnels of different diameters and tunnels constructed using different excavation and lining systems needs to be investigated further.
7.2 General ground response around both tunnels

An overall picture of the ground response during and after construction of each tunnel is derived from the measured field displacements using a systematic approach schematically presented in Figure 7.22.

For tunnel progress, only the centre-line movements of \( u \) and \( w \) are considered in a section parallel to the tunnel axis (e.g. the x-z plane). Vertical profiles of observed vertical and horizontal displacements at Bx and B2i (on the westbound centre-line, see Figure 5.2) and Fx and F2i (eastbound centre-line) are determined for different face positions (\( x \)) by fitting data with polynomial curves. Profiles of \( u \) and \( w \) parallel to the tunnel drive at different subsurface levels are also curve fitted; these comprise measured displacements and/or calculated displacements (from the vertical profiles of displacements above) at each level. Strain components (\( \frac{du}{dz}, \frac{dw}{dz}, \frac{du}{dx} \) and \( \frac{dw}{dx} \)) are then determined by differentiating the fitted equations with respect to depth (\( z \)) and distance from the shield face (\( x \)). These calculations yield gridded data from which contours of vertical, horizontal and shear strain magnitudes can be produced.

For the transverse case, the strain field is calculated only for the completed construction of each tunnel; the data near the tunnel needed to be augmented by other measurements (see Section 7.2.1 on data coverage) for which similar data were unattainable during construction. Polynomial curves are fitted to vertical profiles of \( v \) and \( w \) displacements for each instrument hole. Curves are then fitted to transverse profiles of \( v \) and \( w \); these data comprise calculated displacements at selected depths for each instrument hole. Strain components (\( \frac{dv}{dz}, \frac{dw}{dz}, \frac{dv}{dy} \) and \( \frac{dw}{dy} \)) were then determined by differentiating the fitted equations with respect to depth (\( z \)) and offset from the centre-line (\( y \)). In addition, magnitudes and orientations of major and minor principal strains, and magnitudes of maximum shear strains were determined for each point by analytically solving the Mohr’s circle of strain construction depicted in Figure 7.23. It should be noted here that the strain calculations presented in this section are positive in compression to agree with the usual soil mechanics sign conventions.

For a frame of reference, the patterns of response for a 2-dimensional strain field around a contracting cylinder in an ideal linear elastic and isotropic (\( K_0 = 1 \)) material (ignoring gravity effects) are shown schematically on Figure 7.24. Strain directions are oriented radially (tensile) and tangentially (compressive) to the opening. Contours of the minor tensile strain direction relative to the horizontal (\( y \) axis) are radial. Contours of strain magnitudes (major and minor principal strains and maximum shear strains) plot as concentric circles around the tunnel extrados while the contours of shear strain magnitude (\( \gamma_{32} \)) form four similar lobes of contours as shown with the \( y \) and \( z \) axes coinciding with the zero shear strain contours.

7.2.1 Data coverage

Vertical displacements, \( w \)

Generally the subsurface monitoring points (i.e. extensometer anchors) were spaced so that displacement profiles with depth and offset could be adequately defined. However, at extensometers Ax, Cx and Dx (around the westbound tunnel) there were no anchors installed.
between the westbound tunnel crown \((z = -28m)\) and the borehole bottoms \((z = -40m)\) as shown on Figure 5.4a. It was therefore necessary to estimate the vertical displacement distributions over this range at each hole so that both the vertical strains and shear strain components could be evaluated near the westbound tunnel axis. Figure 7.25 shows vertical displacements near tunnels from the following sources:

(a) field measurements at the Heathrow trial tunnel (New and Bowers, 1994; Deane and Bassett, 1996);
(b) finite element modelling (FEM) performed at Imperial College for a tunnel geometry similar to the St. James's Park control site (Addenbrooke, 1996); and
(c) field measurements at St. James's Park for westbound and eastbound tunnel construction (Chapter 6).

The settlement data for each set are normalised by the observed (or extrapolated) crown displacement \((\omega_c)\) and the depths have been normalised by the tunnel depth \((z_0)\).

Plotted in this form the data on Figure 7.25 all show rapid reductions in normalised settlement magnitudes with depth, beginning above the axis level \((z/z_0 < 0.8)\). A small amount of heave was recorded beneath the eastbound tunnel invert at St. James's Park, and similar amounts were calculated for each of the eastbound and westbound tunnels from the FEM.

A vertical displacement profile extending from the westbound tunnel crown to below the invert was assumed for extensometers Ax, Cx and Dx using a best fit curve to the data in Figure 7.25. The profile starts with the measured settlement value at the anchor level nearest the tunnel crown and returns to zero displacements at \(z/z_0 = 1.2\) following the normalised best-fit curve form. In the case of the westbound tunnel at St. James's Park, this equates to zero displacements at about 3m below the invert of the tunnel. By ignoring potential heave in this area the calculated vertical strains below axis level probably form a lower limit to those actually occurring in the ground.

**Horizontal displacements, \(u\) and \(v\)**

For profiles of horizontal movements with depth near to both tunnels, it was necessary to augment the measured data from the electrolevel monitoring with the borehole mapping results performed about 1.5 years after eastbound construction (see Appendix D).

Figure 7.26a presents displacement profiles with depth for the inclinometers on either side of the westbound tunnel after westbound construction. For each inclinometer the measured profiles were determined by applying the measured rotation at each electrolevel carriage to the instrument gauge length and summing the calculated displacements assuming the borehole bottom to be fixed.

For each inclinometer hole on Figure 7.26a a second profile is given which was determined from the change in slope obtained from the borehole mapping profiles at depths corresponding to each electrolevel position. These changes were applied to the same instrument
gauge lengths used for measured profiles to give comparable displacement profiles with depth from the borehole mapping.

The measured profiles compare favourably with the profiles interpreted from the borehole mapping, generally to within ±2mm, but show larger differences at shallow depths; these result from the superposition of construction-induced displacements for the two tunnelling events which affect the profile determined from the borehole mapping. Figure 7.26b shows similar comparisons for the inclinometers near the eastbound tunnel and yields similar agreements generally.

The similarity between the measured and mapped profiles adjacent to both tunnels implies that only small differential horizontal movements have occurred near the tunnel extrados during the two years after tunnel construction. It is concluded that the mapped profiles reasonably represent the horizontal displacements for the inclinometers nearest to both constructed tunnels for depths below the crown levels.

7.2.2 Westbound tunnel - parallel profile

Contours of vertical displacements and parallel horizontal movements are given on Figures 7.27 and 7.28. In front of the advancing tunnel face the vertical movements are similar at all depths to the tunnel crown. At x=4m the movements start to grow more rapidly in magnitude and are focused near the tunnel crown. At the level nearest the tunnel crown, nearly 75% of the maximum measured vertical displacements for the same level are observed with the shield edge only 1.5m past the line. At the surface for the same tunnel face position the ratio of surface settlement to maximum surface settlement is closer to 55%.

Contours of horizontal displacement magnitudes (u) on Figure 7.28 are focused near the tunnel crown; they emerge about 5 to 7m in front of the tunnel face. Once the shield edge is past the instrument line (x=0), the movements within one or two metres of the tunnel crown become static (indicated by the horizontal contours). At shallower depths, the displacement trends reverse; nearly zero net displacements are seen at the ground level when the shield is more than 20m past the line (x≈20m).

The displacement vectors above the westbound tunnel crown level during construction are shown on Figure 7.29. Movement directions are predominantly vertical at subsurface levels during the entire tunnel construction process, while larger horizontal displacement components are seen near the ground surface above the shield. At positions behind the tunnel face the horizontal component is seen positive (i.e. in the direction of positive x and tunnel advance).

Contours of vertical strain are shown in Figure 7.30. These are nearly vertical in front of the tunnel face. With the shield edge about 3m from the instrument line, the contour patterns shift, marking an increase in the differential vertical movements. The contours tend to shift towards the horizontal as the tunnel shield moves beneath the instrument line. Once the shield edge is about 7m past the instrument line, the contours are essentially horizontal, which indicates no further straining with tunnel advance. For the level immediately above the tunnel crown, some small increases (<0.5mm/m) in vertical strains are seen once the shield is more than 10m
past and a lining ring is erected beneath the instrument line.

In contrast, the contours of horizontal strain shown on Figure 7.31 occur almost entirely in front of the shield face and within about 3m of the tunnel crown level. This pattern suggests that nearly all of the differential horizontal displacement occurs in advance of the shield. Further above the tunnel the horizontal strains are small but tensile in nature once the shield has passed the instrument line (x<0).

The contours of shear strain (γ_22) in the vertical plane parallel to the tunnel drive are presented on Figure 7.32. These show straining which is focused within 1 tunnel diameter of the tunnel crown. Straining is first observed between 6 and 8m in front of the shield edge and grows rapidly in magnitude with tunnel advance. Once the leading edge of the shield is past the instrument line the strains change very little, with contours becoming nearly horizontal. The limit of the 3-dimensional heading influence on the ground response near the tunnel is probably marked by the position when the shear strains are no longer affected by tunnel advance; this is about x=-7m. A line of zero shear strain can be inferred which coincides with the level of the gravel-clay boundary at offsets behind the tunnel face. The patterns of response in Figure 7.32 demonstrate that shearing strains are focused within about one tunnel radius above the crown which may mark the potential zone of plastic deformation for the soil above the crown.

Mair (1979) presented heading tests on model tunnels in kaolin to evaluate the sensitivity of critical stability ratio (N_c, see Chapter 2) to tunnel geometry (particularly the length of unsupported tunnel to diameter ratio, P.D.). Figure 7.33 shows a photograph of one such model test well after tunnel collapse. Lead-shot were used for displacement markers which were initially aligned both vertically and horizontally in a grid pattern. Vertical lines have been sketched on the figure to connect the lead shot and give profiles of horizontal displacements parallel to the tunnel axis, u, with depth.

Strain localisation (marked at A and B on the figure) is visible in front of the excavation, in the form of a planes slightly inclined to the horizontal which emerge from somewhere within the unsupported length of tunnel. The vertical profiles of u show a 'kink' at these levels, with very little differential horizontal movements both immediately above and below. For the westbound tunnel, a similar pattern of displacements is indicated from the profiles of horizontal movement with depth presented on Figure 6.80 (inclinometer B2i), which may imply the formation of shear bands or slip surfaces through the clay near the crown.

7.2.3 Westbound tunnel - transverse profile

Contours of vertical and horizontal displacements after westbound tunnel construction are given on Figures 7.34 and 7.35. The vertical displacements are focused immediately above the westbound tunnel crown and some asymmetry is noted above the tunnel crown (e.g. -25mm contour). The horizontal displacement contours on Figure 7.35 are concentrated near the tunnel axis level with large movements inward towards the excavation. Asymmetry in the horizontal movement contours both above and near the tunnel is also apparent. At distances exceeding two diameters above and to the side of the tunnel both the vertical and horizontal displacement...
magnitudes reduce significantly. The contour patterns are similar to those presented by Deane and Basset (1996) for the Heathrow Express SCL trial tunnel reproduced in Figure 7.36 for two different methods of tunnel excavation. The horizontal displacement contours appear much more concentrated near the spring line at St. James's Park than at Heathrow.

Contours of vertical strain magnitudes shown in Figure 7.37 emphasise the accumulation of disturbance near to the tunnel extrados. Tensile straining is seen above the tunnel crown associated with movements into the excavation and possibly some dilation, while compressive vertical straining is seen to occur at the tunnel spring lines. The contours of horizontal strain magnitudes given on Figure 7.38 are essentially opposite in sense to the vertical strains, with compressive strains occurring above the tunnel crown, and tensile strains focused at the spring line.

Vectors of displacement give a good representation of the overall mechanics of deformation as shown in Figure 7.39a; the implied mechanism is not dissimilar to those proposed by Mair (1979) in his lower-bound calculations for tunnel stability, or those observed by Seneveratne (1978) for centrifuge testing of tunnels in clay shown in Figure 7.39b.

Contours of shear strain magnitude, shown in Figure 7.40 show a 'butterfly wing' pattern broadly similar to the four lobes given for the idealised model on Figure 7.24. Larger shearing strain magnitudes (over 1.2%) are seen near the tunnel shoulder than at tunnel knee level. The contours emerging from the shoulder level show a tendency to flatten (i.e. become more horizontally oriented) with increasing distance from the tunnel extrados.

Major and minor principal strain rosettes presented in Figure 7.41 generally show a radial pattern near to the excavation, with directions of minor principal strain (generally tensile) focused on – or near to – the tunnel centre, and major (compressive) strain directions oriented circumferentially, indicating an arching mechanism around the tunnel. The strain magnitudes (represented at the lengths of each line) are largest near to the tunnel.

Contours of minor (and primarily tensile) principal strain direction relative to the horizontal are given on Figure 7.42; superimposed are lines indicating the similar contours for a perfectly radial orientation of tensile strain. Near the tunnel extrados the minor principal strain direction is radial. At levels above the tunnel crown and at positions offset from the tunnel centre-line, the direction of the minor tensile strain become less radial, and shows a tendency to become more aligned with the horizontal. This is seen by following the observed 45° contour and the 45° radial line, both of which emerge from the right shoulder of the tunnel on Figure 7.42. The two lines diverge beyond about 1 tunnel diameter from the tunnel extrados. Continuing along the radial line, the grid values are smaller indicating a shallower angle between the horizontal and the minor (tensile) strain direction.

The contours of major (\(e_1\)) and minor (\(e_{III}\)) principal strain magnitudes given on Figure 7.43 and 7.44 are concentric in nature around the tunnel, but are elongated horizontally. The values of major principal strain range between 0 and +0.76, and those of minor principal strain vary between -0.49 and 0%.

Contours of volumetric strain magnitude shown on Figure 7.45 indicate a small and
contractive change of less than 0.25% in the zone close to the tunnel, extending from the crown around to the below axis level. This calculation matches very well with slightly larger trough volume (about 0.3% larger) at the tunnel crown of the westbound tunnel shown on Figure 7.12a. Outside the zone immediately around the tunnel extrados, the volumetric strains indicate dilatant behaviour reaching values of about -0.15%.

Contours of maximum shear strain magnitude determined from the Mohr’s circle of strain (see Figure 7.23) are shown in Figure 7.46; these show a pattern similar to the principal strain magnitudes, but with a ‘bulging’ of the contours at the tunnel shoulder vertically upward towards the surface. Maximum shear strains range from 0 to 1.3% but are generally less than 0.2% at distances exceeding two tunnel diameters from the extrados. Similar shear strain patterns have been observed by other authors (Mair, 1979; Eisenstein et al., 1988) although their data generally showed a much greater tendency to ‘stretch’ upward towards the ground surface.

7.2.4 Eastbound tunnel - parallel profile

Contours of vertical displacement on the eastbound tunnel centre-line during construction are shown on Figure 7.47. The contours are aligned vertically at +7m in front of the tunnel face indicating that the movement magnitudes are similar at all depths. Between x=0 and -5 (i.e. above the shield) the contours rotate indicating differential settlements with depth. Above

The horizontal displacement contours presented on Figure 7.48 show initiation of horizontal movement about 10m in front of the shield. A displacement ‘bulb’ immediately above the shield edge is seen where the parallel profiles of u along the x-axis for a given depth can be thought to peak (see the idealised profile for u parallel to the tunnel drive on Figure 2.10). The contours become sub-parallel to the horizontal after the shield passes the instrument line (x<-7m). Only small changes in both vertical and horizontal displacement magnitudes are seen at all depths once the shield face is about 15m past the monitoring point. Comparing the horizontal displacement patterns for the east and westbound tunnels (Figures 7.28 and 7.48) show that larger horizontal movements are seen at most depths in the clay when the eastbound shield edge moves beneath the instrument line (x=0) than were recorded for the westbound tunnel. The vectors of displacement presented on Figure 7.49 consequently show larger proportions of horizontal movement than observed for the westbound tunnel drive.

Contours of vertical strain magnitude presented on Figure 7.50 show tensile strains about 5m in front of the tunnel face. The contours are aligned sub-vertically in front of the tunnel face and rotate to become sub-horizontal as the shield moves beneath the instrument line; this rotation maps the growing strain magnitudes near the tunnel crown. Behind the shield face the contours are sub-horizontal indicating that only small amounts of additional vertical straining occurs after the shield passes the instrument line. Contours of tensile horizontal strain presented on Figure 7.51 are focused immediately in front of the shield crown, while above the shield the horizontal strains are slightly contractive.

Figure 7.52 presents contours of shear strain magnitude (γx2) during the eastbound
tunnel drive. Measurable strains are first seen about 10m from the shield face. The magnitudes grow and the strain contours stretch towards the ground surface with further tunnel progression. Once the shield passes the instrument line and the lining is erected (x=-7m) the strains cease to change and the contours become sub-parallel to the tunnel axis.

7.2.5 Eastbound tunnel - transverse profile

Contours of vertical and horizontal displacements after eastbound tunnel construction are given on Figures 7.53 and 7.54 respectively. The general patterns are similar to the westbound tunnel response with vertical movements focused on the tunnel crown and horizontal movements focused at the spring line. The horizontal movements on Figure 7.54 show a degree of asymmetry, particularly in the overlying granular material where larger displacements are seen on the south side of the eastbound tunnel. Nearer the tunnel the horizontal movements in the clay are marginally greater towards the north at elevations above the tunnel crown; the contours immediately adjacent to the tunnel appear nearly symmetrical (remembering that Gi is about 1m closer to the excavation than Ei, see Figure 5.2).

The corresponding vertical and horizontal strain contours are shown on Figures 7.55 and 7.56. The vertical strains are tensile at the tunnel crown and compressive at the sides of the tunnel. The absence of subsurface displacement measurements north of the eastbound tunnel (y>12m) precludes calculating the strain fields for much of the half-trough. The horizontal strains are opposite in sense to the vertical strains, with tensile straining at the axis level and compressive straining at the crown.

Displacement vectors around the excavation presented in Figure 7.57 indicate a mechanism of response which gives the impression of symmetry in the clay but shows asymmetry in the top 10m of granular material. The width of the inferred mechanism appears marginally narrower than that seen for the westbound tunnel (at least for the less disturbed north half-trough, y>0).

Contours of shear strain magnitude (γxy) around the eastbound drive are presented on Figure 7.58. There is clearly a focus of straining at the two shoulders as was observed for the westbound tunnel, although there appears to be less flattening of the contours with distance from the tunnel extrados around the eastbound tunnel. Strain magnitudes at the tunnel knee level are smaller than those at the shoulder level.

The rosettes of the two principal strain directions given on Figure 7.59 show strong radial and tangential orientations around the tunnel near the excavation indicative of an arching mechanism around the tunnel. The rosettes become less radially oriented with increasing distance from the tunnel.

Contours of the angle between the minor (and generally tensile) strain direction and the horizontal are presented on Figure 7.60. Also on the plot are lines which similar contours for an axisymmetric strain pattern. The contours show that the tensile strains are radial near the tunnel. Further away from the excavation the minor principal strain orientations are less radial in nature, becoming more aligned with the horizontal in the same manner as the observed around
the westbound drive (Figure 7.42).

**Figures 7.61 and 7.62** show contours of the principal strain magnitudes. The contours of major (and usually compressive) principal strain magnitude on Figure 7.61 are focused between the tunnel shoulders and axis levels and are slightly flattened (i.e. elongated horizontally). The contours of minor principal strain magnitude on Figure 7.62 show a similar pattern: the largest strains are seen at the shoulder level on the north side of the tunnel (away from the westbound tunnel). Compared with the westbound drive the contours spread more noticeably towards the ground surface.

The contours of maximum shear strain magnitude presented on Figure 7.63 show tightly grouped contours emerging from the tunnel shoulders, with a maximum strain magnitude of 1.2%. The strain pattern is similar to the westbound tunnel in that the overall pattern is somewhat flattened; generally the strain magnitudes away from the eastbound tunnel extrados are larger than for similar distances from the westbound. This seems to indicate a softer soil response to the stress changes at the tunnel boundary.

Volumetric strain contours drawn on Figure 7.64 indicate a small amount of contractant behaviour (maximum 0.3%) near the spring lines, but with very little volume change just above the tunnel crown.

7.2.6 *Comments on the mechanics of ground response around both tunnels*

The ground response patterns in front of the westbound tunnel shield are characteristic of a 'plug-like' response, where the soil in front of the tunnel face can be thought to move towards and into the advancing excavation face as a block. This type of mechanism would yield contours of shear strain in the plane parallel to the tunnel axis (y) which are focused at levels immediately above the tunnel crown as were observed on Figure 7.32, and would show horizontal strain patterns in the x-z plane similar to those observed above the westbound shield on Figure 7.31. The contours of horizontal displacement (u) parallel to the tunnel axis (Figure 7.28) suggest that movement of this 'plug' is initiated about 1 to 1.5 tunnel diameters in front of the face (5 to 8m), and that the zone over which significant differential horizontal movements occur is very near to the tunnel crown level.

At the sides of the westbound tunnel, the inclinometer mapping results in Appendix D and the variable electrolevel responses described in Section 6.7.2 imply that strain localisation occurs at levels near the tunnel crown, and to a lesser extent near invert level results. This response yields the flattened contours of maximum shear strain presented on Figure 7.46.

In Figure 7.33 it was shown that strain localisation has been observed in front of plane strain centrifuge models of tunnels in lightly overconsolidated kaolin; the soil was consolidated one-dimensionally from a slurry and would be essentially free of depositional features such bedding and fissuring. The vertical profiles of horizontal movement, u, parallel to the model tunnel axis are similar in nature to those observed above the westbound tunnel centre-line (Figure 6.80), and so a correlation may be drawn between the two.

For the eastbound tunnel, the patterns of response parallel and transverse to the tunnel
axis indicate a more continuous ground response than seen around the westbound tunnel. The general ‘plug’ type mechanism is not as well defined as observed around the westbound tunnel, and regions of strain localisation which occur at similar positions for both tunnels around the tunnel extrados in the transverse plane and face are less obvious.

Burland (1990) developed the notion of post-rupture strength in clays which helps to explain why these strain localisations occur. He observed that natural samples of London Clay and reconstituted samples of kaolin both drop from a peak strength to a quasi-stable minimum value strength. For London Clay this drop was achieved after a relative displacement of between 1mm and 3mm across the failure plane. Burland termed this strength after rupture the ‘post-rupture strength’, and differentiated it from residual strength which requires much larger relative movements to be achieved. He then showed results from quick undrained tests which failed prematurely on pre-existing fissures (Figure 7.65). The envelope for these and other quick undrained tests on samples with fissures, and the envelope for the post-rupture shear stresses acting on slip surfaces formed during testing, are in very close agreement as shown on Figure 7.66. This correlation is interpreted as relating the fissured nature of London Clay to its brittle post-peak behaviour. Burland et al. (1996) presented data from triaxial tests on four stiff clays to further investigate the post-rupture strength. They concluded that shearing under higher confining stresses leads to particle orientation immediately after rupture, resulting in the reduced effective strength envelopes. There is also some indication that the changes in the ratio of \(\tau/\sigma'_n\) from peak to post-rupture may be more significant for stiffer materials.

Thus the fissuring and bedding planes in the London Clay probably act as preferential features for strain localisation due to strain softening. The similarities with models of tunnels in kaolin might be the result of particle orientation which occurs in anisotropically consolidated samples (Sheeran and Krizek, 1971); this particle alignment is sufficient to make one-dimensionally consolidated kaolin prone to slip surface formation (Burland, 1990).

The patterns of time-dependent movement described later in this chapter (Section 7.4) suggest that between the westbound crown (-28.5m) and eastbound invert (-23m) there is a change in the material properties vertically upward within the London Clay. This depth range agrees well with the depth to a London Clay layer containing numerous silt and sand partings (see Figure 3.3) that was observed by Burland and Kalra (1986) and Burland and Hancock (1977) within 500m of the instrumented site. Changing properties probably reflects changing depositional conditions, which tend to give rise to stronger bedding features. The bedding and laminations may, in turn, result in more pronounced horizontal fissuring during the overconsolidation process which would contribute to the mechanism observed above and around the westbound tunnel drive.

7.3 Total and effective stress and pore pressure changes around eastbound drive

The measured total stresses and pore pressure changes at the spring lines (see Sections 6.11 to 6.15) are presented on Figure 7.67 along with the calculated total stress and pore pressure
changes around an idealised collapsing cylinder. Details of the cylinder model are found in Chapter 2, and the key parameters used for the calculations are summarised on the figure. The immediate lining support pressure ($\sigma_1$) was estimated from the measured lining loads presented on Figure 6.157. These suggest 25% of the initial mean total stress ($\rho$) assuming an in-situ coefficient of earth pressure at rest of 2. The measured total horizontal stress changes on Figure 7.67 agree very closely with the idealisation.

The pore pressure change calculations are made using three different hybrid models as follows:

- the dashed line is based on Schmidt (1992) who uses Henkel’s pore pressure parameter ($a'$) determined from triaxial compression tests to prescribe the pore pressure changes around the collapsing cylinder in the elastic and plastic zones. It is difficult to choose an appropriate value of $a'$ as it varies with both stress path and strain level, but for presentation purposes a value of $a'=-0.2$ is chosen here (appropriate, according to Schmidt, for a heavily overconsolidated clay);

- the dash-dot line is essentially the same as Schmidt’s but assumes an isotropic porous elastic material ($a'=0$);

- the dash-dot-dot line utilises Mair and Taylor’s (1992) method for incorporating non-linearity of the stiffness modulus $G$ (see Section 2.6), for estimating pore pressures in the elastic zone.

The measured changes in pore pressure at tunnel axis level for SP1 and SP2 ($y/R$ equal to 1.7) shown on Figure 7.67 are smaller near the tunnel extrados than those predicted for all of the three idealisations. At larger offsets the best agreement is seen for the assumption of an ideal isotropic elastic material (dash-dot line) where no pore pressure changes are predicted in the elastic zone. The incorporation of non-linearity for the stiffness modulus reduces the radius of plastic straining ($R_{pl}$) and overpredicts the measured pore pressures changes in the elastic zone.

The measured piezometric changes on the centre-line above the eastbound tunnel crown are compared with the idealised model in the same manner on Figure 7.68. These data show good agreement with Mair and Taylor’s model using non-linear stiffness and with Schmidt’s method (for $a'= -0.2$).

At both the crown and spring line, pore pressures are expected to drop as a result of unloading at the tunnel extrados, followed by recovery as the stresses are picked up by the lining. This response pattern was observed at Heathrow Express as shown in Chapter 2 on Figure 2.30. However, the differences in pore pressure response magnitudes between the spring lines and the crown probably reflect the different stress paths at each location as identified in Chapter 2.

Addenbrooke (1996) presented FEM results for a 34m deep running tunnel in London Clay ($D = 4.15m$). Figure 7.69a shows contours of stress level ($S$) for a modelled volume loss
of 1.4%. $S$ is the ratio of the current deviatoric stress to the maximum deviatoric stress at yield for the same mean effective stress; for $S>0.99$ the soil is considered to have reached yield. For $K_0=1.5$ the contour patterns show small zones of yielding which extend above and below the tunnel from shoulder and knee levels, with little or no yielding at the axis level at the spring lines. Thus the numerical modelling indicates that the soil at the spring line might remain within its elastic strain range, which implies only small pore pressure changes. The FEM indicates that the soil above the tunnel crown experiences more shearing which might be expected to give larger excess pore pressures, particularly if the soil is not isotropic. The observations of pore pressures around the eastbound tunnel at St. James’s Park supports the numerical patterns.

Figure 7.70 shows the measured horizontal effective stresses ($\sigma'_{xy}$) and horizontal effective stress changes ($\Delta \sigma'_{xy}$) with tunnel face position. The responses are both large and rapid at the spade cells nearest to the eastbound tunnel; changes in excess of -300kPa are seen at SC1/SP1 and SC2/SP2, both offset at 4m from the tunnel centre-line. The magnitudes of change reduce with increasing distance from the tunnel extrados.

For an undrained response (i.e. constant $\sigma'$) the large changes in horizontal effective stress at the spring lines must be balanced by changes in either the vertical effective stress ($\sigma'_v$) or the complementary horizontal stress parallel to the tunnel axis ($\sigma'_z$). The radial patterns of major and minor (i.e. compressive and tensile) straining around the eastbound tunnel illustrated by the rosettes of strain direction on Figure 7.59 indicate an arching mechanism induced by the excavation; the vertical effective stresses probably increase at the tunnel spring lines with the vertical compressive straining at the same location.

To the side of the tunnel at axis level, the horizontal ($\sigma_y$) and vertical ($\sigma_z$) stresses respectively correspond to the radial and tangential stresses; changes of these two stresses for an idealised collapsing cylinder are equal and opposite for elastic response ($\Delta \sigma'_x = 0$). In three dimensions, the complementary horizontal stress parallel to the tunnel drive ($\sigma_z$) would intuitively be thought to reduce near the tunnel opening. Lee and Rowe (1990) presented stress change patterns for some simplified 3-dimensional numerical modelling of an unlined tunnel in an isotropic undrained soil ($K_0=1$). Ahead of their tunnel at axis level, the removal of initial stresses resulted in large reductions in axial stress ($\sigma_x$), while at the crown and invert (and probably the spring line) the parallel stress increases due to arching. This pattern might also be expected from the idealised model of a collapsing sphere at the tunnel face presented in Chapter 2.

With these ideas in mind, the potential changes in the ratio of horizontal to vertical effective stresses around the eastbound tunnel are investigated for two scenarios:

(a) assume plane stress ($\Delta \sigma'_x = 0$) and undrained response (constant $\sigma'$): for this case it can be shown for the idealisation of a contracting cylinder that $\Delta \sigma'_z = -\Delta \sigma'_{xy}$ and hence the mean stress ($s'$) remains constant

(b) assume undrained response (constant $\sigma'$) and that changes in vertical effective stress equal negative one half of the transverse horizontal stresses change ($\Delta \sigma'_z = -\frac{1}{2} \Delta \sigma'_{xy}$)

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\[ = -\Delta \sigma'_{x} / 2 \]. This implies that the complementary horizontal stress direction, \( \Delta \sigma'_{y} \), also changes by \(-\Delta \sigma'_{x} / 2\) to maintain constant mean effective stress.

The effective stress paths near the eastbound tunnel for these two stress response scenarios are shown on Figure 7.71 in \( t' \) and \( s' \) space; the paths are calculated from the measured changes in total horizontal stress and pore pressures at the tunnel spring lines at spade cell SP1 near to the tunnel extrados, where the largest stress changes during eastbound construction were measured. The other spade cells show similar patterns but with smaller magnitudes of change from the initial stress state. Also shown on Figure 7.71 are the residual strength envelope, the fissure strength envelope, and the upper-bound peak strength envelope (in compression) presented by Hight and Jardine (1993) for intact London Clay at similar depths.

The stress path for the plane stress assumption is vertical (\( s' = \) constant) and shows rapid positive changes in deviatoric stress (reflecting the unloading from high initial horizontal stresses) beginning when the tunnel face is within about 1 diameter (\( x=5m \)) of the instrument line; this continues until the shield edge is about 5m past (\( x=-5m \)). The deviatoric stresses reverse at this point, as a result of the lining expansion, and reduce to the end of construction. The path is shown on Figure 7.71 to just reach the upper-bound failure envelope.

For second assumption, positive changes in deviatoric stress and reducing mean stresses are first recorded when the shield is 5m from the instrument line. A maximum deviatoric stress is deduced when the shield is about -5m, after which the stress path reverses as the shield moves away from the instrument line. The path reaches both the residual and fissure strength envelopes.

In practice the soil is likely to follow an intermediate stress path between these two idealised scenarios. The plane stress scenario probably represents an upper-bound for the shear stresses, and it can therefore be inferred from these paths that plastic yield of intact London Clay at 4m offset from the tunnel centre-line at the spring lines (i.e. the position of SC1) is unlikely. However, it appears that the shear stresses are probably sufficient to mobilise the full strength on both fissure and residual sliding surfaces.

Deduced changes in the horizontal to vertical effective stress ratios (\( \sigma'_{y} / \sigma'_{z} \)) are shown on Figure 7.72 against eastbound tunnel face position for the same two assumptions above; for comparison between the four spade cells, \( K_{0} \) was assumed equal to 2 and the measured changes in horizontal effective stress were then applied to this base value. \( K_{0}=2 \) is the average ratio of \( \sigma'_{y} / \sigma'_{z} \) from the total stress cells measurements determined at the end of Period 1 prior to any tunnelling at the section (see Section 6.11).

The two different stress change assumptions described above yield broadly similar patterns of response, with the stress-ratio magnitudes for a given spade cell position differing by between 0.3 and 0.7 at any tunnel face position; the changes tend to be larger in magnitude for the plane stress assumption.

About 15m in front of the tunnel face the ratios of \( \Delta \sigma'_{y} / \Delta \sigma'_{z} \) increase slightly for all instruments. With the shield about 5 to 7m from the instrument line the ratios peak at values
between 2.1 and 2.5 for all cells; some delay in the peak is seen at SP4 (furthest from the tunnel) when compared with the instruments closest to the tunnel. The peak is followed by a rapid reduction as the tunnel moves through the instrument line. Values of $\Delta \sigma'_y / \Delta \sigma'_z$ reach between 0.1 and 0.6 at SP1, SP2, and SP3 when the shield edge is about 5m past the line; SP4 shows ratios of between 0.8 and 1.3. The smallest ratios are given for the plane stress scenario.

The ratios begin to increase gradually once the shield is about 7m past the section and the lining has been erected at the instrument line. The ratios of $\Delta \sigma'_y / \Delta \sigma'_z$ when the shield is over 40m past the section ($x=-43$) are between 0.5 and 1.2 at SP1, SP2 and SP3. At SP4 the values are higher at between 1.3 and 1.9. This final pattern is broadly similar in form to those seen for numerical analyses of a 30m deep tunnel in London Clay presented by Addenbrooke (1996) and reproduced on Figure 7.69b. The tunnel construction effectively disperses the initial high horizontal stresses at the spring lines; this result is reflected in part by the lining load build-up presented in Section 6.16.2 which show lower horizontal lining loads than vertical.

**7.4 Time-dependent displacements and tunnel lining response**

The subsurface vertical displacement observations presented in Sections 6.6 to 6.10 typify the time-dependent response after the construction of each tunnel. The movements generally occur more widely across the site than for construction-induced displacements. For transverse profiles at subsurface horizons, the largest increments of vertical displacement are offset from the tunnel axis, except at levels very near to the tunnel extrados where the maximum increments occur at the tunnel centre-line.

In this section the time-dependent displacements are considered at subsurface levels which were unaffected by the seasonal influences nearer the surface. The changing trough shape is investigated by evaluating variations with time of the trough width parameter, $i_v$. The pore pressure and total stress changes with time after each tunnelling event and the in-tunnel monitoring results of diametric change and lining loads are related to the observed displacement patterns.

In order to illustrate some differences in the overall time-dependent response between the two tunnels, the vertical displacements on the centre-line of each tunnel are shown for selected depths against log time on Figures 7.73 and 7.74. After westbound tunnel construction the magnitudes of response on the westbound centre-line vary with proximity to the tunnel, with the largest movements recorded at the tunnel crown. The time-dependent settlement response on Figure 7.73 appears to be far from complete by the time the eastbound tunnel is constructed (after about 250 days).

The construction of the eastbound tunnel has a dramatic influence on the time-dependent response trends for anchor Bxl (shallow depth), while at the deeper anchors only a small increases in the settlement rate with time are seen. Nevertheless, all three trends imply a change in the consolidation pattern. That is to say that the eastbound tunnel is not necessarily influencing the consolidation immediately above the westbound tunnel crown by acting as a
drain, but that excess pore pressures were generated during eastbound construction near the
westbound tunnel which are reflected in the subsequent changes in time-dependent displacement
responses.

On the eastbound tunnel centre-line, the settlements with log-time presented on Figure 7.74 show very similar displacement trends at the three depths considered, with slightly larger
settlements for the anchors nearest the tunnel crown. The magnitude of movement for Fx1 (near
the surface) at 250 days after eastbound tunnel completion is between two and three times that
seen above the westbound tunnel centre-line at anchor Bx1 (see Figure 7.73) for the same time
span after westbound construction. This difference between the two tunnels reflects both the
shorter drainage paths anticipated for a shallower tunnel and the more permeable soil at shallow
depths.

The magnitudes of movement with time can be broadly compared with those derived
from an FEM parametric study of tunnels in London Clay; the geometry considered is shown
on Figure 7.75. The tunnels were modelled for hydrostatic (H) and under-drained (U) initial
pore pressure profiles, with linear anisotropic permeability. Two tunnel sizes corresponding to
running tunnels (R, 6.5m diameter) and station tunnels (S, 12m diameter) were considered, and
for all instances the tunnel lining was modelled as fully permeable. A complete summary of this
and further work is given by Addenbrooke (1996).

The FEM results presented on Figure 7.76 for a running tunnel and a hydrostatic pore
pressure profile show that the total vertical displacements at the ground surface might reach
twice the construction movements after 1 year, and nearly three times the construction
disturbance after 10 years. Above the eastbound tunnel (the comparable depth for the FEM
results) the increment of vertical movement after one year is about -20mm (nearly equal to the
-23mm observed during construction) and another -20mm of movement will probably occur in
the coming years.

7.4.1 General response across St. James’s Park

Long-term surface displacement contours from the contractor’s levelling data are presented on
Figure 7.77; the contoured data are from June 1997 – about 1.5 years after eastbound tunnel
construction. The surface monitoring point locations for these data are shown on Figure 5.5.
There is a clear difference on this figure between the area south of the St. James’s Park lake,
where movements in excess of -25mm are seen, and the area north of the lake where the time
dependent movements are negligible. These differences follow the contrasting settlement
magnitudes and volume losses between north and south of the lake discussed in Section 7.1
where much larger volume losses were observed south of the lake during construction.

Figure 7.78 shows (a) a simplified geology across the Park, (b) long-term settlement on
the eastbound tunnel centre-line (determined from Figure 7.77), (c) the distance between the
eastbound and westbound centre-lines, and (d) observations pertaining to the long-term tunnel
inglining performance. South of the St. James’s Park lake: the two tunnels are within 25m of each
other; the long-term surface settlement ranges from -15mm to -25mm; and the eastbound tunnel
lining over this length is 'damp to wet' (see Appendix I), implying a small but steady infiltration of water into the tunnel. North of the lake: the two tunnels are at least 40m apart (increasing to 80m at the north end of the Park); the long-term surface movements are less than -5mm (settlement) at all monitoring lines; and the eastbound tunnel lining is almost entirely dry, with very occasional damp patches.

These observations indicate differences between the soil-lining interaction between the tunnel lengths north and south of the lake. This is described further in Section 7.4.3 on tunnel lining performance.

7.4.2 Soil response after westbound tunnel construction

The changes in transverse trough shape with time above the westbound tunnel after construction are presented on Figures 7.79 and 7.80 which present profiles of cumulative displacement normalised by the centre-line settlement against offset normalised by tunnel depth, \( y/z_0 \). Two subsurface levels are considered: the first at -5m (a typical foundation level and below seasonal effects) and the second at -17.3m (about half the westbound tunnel depth).

At both levels, the trough shapes become wider with time relative to the profiles immediately after construction (shown as a bold line). Near the edge of the construction settlement trough (at offsets, \( y/z_0 \) of about 1) on Figure 7.79 both the displacement magnitude and the differential movement (slope) are increasing with time from the construction profile. This trend may be of little practical significance because the settlement magnitudes and slopes as a result of construction are very small.

Variations in the normalised trough width (\( i/y/z_0 \)) presented on the bottom of Figures 7.79 and 7.80 each trace an increase in \( i \) with time which reflects the widening trough shape. The values of \( i/y/z_0 \) increase by about 0.1 (equal to about 3m or between 25 and 30% of the initial values) after 250 days. Note that the trough width parameter values are all determined assuming a Gaussian curve fit as shown on Figure 2.16. Statistical analyses (not presented here) indicate that both the construction profiles and the longer-term profiles can be matched to a Gaussian form with similar errors on the curve fit (see Appendix H).

Figure 7.81 shows contours of vertical displacement in the transverse (y-z) plane 250 days after westbound tunnel construction. The displacement contours form a bulb which emerges from the tunnel crown, and a maximum movement of nearly -11mm. Ignoring the seasonal effects at the ground surface, the settlements are nearly constant with depth across the site (within \( \pm 0.5 \)mm) to about -17.5m. Below -17.5m, increased settlement with depth is seen on the westbound tunnel centre-line, while at positions offset from the tunnel the movements decrease with depth.

The corresponding contours of average vertical strain magnitude given on Figure 7.82 reveal a zone of tensile straining above the tunnel crown with a maximum strain magnitude of \(-0.2\%\). The rest of the soil mass is experiencing small compressive straining. At positions offset from the tunnel, there is some evidence of an increase in compressive strain magnitudes at levels below -25m. At the ground surface relatively large compressive strains are recorded which arise
from seasonal changes in the top few metres of soil.

The positions of the piezometers and spade cells superimposed on Figure 7.82 show that piezometers BP1 and BP2 are positioned within the zone of tensile straining above the westbound tunnel crown. The piezometric changes for these two instruments are re-plotted on Figure 7.83 along with the measured average strains between extensometer anchors near to the westbound tunnel crown (see key diagram).

The very rapid initial pressure recovery at BP2 occurs in parallel with compressive straining between anchors Bx7 and Bx8 which lasts about 15 days. After 15 days the strain pattern between Bx7 and Bx8 reverses trend, and after 250 days the net straining between these two anchors is zero. The initial compressive straining probably results from the expansion of the lining ring which reloads the soil; this action can be imagined as a stress path reversal, where the vertical stress is re-applied after having been removed during construction. It can also be envisaged that the same response may occur — to a smaller degree — at the tunnel invert, where the stress path on construction for the soil would be similar in nature to the crown. The softened soil seems to react initially by compressing, which is accompanied by positive pore pressure changes (i.e. the more rapid increase in pressures at BP2). However, a short time later the balance between compressive volume changes at the invert and crown and tensile changes (due to soil swelling as a result of net unloading) tips in favour of swelling. The reloading effect is very localised, as compressive straining above the crown on the centre-line is only recorded between the anchors within about 1 m of the tunnel crown.

Between anchors Bx5 and Bx6, and Bx6 and Bx7, the strains begin (and remain) tensile, and grow in magnitude during the 200 days of monitoring presented. These tensile strains are probably the result of soil swelling due to net unloading and the equalisation of negative pore pressures which developed during construction.

7.4.3 Soil response after eastbound tunnel construction

Normalised transverse profiles of vertical displacements after eastbound tunnel construction are presented on Figures 7.84 and 7.85. The pattern of trough widening with time is more marked than observed after westbound tunnel construction. Normalised trough width parameters \( i_z/z_0 \) calculated for both north and south half-troughs at \( z = -5 \text{ m} \) on Figure 7.84 increase significantly with time: the changes are much more rapid than were observed for the westbound tunnel. After one year, the trough widths are nearly double for the south half-trough near the eastbound tunnel, and about 1.8 times the initial value for the north half trough.

At \( z = -12.8 \text{ m} \) (Figure 7.85) the change with time of the normalised trough widths appears asymmetrical, with different trends for the south and north half-troughs. However, at this depth, the widths of both half-troughs also increase to about 1.8 times the initial values after one year from. Note that \( i_z \) is determined for the north half-trough using only three data points, all of which are within an offset of half the tunnel depth \( (z_0/2) \) from the centre-line. This yields narrower calculated trough widths than if data from larger offsets were also used.

The long-term analyses after eastbound tunnel construction must be regarded with a
degree of scepticism. It is shown in Appendix H that the profile after one year is far from the Gaussian form. Therefore, although the pattern of trough widening may be reasonable, the actual values $i_*$ estimated from these analyses perhaps are not.

The contours of vertical displacement and vertical strains above the eastbound tunnel are presented on Figures 7.86 and 7.87 at 280 days (extensometer survey no.73) after tunnel completion for comparison with the westbound tunnel. The displacement magnitudes with depth are similar for all levels above tunnel crown level indicating small amounts of vertical strain with depth. Sub-horizontally aligned contours of vertical displacement at levels near the tunnel axis level indicate a concentration of vertical straining to either side of the eastbound tunnel between the crown and invert levels. The superposition of on-going time dependent effects at the westbound tunnel draws the contours downward towards the westbound extrados.

The strain contours on Figure 7.87 show a zone of tensile vertical straining above the eastbound tunnel crown, with strain magnitudes generally less than 0.06%. The tensile region becomes wider with increasing distance above the crown, but the distribution is generally similar in shape to that seen above the westbound tunnel.

At the sides of the tunnel the contours delineate a zone of compressive straining which extends horizontally from the spring line across the site. The largest strain magnitudes are seen immediately next to the eastbound tunnel spring line (reaching up to 0.2%); immediately above the westbound tunnel crown, vertical compressive strains of 0.1% are noted. Consolidation near the westbound tunnel increases the compressive ground straining locally, drawing the contours downward because of the additive effect of time-dependent response between the two tunnels.

Superimposed on the plot of vertical straining contours in Figure 7.87 are the positions of piezometers and spade cells at the site. Piezometers FP1, FP2 and FP3 are positioned in the zone of tensile vertical straining above the eastbound tunnel crown. The piezometric response for these instruments are plotted on Figure 7.88 against time, along with the measured average strains between extensometer anchors near to the tunnel crown (see key diagram). Note that the strains are plotted as tension positive.

Tensile strains increasing in magnitude with time from tunnel completion are observed between all three extensometer pairs considered on Figure 7.88. The straining develops most quickly for spans Fx5-Fx6 and Fx3-Fx4, while at the intermediate span (Fx4-Fx5) the strain development with time is slower. These differences probably indicate a more rapid swelling of soil: (a) at the tunnel crown where the soil is most disturbed and unloaded vertically, and (b) at the top of the London Clay where the clay is likely to be more permeable because of fissuring. The straining trends of the intermediate span Fx4-Fx5 can be paired with the pore pressure recovery of piezometer FP2 which shows a small irregularity in the response after construction. The pore pressures rise initially after construction (i.e. in the grey area on Figure 7.88), but are followed closely by a pressure reduction which ultimately takes the measured change in pore pressure to a level below that induced during construction. The reason for this response is unclear, but it may be that the more rapid swelling of the soil both above and below this level is drawing pore water away and reducing the pressure head.
The longer-term pressure recovery at FP2 is much slower than at FP1, a result which probably reflects both the more fissured nature of the clay at the top of the unit, and the shorter drainage path for pressure equalisation.

The vertical strain pattern around the eastbound tunnel at the control site suggests that there is strong anisotropy in the clay permeability, with $k_h > k_v$, and that the tunnel is acting as a drain. Vertical and horizontal profiles of pore pressures immediately prior to construction, immediately after construction and one year after construction are shown on Figure 7.89. The pore pressures above the crown (7.89a) are seen to drop during construction from the near hydrostatic initial conditions; after one year the pressures have almost completely returned to their initial levels.

The pore pressure profiles horizontally at axis level from spade cell measurements show only small changes during construction on Figure 7.89b. However, after one year the pressures are substantially depressed in this area. This is thought to reflect the presence of silty or sandy horizons, and silt or fine sand partings, through which preferential drainage might occur. The vertical compressive strains seen this level (Figure 7.87) probably result from rapid consolidation of clay between and within these bands or partings, which has much shorter drainage paths. The settlement patterns around the westbound tunnel hints at a similar but less dramatic response, and may be attributable to the natural variation in horizontal and vertical permeabilities, without the presence of sand or silt bands.

### 7.4.4 Tunnel lining performance in relation to soil response

A segmentally lined tunnel in London Clay is generally expected to act as a drain, as the lining permeability is almost certainly greater than that of the surrounding ground. This presumption was verified in both tunnels at St. James's Park, where continued water ingress has been observed in small amounts. Appendix I summarises the qualitative observations of water leakage through the tunnel lining along the route of both tunnels from Westminster to Green Park. In the eastbound tunnel the section between Westminster and the St. James's Park lake shows larger amounts of leakage into the tunnel than at positions further northward in the park, where the tunnel is essentially dry.

This observation, in conjunction with the contours of long-term settlement across the Park presented on Figure 7.77, leads to several possible conclusions: (1) north of the lake the soil is much less permeable at the tunnel horizon; (2) the ring build quality is better at the north end of the park and thus provides better resistance to water ingress; (3) the initial pore pressure profiles are different.

A difference in the soil permeability is certainly a reasonable conclusion; it is shown in Appendix A and on Figure 3.3 that zones of more silty and sandy clay exist within the London Clay. The tunnel lines through St. James's Park broadly follow the north-south section across the London Basin given. Changes in basement bedrock level (top of chalk) north of the instrumented site (which show changes of about 50m over 2km. see Figure 3.1b) might control the level of different lithologies in the London Clay. This would mean that the unit through
which the tunnelling is performed could change without significantly changing the axis depth relative to ground level.

The ring build quality is also a significant factor in the overall time-dependent response. Packing (comprising thin wood veneer sheets) was often inserted between adjacent rings to assist with steering of the shield (as recorded in the daily works reports). This action could significantly increase the tunnel permeability relative to the clay, although the observations in the tunnels indicate that water ingress is primarily at and around the key segments which were infilled some time after lining erection.

FEM modelling of long-term displacements into tunnels in London Clay presented on Figure 7.76 show that differences in the initial pore pressure conditions (from hydrostatic to underdrained) could significantly alter the long-term response. The JLE SI data (not presented here) did not appear to show any significant difference between the pore pressures across the Park, but most of the piezometers were installed at levels above the westbound tunnel axis level.

To relate the long-term ground response at the instrumented section to tunnel performance, measurements of horizontal diametric changes in the eastbound tunnel presented in Section 6.16.1 are re-plotted on Figure 7.90 along with measured changes in vertical length between extensometer anchor pairs which span the tunnel horizon. The key diagram shows the anchor levels, and the approximate positions of the anchors to either side of the tunnel. After eastbound tunnel construction the diametric changes (open symbols) are initially larger in magnitude than the measured changes in span length for the extensometers, but after 100 days the changes are nearly identical in both magnitude and response trend. Swelling of the soil above the tunnel crown is likely to be a contributing factor to the difference at an early stage. The strong similarities in response between the two measured quantities suggest that the squating distortion of the tunnel in this case is controlled primarily by the vertical straining of the soil at the spring lines as a result of consolidation and drainage into the tunnel. This result is perhaps not surprising; O'Rourke et al. (1984) emphasised that the soil around the lining ring acts as a structural element of the tunnel, providing passive pressures which enable an expanded segmental lining to stand as a continuous structural ring when wedged into contact with the ground.

The vertical diametric change after 280 days can be estimated to be about -16mm (from the horizontal diameter changes presented on in Section 6.15.2) while the maximum settlement at the crown from the displacement contour plot on Figure 7.86 is -20.4mm. This would suggest that about -4mm of displacement (settlement) has occurred at the tunnel invert level – a result which is consistent with the estimated displacement contours (drawn as dashed lines on Figure 7.86) beneath the tunnel.

The large radial distortions (nearly 0.5% radial strain) of the eastbound tunnel raise the question of lining performance. Of concern for expanded segmental linings are the position of the thrust line for circumferential stresses, and water tightness. The line of thrust is unlikely to be of great concern as the completed lining is a constrained arch which will benefit from
the thrust line for circumferential stresses and water tightness. The line of thrust is unlikely to be of great concern as the completed lining is a constrained arch which will benefit from increased lateral confining pressures as the tunnel squats. However, the water tightness may be jeopardised as larger radial deformations can result in unacceptably high lining permeability.

In the westbound tunnel, measurements of diametric change did not begin until over 200 days after construction. Figure 7.91 presents the measured changes in vertical length between extensometer anchors pairs which span the tunnel horizon, and the recorded horizontal diametric changes presented in Section 6.16.1. The key diagram shows the anchor levels, and the approximate positions of the anchors on either side of the tunnel. The extensometer spans over the first 200 days after westbound tunnel construction show changes of between 7 and 8mm (compressive). The data are then re-zeroed at 212 days to coincide with the start of diametric measurements.

During the next 600 days, the changes with time for both the extensometer anchor spans and the horizontal diameter are broadly similar to each other, and parallel the general pattern seen for the eastbound tunnel on Figure 7.90. At nearly 650 days after tunnel construction the horizontal diameter is about 5mm larger than initially measured, while the compressive changes in anchor span are at about 8mm (from the re-based date). If it is assumed that a direct correlation exists between the anchor span changes (as seen for the eastbound tunnel), the horizontal diametric elongation is estimated to be about 13mm.

The traces of measured hoop stress in the eastbound tunnel lining presented in Section 6.16.2 show greater thrust vertically (i.e. at the spring lines) than horizontally. This result conforms to the mode of time-dependent response where vertical compressive straining is focused at the spring lines and the vertical diameter is shortening.

Time-related changes in circumferential loads are presented on Figure 7.92. This plot shows lining loads measured in segmentally lined tunnels in London Clay at two different locations for 1 week and 1 year after lining erection. Compared with the hoop stresses measured for a tunnel of similar size and depth at Regent’s Park (Barratt et al., 1994), the loads in the eastbound tunnel lining at St. James’s Park are initially 20% larger (in terms of the percentage of overburden stress) at 1 week. After 1 year the measured loads increased for both sites, but compared with Regent’s Park the St. James’s data still are about 10% larger after 1 year. The similar gain in loads may suggest that a similar pattern of time dependent response to the eastbound tunnel presented in this dissertation was occurring around the northbound tunnel at Regent’s Park, although observations pertaining to the tunnel lining performance need to be considered as well. Barratt et al. showed much higher vertical lining loads than horizontal loads (see Figure 2.31) which also agree with the St. James’s Park data.
8 Conclusions

The field measurements above the twin Jubilee Line Extension running tunnels in St. James’s Park were specifically aimed at characterising the greenfield response pattern for comparison with monitored structures nearby so that the effects of tunnelling on the urban environment could be investigated. The ground responses were to be compared with the empirical methods for predicting movements, pore pressure and stresses, and with some of the FEM modelling results for a similar tunnel geometry which had already been published. The performance of each monitoring system (i.e. precision, accuracy, stability, etc.) was to be evaluated for both rapid ground response during tunnelling and slower time-dependent response after construction. Subsequently, the project goals extended to explaining why these tunnels caused much larger settlements than have been observed above other tunnels built in similar ground conditions. While this last question has not been fully answered, the field measurements at the instrumented section in St. James’s Park have proved essential in identifying probable causes, and have given direction for future work in the park.

This final chapter comprises summary statements of conclusions and lessons about instrumentation and monitoring of tunnelling-induced displacements; ground response above and around tunnels in London Clay; displacement development during construction; and longer-term ground response. The final section identifies further research which might be undertaken to improve our understanding of the ground response at St. James’s Park.

8.1 Conclusions and lessons learned from instrumentation and monitoring of tunnelling-induced displacements

The surface and subsurface monitoring described in Chapters 4 and 5 utilised several systems of varying sophistication to obtain the 3-dimensional components of movement above and around both tunnels at St. James’s Park. Sub-millimetre precisions were achieved during the stable period prior to westbound tunnel construction for measurement techniques which focused on obtaining a single displacement component at the ground surface (i.e. the collimation tool for $u$, micrometer stick for $v$, and precision level for $w$). A precision of better than 1.5mm was recorded for 3-dimensional displacement monitoring using total station surveying; this could have been further improved if relative movements along the SMP line were evaluated (i.e. assuming one of the SMP points to be stable). With few exceptions, the final transverse surface displacement profiles (of $u$, $v$ and $w$) determined from different monitoring techniques during tunnel construction agreed to within ±1mm.

The subsurface vertical displacement monitoring using rod extensometers showed sub-millimetre precision for measurement at all depths. The overall precision for measurement depends on the surface levelling to the extensometer reference heads, except perhaps for extensometer Dx which had a deep extensometer below the zone of influence for both tunnels.

The elec...
performed well during the construction events. However, changing from an auto-logged computer-driven monitoring system with very regular energisation of electrolevels to a less frequent manual recording method affected the time-dependent response for many electrolevels by changing the apparent rotation. The variability and magnitude of these effects prevented the time-dependent response from being quantified for both Periods 3 and 5, and hence the development of long-term horizontal displacements was indeterminable. The more conventional monitoring tool, an inclinometer torpedo, was used prior to tunnelling beneath the site, and again one year after tunnel construction to map the changes in verticality for every inclinometer tube. These data were useful for evaluating the horizontal movements near the tunnel, and movements which occurred after tunnel construction.

A critical review of the monitoring systems leads to the following comments and conclusions related to the monitoring performed at St. James's Park:

**Datums**

For all surface and subsurface displacement monitoring, successful measurement of movement and interpretation of the results relied closely on the datum or reference points chosen. Vertical movement measurements for precision levelling were made relative to a nail located in the pavement near to the instrumented site. The levels were subsequently adjusted assuming that the deepest extensometer anchor (Dx7, z = -40m) was stable for all monitoring at the park. The total station surveying (and the collimation measurements) relied on reference points affixed to nearby buildings: these were shown to be stable relative to each other during construction of each tunnel (Appendix B). Consideration of the apparent movements for stable points within the total station survey enabled the errors in positioning instruments relative to the datums or movements of the datums themselves to be assessed, quantified and corrected if necessary. For long-term monitoring, differences between the magnitudes of vertical movements recorded for total station surveying and for precision levelling demonstrate that the reference points for the total station surveying were affected by seasonal changes.

The choice of a datum was more difficult for the horizontal displacements components parallel and transverse to the tunnels. Interpretation of the micrometer stick measurements assumed a fixed point at the end of the SMP line. This was suitable for the short-term construction response as the levelling showed the end of the SMP line to be stable during each tunnelling event. For long-term monitoring, the micrometer stick measurements and the total station surveying show excellent agreement for the relative transverse horizontal movements, v, along the SMP line. However, differences in the displacement magnitudes between the two methods imply that the last SMP had moved, or that the reference points for the total station surveying were affected by seasonal changes.

For parallel horizontal movements, u, the long-term results are slightly more onerous to assess. The analysis of the total station survey sensitivity in Appendix B showed that reference point movements due to seasonal changes might give apparent profiles of u displacements at the SMP line which are different for the different total station locations (i.e. ST1 and ST2). This
gives a lower degree of confidence in the differential movements across the SMP line than for the transverse horizontal movements, but with a similar uncertainty about the absolute displacement magnitudes along the SMP line. The interpreted profiles for Period 3 and Period 5 after each tunnelling (Figures 6.29 and 6.52) show variations in both horizontal components of movement of about ±2.5mm between the different techniques after one year.

The use of more reference points within the total station survey would probably improve the precision for long-term monitoring and perhaps make the survey less sensitive to seasonal movements. This was impractical at the instrumented section as very few buildings are visible through the trees.

**Instrumentation layout**

The vertical and horizontal displacement magnitudes change rapidly with depth at levels near the tunnelling horizons for both construction events. For subsurface vertical movements, there were not enough measurement points near the tunnels to define the entire profile. In particular, it would have been useful to have anchors positioned at elevations between the tunnel crown and invert where the displacement patterns change most rapidly.

At the inclinometer holes near the tunnels, the distances between adjacent electrolevels were too large to give adequate resolution of the vertical profile of horizontal displacements for the desired degree of accuracy. For better results, the electrolevel spacing in the inclinometers less than one tunnel diameter from the spring lines should have been reduced (i.e. smaller gauge lengths used). The patterns of response around both tunnels suggest that closer spacings should be adopted (a) from just above the crown to below the tunnel shoulder and (b) from the tunnel axis level to below the invert, where the ground strains are largest.

The bottoms of each inclinometer were generally assumed stable for each tunnelling event when the vertical profiles of horizontal displacement were determined. Near the eastbound tunnel the inclinometer tubing should have been deeper to improve confidence in this assumption.

**Monitoring tools**

The monitoring tools and techniques were varied in terms of practicability and practicality. Monitoring vertical displacements using precision levelling and rod extensometers was straightforward (taking two people about 40 minutes for each) and provided the most stable, repeatable and reliable results. The measured data were quickly reduced to understandable movements which could quickly be used to assess the tunnelling performance.

The micrometer stick surveys could be done by a single person, with one traverse taking about 60 minutes from start to finish. The measured data could be quickly reviewed to give a rough idea of the horizontal movement profile, but temperature corrections were essential to determine accurate movement magnitudes. The instrument was cumbersome (more so than, say, the levelling staff) but was reliable and repeatable. The temperature sensitivity of the bar was a characteristic which resulted in cumulative errors when calculating the displacement profiles.
The temperature effects could have been reduced significantly if the bar had been made of a plastic with a low coefficient of thermal expansion (note: expansivity one-third of aluminum can be obtained from readily available plastics); this would have also made the bar lighter and more easily manoeuvrable, although perhaps not as robust as the aluminum bar. The integrated circuit which indicates contact between the micrometer tip and the ball seating was influenced by moisture, and could not be used in wet weather without careful shielding of the instrument and the ball seat.

The collimator tool gave measurements of the parallel horizontal displacement profiles along the SMP line and each survey took two people about 70 minutes to complete. The measurement precision for the collimation tool was related closely to the accuracy with which the total station could be located above the station location (i.e. ST1 or ST2, marked by a nail in the pavement). Slightly better repeatability was seen for the collimation tool than for the total station surveying. However, the collimation tool could not be used during rainy conditions as the digital vernier callipers were not designed for outdoor use, nor could the tool be used at night.

The total station surveying took two people about 50 minutes to complete a full traverse from one station location. The displacements could not easily be determined from the raw measurements (i.e. angles and distances relative to the reference points). Instead, the movements could only be assessed after a careful review and analysis of the measurements (as described in Chapter 4). The final results are 3-dimensional movements of each SMP. Measurements to the reference points were difficult at night but would probably have been easier if larger reflective targets had been used.

This being the first time that a field monitoring project was specifically aimed at measuring horizontal displacements above tunnels, it was important that redundant measurements were made using different monitoring systems. The mechanical measurement of strains using the micrometer stick is a more preferred primary monitoring tool but – in the opinion of the author and considering the success of the total station monitoring at the Park – a carefully designed total station survey would give adequate results for the least amount of effort. The value added by using the collimation tool or the micrometer stick would be to verify the total station measurements. All three methods are subject to problems with datums over long time periods.

The electrolevels in the inclinometer holes provided measurements for rotation during each tunnelling event which appeared to be accurate when compared with the borehole mapping results from the inclinometer torpedo. Changing the logging system before and after each tunnelling event was undesirable, but necessary. A permanent auto-logging system would have been too costly for the research project (the extra costs being for cabling, a permanent computer, multiplexers, and ground works to bury the cabling), while manual monitoring during each tunnelling event was impractical as each manual traverse took over two hours. In retrospect (and without adopting a single system for logging), it would have been better to map the boreholes with the inclinometer torpedo during the periods before, between and after the two tunnels: the
Electrolevels could have been installed a week or two before each tunnelling event and autologging could have begun immediately prior to tunnelling. It would have also been useful to take very frequent manual readings after the systems were changed to obtain a better idea of the horizontal movements from the electrolevels.

Improvements in the design and manufacture of electrolevels (Rasmussen, 1998) give promise of instruments which are very stable over long time periods. This development might lead to deep horizontal datums comprising inclinometer tubing and electrolevels for which the surface variations can be deduced assuming the base of the tubing is stable. Surveying onto the top of this tubing could provide better horizontal control for total station measurement.

8.2 Conclusions relating generally to tunnelling in London Clay

The observed volume losses at St. James's Park of 3.3% ($w_{max} = -20.4\text{mm}$) above westbound tunnel and 2.9% ($w_{max} = -23.4\text{mm}$) for the eastbound tunnel are larger than those recorded for previous tunnelling projects in London Clay using similar construction techniques. At both Regent's Park (Barratt and Tyler, 1976) and Green Park (Attewell and Farmer, 1974), only a few kilometres north of St. James's Park, the volume losses were less than 1.5% for open-faced tunnelling shields of similar diameter. Volume losses for surface profiles of less than 1.3% were recorded for the Heathrow Express running tunnels (Barakat, 1996). It is interesting that both the JLE running tunnels (discussed in Chapter 5) and the Heathrow Express tunnels were excavated in London Clay using a similar shield at a similar range of depths, excavated at similar speeds, supported with a similar expanded lining, and were even built by the same contractor.

The large volume losses at the site become even more of an enigma when compared with the monitoring data obtained by the JLE contractor. Their data show that volume losses in the Westminster area for the westbound tunnel are over 3%, but that 200m north of the instrumented section (and on the north side of the St. James's lake) the volume losses reduce to between 1.5% and 2% ($w_{max} \approx 11\text{mm}$). For the eastbound tunnel, the contractor's data show volume losses of about 1.4% ($w_{max} \approx 10\text{mm}$) north of the lake. It is worth noting that the tunnels are far apart (over 60m between centre-lines) at the north side of the Park.

The long-term displacement patterns also vary along the route through the Park; after eastbound tunnel construction dramatically larger increments of vertical displacement are seen south of the St. James's Park lake compared with the area north of the lake.

**Variations in volume loss and long-term response along the route**

Investigating the ground response and the possible reasons for the larger volume loss and variable long-term response revealed the following:

1. The daily works records for the westbound tunnel drive suggest that the overall excavation procedure was not closely controlled in the Westminster area; only after the tunnel shield moved past the instrumented site were there systematic procedures for face
excavation.

(2) The considerable reduction in volume losses recorded north of the St. James’s Park lake above the westbound tunnel coincide with changes in the construction method. The tunnelling contractor was investigating different tunnelling procedures to determine the most suitable way of progressing beneath important and sensitive structures further to the north in light of the large volume losses observed at the instrumented site.

(3) The observed response at the instrumented site for the eastbound tunnel construction were large because of the influence the previously constructed westbound tunnel had on the soil. The smaller volume losses recorded above the eastbound tunnel north of the St. James’s lake are associated with increased distance between the two tunnels, thus reducing the potential interaction.

(4) Inspection of the eastbound tunnel showed that the amount of water ingress becomes markedly less as one moves northward from Westminster area beneath the lake towards Green Park (see Appendix I) which implies that the lining-soil interaction is changing along the route. The westbound tunnel inspection revealed damp patches along the entire westbound route, with a slightly wetter zone seen north of the Park.

(5) The average undrained strengths at the westbound tunnel horizon (determined from UU tests on 38 and 100mm samples, and from SPT ‘N’ correlations) increased by about 50kPa from the Westminster area south of the lake to the St. James’s area north of the lake (Appendix A).

These observations indicate that work quality, ground characteristics and recent stress history all play a role in the large volume losses and variability in observed response along both drives, and the differences in long-term settlement and tunnel lining performance.

It seems probable from the data presented in this dissertation that the large volume losses recorded above the westbound tunnel during construction are primarily the result of low undrained strengths in the London Clay south of the St. James’s Park lake in the Westminster area, in combination with the practise of excavating in advance of the shield and leaving a larger length of unsupported tunnel.

The profile of $s_u$ with depth from the JLE site investigation presented on Figure 3.5 shows a nearly constant strength profile with depth between $z = -23$ and $-34$m (i.e. westbound tunnel crown level) in the Westminster area: these strengths are somewhat lower than observed at other London Clay sites for similar depths. Larger strengths at this same level are recorded north of the lake which corresponds with the reducing volume losses for westbound tunnel construction.

Burland and Kalra (1986) related this zone of constant undrained strength with depth to the presence of silt and sand partings: low strengths from the UU tests resulting from water in the silt and sand partings being absorbed by the clay on sampling, which increases the water content. If these silt and sand partings are present above the westbound tunnel crown, the response of the soil may also be partially drained during construction. The undrained strength
of the soil could be reduced if the negative excess pore pressures which develop during unloading at the crown dissipate during the face excavation and shield advance. The greater length of unsupported excavation during the westbound drive in the Westminster area means that the soil at the crown was unsupported for a longer time period before the lining was erected, and thus more movements could occur than for tunnelling north of the lake.

For the eastbound tunnel, the ground displacement patterns confirm that the first tunnel construction had a significant influence on the ground response magnitudes observed for the second tunnel. This is simply illustrated on Figure 8.1 which shows the whole transverse profile observed for eastbound tunnel construction; superimposed on the south half-trench is the mirror image of the north half-trench. The ratio of the two half-trench volumes \( \frac{V_1(\text{south})}{V_1(\text{north})} \) is about 160%. Thus, earlier westbound tunnel construction resulted in significantly larger volume losses for the later eastbound tunnel.

The observed volume loss for the whole transverse settlement trough of 2.9% is not considered representative of what would have been recorded for greenfield conditions at the site. Instead, a value of 2.2% (i.e. the equivalent whole-trough volume loss based on the less disturbed north half-trench) is a good upper-bound estimate, with the same tunnel in undisturbed soil being potentially less. North of the St. James’s Park lake, the reduced volume losses recorded above the eastbound tunnel (less than 1.5%) are accompanied by an increase in the distance between the two tunnels. The diminishing influence of the westbound tunnel on the eastbound construction probably contributes greatly to the measured changes in volume loss, although differences in the lithology may also be a factor as the tunnel performance in the long term is very different on either side of lake.

**Mechanics of ground response**

For westbound tunnel construction, the overall response pattern indicates a plug-like movement of the ground into the advancing face; strain localisation (possibly in the form of sub-horizontal slip surfaces or shearing bands) is evident. In the plane transverse to the tunnels (\( y-z \) plane) similar strain localisation sub-horizontally is also seen; this is implied by the vertical profiles of horizontal displacements obtained by borehole mapping (given in Appendix D), and the flattened contours of maximum shear strain magnitude which tend to focus at the tunnel shoulder level. Similar but less dramatic patterns are observed for the eastbound tunnel.

The possible presence of silt and sand partings in the clay at the westbound tunnel crown level might reflect more variable depositional conditions than for other units within the London Clay, which would tend to give rise to stronger bedding features. The bedding and laminations may, in turn, result in more pronounced horizontal fissuring in the clay during deposition and removal of overburden, which could contribute to the mechanism observed above and around the westbound tunnel drive.

**Soil strength parameters for design**

The largest shear strains determined from the field measurements in the transverse \( y-z \) plane
are less than 1.5% for both tunnels and are accompanied by strain localisation at the shoulder level. These strains can be compared with undrained triaxial extension tests for samples of London Clay. Stress-strain curves (deviatoric stress, \( t \), against axial strain) show larger variability in the axial strain magnitudes at failure, but have a lower-bound of between 1 and 2%. The lower-bound results probably represent failure on existing surfaces or features in the tested sample.

If 1.5% maximum shear strain (i.e. the largest value determined from field measurements) is hypothetically taken as the point at which a shear surface forms in a test sample at 45° to the vertical, it can be shown (assuming constant volume and typical sample geometry) that the axial strain required to generate that maximum shear strain would be approximately 2%. The lower-bound strains for failure determined from triaxial extension tests closely match the calculated maximum strains around both tunnels. It suggests that early failure of samples in the laboratory is indicative of the bulk response for larger scale stress changes in the field. Thus, the strength on fissures or bedding features determined from laboratory testing is most appropriately used for tunnel design rather than the peak strengths for intact samples.

Further support for this is given from the measurements around the eastbound tunnel where the maximum shear strain magnitudes calculated were similar to those for the westbound tunnel, and similar – but less well defined – shear banding was inferred from the inclinometer mapping and the electrolevel responses during construction. It was also deduced from the total horizontal stresses and pore pressures measured near the eastbound tunnel (with some simplifying assumptions) that the changes were probably sufficient to mobilise the full fissure strength but were unlikely to be large enough to mobilise the peak strength.

### 8.3 Conclusions relating to characterisation of construction displacements above tunnels in London Clay

The ground response observed above and around the tunnels at St. James's Park is reasonably well described by the empirical framework presented in Chapter 2 for assessing the ground displacements due to tunnelling at surface and subsurface levels. Some notable aspects are discussed below.

**Transverse and parallel vertical displacement profiles**

The idealised forms of a Gaussian curve for the transverse profile and a cumulative probability curve in the longitudinal section give reasonable fit to the measured field data above both tunnels at St. James's Park, although a single curve cannot be fitted to the asymmetrical eastbound profile. The profile above the westbound tunnel and the less disturbed north half-trough above the eastbound tunnel are not exactly Gaussian; estimating the trough width parameter (\( i_g \)) from these measured data for the different definitions of \( i_g \) (which all coincide for the idealised Gaussian curve) results in differing predictions of potential damage (see Appendix H). The commonly used plot of the natural logarithm of normalised settlement (\( \ln w/w_{max} \)) versus offset squared (\( \nu^2 \)) to determine \( i_g \) gives potentially unconservative estimates of damage parameters for
structures (i.e. maximum slope, deflection ratio) when compared with those determined directly from the greenfield settlements actually measured. Nevertheless, for the purpose of comparison this conventional method is used in this dissertation to relate the different tunnel responses and to identify the relative trends.

The magnitude of trough volume ($V_s$) and the centre-line settlement ($w_0$) were shown in Chapter 7 to increase correspondingly during tunnel advance beneath the site for both tunnels, while at the same time the trough width parameter ($i_y$) was observed to reduce in magnitude, indicating that the trough shape became narrower. This indicates that centre-line settlement is the key parameter for relating the settlement magnitudes to the overall disturbance (i.e. $V_s$ or $V_f$). but a fixed relation between the three parameters (as defined in Chapter 2) is perhaps not appropriate for developing settlement profiles. The change in the trough width probably reflects the 3-dimensional influence of the tunnel heading on the overall mechanism of ground response, which changes at the SMP line as the shield moves beneath and past.

The calculated values of the trough width parameter ($i_y$) with depth above the westbound tunnel showed good agreement with case history data and Mair et al.'s (1993) relationship for $i_y$ with depth. Above the eastbound tunnel, the values of $i_y$ were larger than expected; this is probably the result of the excavation of the eastbound tunnel in disturbed ground which resulted in a wider subsidence trough.

The narrowest (and potentially most damaging) trough is the final profile after tunnel construction. Using the trough width parameter for this final profile and the observed variation of $V_s$ with tunnel advance in the empirical correlation between $i_y$, $V_s$, and $w_0$ (Equation 2.15) would lead to conservative estimates for the potential damage from the developing transverse troughs above both tunnels.

Movements around the two tunnels at St. James's Park in comparison with the field data presented by Mair and Tabor (1992) appear in poor agreement when the normalisation of movement and distance from the tunnel centre by tunnel radius is applied. There is much improved agreement by normalising the movements by both radius and observed volume losses (Figure 7.20).

The observed ratios of trough length to trough width parameter ($i_x/i_y$) are less than unity at surface and subsurface levels for both eastbound and westbound tunnels. The commonly used assumption of unity gives unconservative predictions of maximum slope and deflection ratios for potential building damage (see Table 7.3) for the parallel profiles, although the transverse displacement profile is still potentially the most damaging of the two. This result might be specific to this site (i.e. a result of the large volume losses and the mechanics of the ground response), but damage in the longitudinal profile should be considered carefully for future tunnelling works under urban areas.

**Volume losses with depth**

The transverse trough volumes above each tunnel were shown to be reasonably constant with depth except at levels very near to the westbound tunnel crown where they were about 10%
higher than at the ground surface. This shows that dilatant volume change anticipated above tunnels in overconsolidated London Clay does not occur or does not influence the trough volume calculations for construction induced settlement.

**Horizontal movements**

Horizontal displacements and strains at the ground surface and in the granular material overlying the London Clay were shown to be much larger than predicted from the empirical framework, resulting in vectors of displacement at the surface aimed at half the depth to the tunnel axis. This has potential significance for empirical damage calculations for near-surface services such as gas and water mains, and sewers (Bracegirdle et al., 1996) which could be underestimated if the surface vectors are assumed to be aimed at the tunnel centre-line. At the gravel-clay interface the horizontal movements formed a much smaller component of the overall displacement than at the surface. At greater depths in the London Clay (i.e. nearer to the tunnel crown) the horizontal movements again become more significant. Appendix G reviews the horizontal displacement data in the subsurface and presents a simple improvement to predict more closely subsurface horizontal displacements. The proposed relationship given as

$$\frac{v_{x}}{w_{x}} = \frac{y}{\zeta(z-z_{0})}, \zeta = 2$$

forms an upper-bound limit for horizontal displacements from case history field data and model test data for overconsolidated clays. Other recent investigations have been undertaken which should help to establish appropriate relationships for soil at shallow depths, particularly for layered ground (Grant, 1998).

**8.4 Conclusions relating to characterisation of long-term displacements above tunnels in London Clay**

In Chapter 7, the trough width parameters determined from time-dependent settlement profiles at different depths above both tunnels were shown to increase with time. In Appendix H it was shown for the westbound tunnel that the maximum slope and the maximum deflection ratio in hogging determined directly from the field measurements in the transverse profile reduced slightly between the end of tunnel construction and 250 days after tunnel completion. Similar changes are estimated from the empirical framework using the calculated trough width parameters from best-fit Gaussian curves to the long-term data.

For the eastbound tunnel, the trough width parameters were shown to increase more rapidly with time than for the westbound tunnel. These magnitudes of change are somewhat questionable as the form of the settlement profile in the long-term above the eastbound tunnel becomes less Gaussian with time, and the results should be viewed with some scepticism.

Comparisons between the long-term and construction profiles after the eastbound tunnel drive show that the maximum slope calculated directly from the measured settlements has increased with time on the undisturbed north half-trough, while the maximum observed
deflection ratio in hogging has reduced slightly.

The long-term patterns of response after both tunnels vary considerably, which leads to the conclusion that the time-dependent response, particularly the patterns of movements above and around the tunnels, is very specific to the tunnel and can only be assessed with detailed knowledge of the ground conditions at the tunnel boundary.

8.5 Further work
The primary focus of further work at St. James’s Park should be directed at resolving the causes of (a) the large construction disturbance, (b) the variability along the route, and (c) the varying long-term response across St. James’s Park. To this end, further work is required to obtain in detail the geological profiles and key engineering parameters across the Park.

Cored boreholes two north, and two south of the St. James’s lake
The presence or absence of silt and sand partings at the eastbound tunnel horizon seems the most likely reason for the variations in long-term ground movements along the route of the tunnels through the Park and the differences in tunnel performance (in terms of water ingress). They may also go a long way to explaining the construction response at the control site.

The borehole positions should be reasonably close to the tunnels to be confident that the logs are relevant to the observations of lining performance. Ideally, four holes would be completed, two north of the St. James’s Park lake and two south. Some of the holes could be taken to the top of the underlying chalk formation, so that changes in top level of the ‘basement’ rock level may be delineated. The basement structure may be a controlling factor on the unit of London Clay through which the tunnelling is being performed, and may also influencing the pore pressure patterns in the clay and in the underlying aquifer.

Piezocone testing
Carefully controlled piezocone testing will give supplementary information on undrained strengths to complement the JLE site investigation data, but would be used primarily to characterise permeability variations with depth. It is envisaged that frequent measurements of excess pore pressure dissipation through the London Clay (with the elevations to be determined from the corings) will delineate the presence of more permeable layers for which estimates of the relative permeability may be made. These results can be correlated with the borehole core logs.

High quality thin-walled undisturbed samples
Samples should be obtained over the range of tunnel depths for later study, and be as large as possible to capture the natural fabric of the clay. The clay samples should be tested for stress paths which adequately mimic the changes in the ground around the tunnels and be instrumented locally to obtain appropriate small strain stiffness parameters. Strain-controlled tests, for which the appropriate straining components and rates could be selected from the displacement patterns
observed near to the tunnels at St. James’s Park, might also be feasible with further analysis of the field data.

**In situ horizontal ground stress measurements**

In situ stress testing could be undertaken to verify the original stress state and any change across the length of the park. Possibly the directionality of the horizontal stress should be considered, as the direction of tunnel progress changes along the route relative to the regional trends of geological structure (i.e. the axis of the London Basin, see Figure 3.1 and Appendix A).

**In situ pore pressure measurements**

Piezometric measurements would be useful to investigate potential differences in the clay along the route as a result of under-drainage from the deep aquifer. The area is rather anomalous in terms of the recovery of pressures in the lower aquifer, as shown on Figure 8.2. The centre of water level depression in the lower aquifer is the area of St. James’s Park and Westminster. However, the pore water profile with depth in the clay at St. James’s Park and in Westminster (Figure 3.11) does not show strong underdrainage, at least at tunnelling depths, which might be expected from the depressed water levels beneath the clay. In the CIRIA report about the implications of ground water rise (Simpson et al., 1989) it is stated that ‘...the vertical effective stress \( \sigma'_v = \sigma'_l \) will generally reduce by an amount equal to the increase in pore pressure, \( u \). Hence, the shear strength on horizontal surfaces in the soil will also reduce significantly.’ This may be a factor contributing to the mechanism of ground response at the westbound tunnel face and should be investigated.

**Numerical analyses**

It would be very useful if, prior to any laboratory testing of clay samples (if obtained), numerical analyses of the tunnelling works could be undertaken to identify the most suitable stress paths (see for example Ng and Lo, 1985). Ideally this would be achieved using 3-dimensional finite element methods with realistic pre-yield soil models (incorporating non-linearity of stiffness), or Fourier-series assisted axisymmetric analyses which can feasibly accommodate non-axisymmetric stiffness variations and initial stress conditions.

There are some useful parametric studies which could be undertaken using the FEM which would complement the measured results. One could be to investigate the influence of different ratios of horizontal to vertical permeability on the long-term consolidation patterns for idealised tunnel linings. For typical tunnel geometries, this would give some guidance for potential damage to overlying structures from time-dependent movements. The analyses could be calibrated with the field data collected at the instrumented site and with the data from the contractor’s monitoring points north of the lake.

The second parametric study would be aimed at establishing how appropriate is the approach of normalising displacements near tunnels within the framework of the collapsing cylinder model. The data from the Park indicate that the normalisation suggested by Mair and
Taylor (1992) has limited application. The analyses would be useful for investigating the appropriateness of normalising displacements by both volume loss and tunnel radius as proposed in Section 7.1.4, particularly for different sized tunnels and tunnels of different depth. At the moment the field data in Figure 7.20 show that the normalisation may have some promise.

**Long-term monitoring**

Long-term monitoring has continued at St. James’s Park with financial support from CrossRail to cover the cost for the licence agreement with the Royal Parks in 1997 and 1998. A deep datum founded in the chalk unit beneath the Lambeth Group and Thanet Sands would be a valuable contribution to the success of the long-term monitoring at the site for the coming years.
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Appendix A
London Clay: investigations into fabric, structure and variability

The London Clay has been highlighted as having a strongly laminated fabric in most instances which is often exemplified by discrete partings and lenses of more silty clay, silt, and silt to very fine sand; indeed the JLE SI core logs noted silt and sand partings with moderate frequently. Burland and Kalra (1986) established ground conditions for the Queen Elizabeth II conference centre only 200m east and south of the St. James's Park site (see Figure 3.2). They observed a zone of clay marked by frequent silt and fine sand partings at between -20m and -27m (depth) which was similar to that seen at the House of Commons car park (Burland and Hancock, 1977).

In this first part of this appendix, clay samples obtained at the site during instrument installation (i.e. cuttings from the clay cutter and a single U4 sample above the westbound tunnel) are reviewed to define qualitatively the presence of more silty and sandy horizons in the clay at the instrumented site. Their presence may have a significant influence on the clay behaviour in both the short- and long-terms (Burland and Hancock, 1977). It was also thought useful to reinvestigate the available clay cores from the boreholes near the park to obtain an appreciation of lamination within the London Clay at tunnelling horizons and whether there are differences between Westminster and Green Park areas.

The second part of this appendix looks at the presence of sub-vertical jointing and random fissuring both sub-parallel and sub-normal to the nearly horizontal bedding planes. Terzaghi (1936) stated that ‘such minor structural features are characteristic of over-consolidated clays’ and that ‘the overall strength of these stiff, fissured clays could be as low as one fifth to one tenth of the strength measured on small samples of intact clay’. Later investigations reported by Webb (1966) and Bishop and Little (1967) showed a marked influence on both the undrained and drained strengths of London Clay. Bishop (1966) showed that peak strength reduced with sample size, a result which was attributed to lower effective strengths along the discontinuities. Sandorini (1977) investigated the deformability characteristics of London Clay with fissures through tests on clays from two sites where the fissure spacings were very different. His results showed for very finely spaced fissures the sample size had little effect on the normalized undrained secant moduli at 50% of the maximum deviator stress, $E_u(50)/P_c$; for a second site there was a tendency for the normalised modulus to increase with increasing sample size. Costa-Filho (1980) compared intact London Clay and samples with a single fissure, and concluded that the presence of a discontinuity could yield an apparent Young’s modulus 10% smaller than in the laboratory. He noted that the effects of fissures on the deformability in field situations may be more significant, particularly if the fissures open as a result of stress relief.

Recognising that the structural fabric could play a key role in the ground response around a tunnel, a review of the records of discontinuities made periodically by JLE engineering works inspectors was undertaken by the author to assess the structural fabric in the clay near to
St. James’s Park. Measurements of dip direction and dip angle were recorded for over 150 discontinuities in the eastbound tunnel and nearly 80 in the westbound tunnel. The measurements from both tunnels are first presented, from which the prevalent structural fabric is estimated. Comparisons with other data on the London Clay fabric are also made.

**Samples obtained during instrument installation**

**U4 Sample from BP1 (-25.1m to -25.6m)**

This sample was taken from the bottom of piezometer borehole BP1 on the westbound tunnel centre line. It is a very stiff, horizontally and thinly laminated, highly fissured grey to grey-brown silty clay. Smooth planar horizontal fractures parallel to the laminations are found at close spacings (<10cm). The sample contains three sub-vertical fissures (dip angles of between 75° and 85°), as well as numerous smooth and locally polished conchoidal randomly orientated surfaces. Some of the fissure surfaces appeared to be coated brown, or very occasionally black.

Horizontal reddish brown to light brown silty fine sand partings sub-parallel to the bedding and up to 1mm in thickness are seen at close spacings (<10cm) over the sample length. A 2-3mm thick fine sand parting is present at the mid-length of the sample, extending through the entire sample girth; the clay on the upper side of this parting is sandy to very sandy silty clay for a thickness of between 1cm and 2cm.

At the top of the sample there is a lens of orange brown clayey silty fine sand; a small grey nodule (possibly calcareous) with a yellow dusting (sulphurous?) is also present within this sand pocket.

**Samples taken from clay cutter tool during instrument installation**

Samples were taken from the clay cutting tool during the boring of two holes (A1 and D1) approximately 14m apart and on either side of the westbound running tunnel centre line. The samples were inspected visually to assess the general characteristics of the London Clay. The relevant observations are summarised as a borehole log on Figure 3.3d.

The samples indicated two zones in the London Clay (for the 40m below ground level investigated) in which the presence of silt and very fine sand are more prominent. The upper zone is between -22.5 and -27.5m while a deeper zone is seen starting at about -33m and extending to at least -40m depth.

**Cores from JLE Boreholes 115T and 104T**

Borehole 115T is located approximately 500m east of the instrumented site on the embankment at Westminster. Borehole 104T is positioned north of St. James’s Park and south-east of Green Park station. Both were rotary cored through the London Clay and later mapped using
geophysical techniques. A full description of the geophysical mapping results for Borehole 115T are included in Chapter 3. The cores were re-investigated in April 1998 with an aim to explore more thoroughly the occurrences of sandy and silty partings in the clay.

Figure A1 shows the logs for the two boreholes; lists of the partings recorded in each hole during the original JLE SI along with additional data collected in April 1998 are given on the figure. The majority of the London Clay is laminated in appearance and contains a significant proportion of silty material uniformly distributed and as very small lenses; this is reflected in the gradings (see Chapter 3) which showed up to 50% silt to very fine sand in the deeper basal beds. In each of these two boreholes a distinct thickness of clay is noted where sandy silty partings were observed with frequency. The partings were predominantly a mixture of clay, coarse silt and very fine sand; some of the thicker bands were laminated and many of the partings contained pyrite nodules.

The tunnelling levels in St. James's Park indicated on the plot demonstrate the different lithologies through which the tunnels were potentially excavated and the changes at the tunnelling horizons from Westminster to the area north of the park. Further discussions on this is given in Chapters 3 and 7.

Observations at the tunnel face during westbound and eastbound tunnel construction

Information on features within the London Clay

Randomly oriented, curved and conchoidal fissures of low persistence (10 to 100mm) were found in many London Clay cores. These were predominantly sub-horizontal (parallel to bedding) or sub-vertical. The horizontal fissuring is generally closely spaced (60mm to <20mm) but reduces in frequency with depth. The sub-vertical fissures (or 'joints' after Skempton and Petley, 1967) were observed about every 5 to 10m vertically within the clay cores, occasionally with some striations.

There are only a few well documented observations of these features in situ. Near to St. James’s Park, Burland and Kalra (1986) showed a zone of highly fissured clay at -28m to -32m (depth) below which the clay showed a more silty and less fissured nature.

Fissuring, both sub-parallel and normal to bedding, and sub-vertical jointing were observed by Skempton et al. (1969) at several London Clay sites around the London Area (Wraysbury, Buckinghamshire; Edgwarebury, Middlesex; and Bradwell, Essex). The fissures were noted as having smooth or ridged surfaces with matt surface texture and undulating shape. They had a random orientation of strike but showed a clear tendency for low angles of dip (less than 20°) together with a minor grouping of high angle dips. The fissures generally extended less than a few inches. A small proportion (<5%) showed slickensided or polished surfaces indicating prior movement. The fissure frequency was noted to reduce with depth while the mean size of fissures was seen to increase in the deeper clay samples. Contours of the poles to
planes for the fissures at Edgwarebury (about 12m depth) are given Figure A2a.

Sub-vertical joints observed in the walls of the Wraysbury borrow pit were essentially planar, matt textured surfaces. Skempton et al. associated them with the 'backs' noted by Ward et al. (1959) which were often blamed for large over-breaks in tunnel excavations in London Clay. Their data showed principal dip directions of 030/210° and 120/300°. Planar or slightly undulating sub-horizontal joints of considerable size have also been seen in London Clay at Edgwarebury, Middlesex and at Bradwell, Essex.

Discontinuities observed in-tunnel during construction of the Heathrow Express were reviewed by Woodhouse (1996). Contours of the poles to planes for both the sub-vertical and sub-horizontal features are shown in Figure A2b; this plot is analogous to Figure A2a and shows a close relationship between the two sites.

**Interpretation of JLE face logs**

The terms ‘fissures’ and ‘joints’ were used interchangeably to describe sub-vertical features cutting across bedding planes; measurements of dip and dip directions were estimated for these relative to the known strike of the tunnel drive. Other ‘fissuring’ was generally described as horizontal (parallel to bedding), planar and closely spaced. The horizontal fabric is likely to be a combination of bedding features associated with deposition of the soil, and fissuring arising from differences in laminae response during the overconsolidation process (see Skempton et al., 1969). This fabric may be a controlling feature in the soil response but very few references are made in the face logs regarding sub-horizontal discontinuities.

The quality of observations varies significantly between face logs (and particularly observers) and thus care is necessary in interpreting the observations. The discontinuity observations (i.e. dip direction and dip angle) were estimated at a short distance from the tunnel face and are therefore prone to error. It was also indicated that the fabric was often concealed or marred by smearing of the face from the mechanical digger.

In order to assess the general fabric in the area with confidence the measurements need to be representative of the overall properties. This relationship is difficult to quantify because the account of discontinuities is almost certain to be observer dependent (in the absence of a rigid methodology for recording), and may be substantially influenced by the activities of construction and the ground response. In the time available to make records, most observers will be biased towards larger and more noticeable features in the soil mass, although it may be argued that it is these very discontinuities which are likely to be controlling the ground response.

**Face observations**

During westbound tunnel construction from Waterloo to the southeast corner of St. James’s Park very few face logs were completed by JLE inspectors, and only 21 joints and/or fissures were recorded. Once the tunnel passed the instrumented section both the face log frequency and the number of observed discontinuities increased significantly. In total 78 discontinuities were observed during westbound construction. The dip angles presented in a histogram on Figure
A3a show a large proportion (>65%) of steep sub-vertical dipping features with a smaller grouping of sub-horizontal discontinuities. Figure A4a shows a rose diagram of the dip directions for all discontinuities. Two clear preferential groupings are seen oriented at about 027/207° and 117/297°.

Records for the eastbound tunnel drive are, in general, more comprehensive than for the westbound drive with more frequent logging of the tunnel face. The face logs between Waterloo and St. James's Park contain over 60% of the observed joints and fissures. This is thought to be the result of much slower tunnel advance along Great George Street, where grouting was being performed. The dip angles shown on Figure A3b have a strong bias towards the vertical; nearly 90% of all discontinuities observed dip between 70° and 90°. Notably there are virtually no horizontal fissures recorded during the eastbound tunnel drive. A rose diagram for the dip directions observed during eastbound construction is given on Figure A4b for 154 observed discontinuities. The dip directions have a bimodal distribution similar to that observed for the westbound tunnel, oriented toward 015/195° and 105/285°.

Contours of the poles to planes for both tunnels are presented on Figure A5. These are very comparable to the data from Heathrow and from Edgwarebury given on Figure A2.

Attempts were also made to investigate the variation in structure along the tunnel lengths by considering the observations made from Waterloo to the instrumented site separately from those north of the St. James's Park lake and towards Green Park; these plots are not shown here. The eastbound data show virtually no difference in predominant orientation of features. The westbound data show more randomness and a less well-defined orientation bias in the Westminster prior to passing the control site, but the overall trends remain similar for the two lengths of tunnelling.

**Conclusions**

The structural fabric assessed from observations of discontinuities at the face of the two running tunnels from Waterloo to Green Park are similar to those observed at other London Clay sites. For both tunnels the features show similar bimodal distributions of the vertical features. No distinct changes are seen in the overall structural fabric between Westminster and the area south of the St. James's Park lake and north of the lake towards Green Park for either tunnel.

**Variations along route from JLE site investigation**

Variations in the undrained shear strengths along the tunnel route are evaluated by reviewing the average SPT 'N' values and measured $s_u$ (from quick UU tests) at different subsurface horizons for two separate areas: (1) between Green Park and the St. James’s Park lake, and (2) between the St. James’s Park lake and the river Thames at Westminster (see Figure 3.2).

Figure A6 shows the variations in SPT 'N' recorded along the route. The black bars are data north of the park and represent the average 'N' value for each 5m depth range, while the black symbols indicate the number of tests which make up the average. The grey bars and symbols give are same information from south of the lake. The changes along the route for
depths between 0 and 20m below the top of the London Clay are small. However, a strong
difference of about 10 (approximately equal to 50kPa for a typical SPT 'N' correlation) is seen
at between 20 and 25m below the top of the London Clay.

Variations of (a) undrained shear strength, \( s_u \), (b) liquid limit, and (c) plastic limit are
shown on Figure A7. Again the black bars and markers are for data north of the lake and the
greyed bars and markers are for south of the lake. At depths below the top of the London Clay
of less than 15m the average \( s_u \) values north and south of the lake are generally within 15kPa.
At larger depths, the differences grow to between 25 and 40kPa, with higher strengths seen north
of the lake. The average liquid an plastic limits at each depth are similar north and south of the
lake except at 10-15m below the top of the London Clay. Here, higher liquid and plastic limits
are seen north of the lake, probably indicating a larger percentage of clay at the north end of the
Park.
Appendix B
Details of local coordinate system for total station measurements, and stability of reference points during monitoring

In this appendix the local coordinate system used for processing the total station measurements is described. Details of reference point coordinate calculations are given and the stability of each point within the coordinate system and relative to other reference points is discussed. Stability of the overall coordinate system is considered and potential uncertainties are quantified.

Local coordinate system
Five retro-reflective prismatic targets (numbered RP100, RP200, RP300, RP400, RP500) were affixed to buildings within view of the instrumented section as shown on Figure B1; the first three were seen only from ST1 and the latter two only from ST2. The five points act as `control' or `reference' points which were assumed to remain stationary for both tunnelling events beneath the site. RP100 and RP200 are located on the west facade of the Foreign Office at ground level; RP300 is a reflective surveying target on the west facade of the Treasury Building at the north west corner: RP400 is affixed at first floor level on the north facade (facing onto Horse Guards Road) of 23 Queen Anne’s Gate; and RP500 is affixed to the north-facing facade of the building at 2 Queen Anne’s Gate at first floor level. An arbitrary coordinate system denoted by \( r, s, t \) (where \( t \) is vertical and positive upward) was assumed with RP200 positioned at \( (300, 300, 300) \). The \( s \)-axis is formed by the line intersecting RP200 and RP300. This layout ensured that the calculated coordinates of the remaining reference points and the SMPs were positive for ease of processing.

Calculating the reference point coordinates
\( r, s, \) and \( t \) coordinates for the reference points were calculated for each total station survey in the following manner.

From ST1
(1) Horizontal angles measured between reference points were averaged for both instrument faces.
(2) Vertical angles from the horizontal plane (zenith angles) for each point were calculated for each face and averaged.
(3) Average horizontal distances from the total station to each point were calculated using the average zenith angle and the average measured inclined distance for both faces.
(4) The positions of RP100, RP300, and ST1 were calculated by triangulation using horizontal angles measured relative to RP200.
(5) The position of ST2 was established through steps 1-3 above using the measurements onto the tripod-mounted prism at ST2. Distance and vertical angle measurements from ST2 back to ST1 after exchanging the total station and prism were also used in
calculating the average horizontal distance between ST1 and ST2, and the zenith angle.

From ST2

(6) Steps 1-3 above were repeated for measurements to RP400 and RP500 from ST2.
(7) The positions of RP400 and RP500 were calculated by triangulation.

Where measurements were not made on both instrument faces, no vertical or horizontal collimation corrections were made.

Reference point stability

The stability of the reference points has been assessed in two ways: by reviewing the changes in calculated 3-dimensional coordinates with time, and by evaluating the relative stability within the two separate groupings (see Figure B1). The former assessment gives the variation in three dimensions relative to RP200 while the latter gives only the height difference and distances between points without any measure of relative rotation between them in plan.

Calculated coordinates

Figures B2 to B5 show the calculated \( r \), \( s \) and \( t \) coordinates of RP100, RP300, RP400 and RP500 with time. The average positions, \( \mu \), and the standard of deviations, \( \sigma \), for the coordinates of each reference point are summarised in Table B1.

The coordinates calculated for RP100 and RP300 are well conditioned with standard deviations less than ±1.5mm (excluding statistical outliers) for monitoring Periods 1-4. In Period 5, RP100 shows slight upward (\( t \) coordinate) movement with time with no change in the \( r \) and \( s \) coordinates. RP300 shows a well-defined downward displacement in Period 5, along with a decrease in the calculated \( s \) coordinate.

RP400 and RP500 show significantly larger scatter in the calculated \( r \) and \( s \) coordinates for Periods 1-3, particularly for RP500: better precision is seen for the vertical (\( t \) coordinate). The variability in calculated positions reduces considerably for surveys performed during Period 4. In Period 5 both points show large changes between surveys in calculated \( r \) coordinate, but small variation in the \( s \) and \( t \) coordinates.

Relative elevations and distances

The relative elevations and plan distances between the targets within each point grouping (i.e. between RP200 and both RP100 and RP300, and between RP400 and RP500) were derived from the calculated \( r \), \( s \) and \( t \) coordinates for each survey: these are shown varying with time on Figures B6 to B8. The mean distances and height differences, and the associated standard of deviations are summarized in Table B2.

Between RP200 and RP100, and RP200 and RP300, the distances and height differences are very well conditioned with standard deviations less than 1.4mm for Periods 1-4; these variations are similar in magnitude to the deviations for the calculated coordinates summarised in Table B1.
in Table B1. Small changes (less than ±1mm) are seen between the mean values of each period. However, during Period 5 the height difference between RP100 and RP200 appears to decrease slightly (see the top plot of Figure B6) while the distance between the two points remains constant; at RP300 and RP200 the height difference reduces and the distance between the two points increases markedly as shown on Figure B7.

Between RP400 and RP500 the calculated distance and height difference are very uniform with time, having standard deviations generally less than 1.6mm for all monitoring periods (see Table B2). This variability is well below that seen for the calculated coordinates relative to RP200.

The elevation differences and distances between the reference points depend only on the measurements from the nearest station (ST1 for RP100, RP200 and RP300; ST2 for RP400 and RP500) and these measurements appear repeatable and stable with time. The difference between the variability of the calculated coordinates of RP400 and RP500, and their relative positions can therefore arise only from errors introduced when sighting between ST1 and ST2, or from rotation or translation of the reference targets RP200 and RP300 relative to the initial local coordinate system.

**Errors in measurements between ST1 and ST2**

A trial was performed in November 1995 to check the tripod-mounted prism which is used for sights between the two station locations. Two temporary pins were positioned approximately 5m apart and the total station was accurately centred and levelled over one pin and the prism was centred and levelled over the other. The distance between the instrument and the prism was measured using the total station’s EDM. The prism (and its tribrach bracket) was twice rotated clockwise 120°, recentred and relevelled; distance measurements were made at each new position. The results of these trials are summarised in Table B3. For the three distance measurements a variation in excess of ±5mm was recorded; this is consistent with an error in the prism’s bubble of nearly 2° from the horizontal.

The bubble was adjusted by removing the prism from its tribrach and replacing it with the total station. The total station was levelled using its own fine adjustment bubble, after which the bubble on the prism’s tribrach was adjusted to conform.

The trial was then repeated and the results are given in Table B3; the three distance measurements have a maximum variation of ±0.5mm which closely resembles the instrument limitations.

**Survey sensitivity**

For analysis of total station measurements the reference points are assumed fixed during the site monitoring. However, buildings facades move as a result of diurnal variations in temperature or from seasonal changes in environmental conditions (Forbes et al., 1994). By fixing these points in space, real movement of these targets between surveys can result in apparent displacements of the SMPs. The survey at St. James's Park has no absolute confirmation of
point positions, thus making stability assessment of the local coordinate system difficult.

To obtain a rough estimate of the sensitivity of the survey geometry, some simple analyses were performed to evaluate both the potential magnitude and form of apparent displacements at the SMP line which arise from small relative movements of the reference points. It was assumed that movements of the reference points are adequately described by a rotation in plan about an arbitrary point and a translation of all points as shown schematically on Figure B9. This movement pattern is supported by the observation given above that the distances between control points were essentially stable during the majority of site monitoring.

By applying a specified rotation and translation, the magnitude and direction of reference point movements consistent with causing the same rotation and translation can be back-calculated. A transformation can then be applied to the positions of the SMPs to assess the apparent movement at the monitoring line.

To calculate reference point displacements, first let \((r, s)\) represent the initial calculated coordinates of any reference point, and \((r_c, s_c)\) describe the location of the centre of rotation. The horizontal distance, \(L\), between the centre of rotation and the initial monitoring point location, and the angle from the \(r\) axis to the line formed by joining the centre of rotation and the initial point location, \(\theta\), are calculated as follows

\[
\theta = \arctan\left(\frac{s-s_c}{r-r_c}\right), \quad \theta \text{ is positive anticlockwise} \tag{B1}
\]

\[
L = \sqrt{(r-r_c)^2 + (s-s_c)^2} \tag{B2}
\]

Figure B9 shows these quantities schematically. A rotation in plan, \(\phi\) (positive anticlockwise), and an overall translation \((\Delta r, \Delta s)\) results in new coordinates as follows:

\[
r' = L \cos(\theta + \phi) - \Delta r \tag{B3}
\]

\[
s' = L \sin(\theta + \phi) - \Delta s \tag{B4}
\]

A vertical translation, \(\Delta t\), can also be included to give a new \(t\) coordinate,

\[
t' = t + \Delta t \tag{B5}
\]

Thus displacements of the reference points are determined from the difference between the new and initial coordinates.

The process of calculating the apparent movement of the SMPs relative to the initial reference point positions is given diagrammatically in Figure B10: (i) shows the initial relative positions of two reference points and the monitoring point line; (ii) illustrates the 'real'
movement of the reference targets resulting from rotation and translation with no 'real' movement at the SMP line; and (iii) shows how the assumption of reference point stability manifests itself as apparent displacements at the SMP line.

The apparent displacements are, in fact, consistent with and equal, but of opposite rotation and translation, to that of the reference points about the same centre. Thus the apparent coordinates \( (r_a, s_a, t_a) \) are determined from:

\[
\begin{align*}
    r' &= L \cos(\theta - \phi) - \Delta r \\
    s' &= L \sin(\theta - \phi) - \Delta s \\
    t' &= t - \Delta t
\end{align*}
\]

where \( \theta \) and \( \phi \) are as detailed above. The apparent displacements can be determined from the difference between the apparent and the initial SMP coordinates. The coordinates can also be transformed to the \( x, y, z \) axis system given in Figure 2.8 and the apparent displacement in each direction calculated.

**Sensitivity analysis results**

Rotations of ±20 arc-seconds were applied and the corresponding movements of the other reference points determined. For simplicity, it was assumed that the point of rotation coincided with one of the reference point positions (either RP200, RP300, RP400 and RP500) and that there was no translation. The two reference point groupings seen from either end of the SMP line are considered separately.

**Figure B11a** shows the displacements components \( (r \text{ and } s) \) of RP100 and RP300 which equate to a range of rotation of ±20° about point RP200. Because all three reference targets lie nearly in a line and sub-parallel to the \( s \)-axis, no change in the \( s \) coordinates are seen for a rotation about RP200. **Figure B11b** shows the apparent movements in the \( x \) (parallel to tunnel axis) and \( y \) (perpendicular) directions for SMP nos.1 and 24 at either end of the monitoring line; SMP no.1 is the furthest from the rotation centre. The rotation about RP200 gives rise to larger \( x \) displacements than \( y \) displacements for both points, but for either point the apparent \( y \) displacements are the virtually the same.

**Figure B12a and b** shows the same for rotations about RP300. In this case the relative magnitudes vary between apparent displacements in \( x \) and \( y \) depending on the end of the SMP line being considered. However, again the apparent movements in the \( y \) direction are the same for both monitoring points.

**Figures B13 and B14** summarise the results for RP400 and RP500 as the centres of rotation: these two points are in a line nearly parallel to the \( r \) axis. **Figures B13a and B14a** show the calculated reference point movements equating to the rotation of reference coordinate system. Only small displacements are calculated for the \( r \) coordinate while much larger displacements are calculated for the \( s \) coordinate. **Figures B13b and B14b** show the apparent
and y displacements for SMP nos. 1 and 24. For either rotation centre a given rotation results in apparent x displacements which are larger in magnitude than the apparent y displacements for both SMPs. The y displacements are the same for a given rotation while there is a difference between point 1 and 24 in calculated x displacement.

Figures B15 and B16 show the calculated transverse profiles of apparent displacements in x and y (i.e. u and v) across the SMP line reference point movements consistent with an overall 8 arc-second rotation about RP200, RP300, RP400 and RP500. The differential movements of the other reference points are tabulated on each plot.

The apparent displacement profiles at the SMP line are generally all of the same form regardless of the rotation centre. Profiles of parallel horizontal displacement (u) consistently show a differential movement across the line of between 3 and 4mm, while transverse horizontal displacement profiles, y, show nearly identical movements right across the SMP line. Comparing rotations about RP200/RP300 and RP400/RP500 there is an opposite sense of movement which reflects the relative position of the SMP line to the centre of rotation. These profiles represent the sort of apparent displacement patterns which might be seen at the control site if the control points are moving relative to one another.

Conclusions
The relative stability within each reference point grouping is good throughout all monitoring periods, as are the calculated coordinates of RP100 and RP300 relative to RP200 indicating that the sights from each total station to the reference points are precise. The calculated coordinates of RP400 and RP500 for Periods 1-3 show large scatter indicating an error in the measurements between ST1 and ST2; improvement in the precision of calculated coordinates for RP400 and RP500 was observed in Period 4 (see Table B1) appears to coincide with adjustments made to the tripod-mounted prism near the end of Period 3.

Because of the error in sighting between ST1 and ST2, the analyses of total station measurements were undertaken for each instrument position separately, and the calculated displacements compared and contrasted as independent displacement measurements.

Investigation into the potential influence of reference point movements shows that significant apparent displacements at the monitoring line can be calculated if the reference points on the surrounding structures move. The transverse profiles of apparent displacement at the SMP line are shown to follow a pattern where the magnitudes of horizontal movement (v) perpendicular to the tunnel axes are similar for all points, while a linear profile of horizontal movements, u, is shown. Apparent u displacements are generally the largest in magnitude, with the SMP most distant from the station position (and the rotation centre) being considered moving the most.
Appendix C

Temperature calibration testing for micrometer bar

Temperature-controlled calibration tests were performed at Imperial College to assess the thermal sensitivity of the micrometer bar used to measure horizontal strains between adjacent monitoring points. In this appendix the calibration set-up is described and the testing procedures summarised. Some typical test results are presented and a coefficient of thermal expansion derived from the interpreted data.

Test apparatus

A photograph of the testing apparatus and a schematic drawing are both shown on Figure C1. Two standard BRE monitoring sockets were fixed into mounting plates which were welded ~2.5m apart on a steel I-beam; the beam was shielded from any direct heat/ambient sunlight as testing was performed in a cool basement laboratory. The extended posts used during site monitoring (see Chapter 4) were screwed into these sockets and the verticality of both posts were roughly verified. The micrometer bar was then placed on the posts and a large polystyrene insulating box (~0.8m x 2.0m x 3.0m) was constructed around its length. Care was taken to minimise heat transmission from inside the box to the underlying I-beam by incorporating both particle board and extra layers of polystyrene along the box bottom, and by ensuring that any gaps between adjacent polystyrene sheets were sealed with tape. The micrometer ratchet at the bar end was left protruding from the end of the box to facilitate readings. A thermostat-controlled heater and an oscillating fan were placed inside the box near the top as shown on Figure C1. Air baffles were positioned to prevent direct heating of the micrometer bar.

Four computer-logged temperature transducers were used within the insulating box: two transducers were affixed directly on the bar's surface and insulated from direct air flow with polystyrene covers, and two were positioned immediately above the bar (approximately 2cm above) to measure the air temperature within the box above the bar. Two additional transducers were affixed to the surface of the I-beam outside the box and sealed beneath polystyrene covers.

Testing procedures

The temperature in the insulating box was varied by adjusting the heater setting, the fan strength, and the positions of the baffles; the temperature range tested was between 15 and 35°C. During the tests the micrometer bar was allowed to equilibrate with the air inside the box until stable and similar temperatures were recorded for two bar-mounted temperature transducers. This was usually accompanied by temperature stability of the transducer pair positioned above the bar. The bar was considered to have equilibrated when both bar-mounted transducers were registering temperatures within ±1°C, and when the temperature change for both transducers over the previous 60 minutes was less than 0.5°C.

Three micrometer readings were then taken, removing and relocating the slotted end on the ball seat between measurements to replicate field techniques (see Chapter 4).
temperature of the beam outside of the insulated box was also monitored.

A typical result is shown in Figure C2 where the responses of the six temperature transducers are shown with time. Changes in the baffle set-up or the heater settings were made during the testing to vary the air circulation or temperature within the box; the response of the transducers within the box usually reflect these changes. The transducers located above the bar were significantly more sensitive to the heat bursts emanating from the space heater and hence show more variable responses. The transducers on the I-beam are stable during this testing period to within +0.5°C. The points at which measurements were taken are also indicated on the plot.

Total station surveying was employed at the end of testing to determine the distance between the two sockets at ambient lab temperatures for strain calculations. Eight measurements were performed, four each from two different station positions.

**Test results**

Figure C3 shows the calibration data collected during testing. The upper two plots show (a) the micrometer bar measurements versus the measured temperature of the micrometer bar and (b) the average temperature of the I-beam for each measurement point against temperature of the micrometer bar. The temperature of the beam varied by ±1.5°C during the 10 day testing period.

These variations were addressed by assuming a thermal expansivity of the steel I-beam to be half that of the aluminum micrometer bar (published values of about $12 \times 10^{-6}/°C$ for standard steel and $24 \times 10^{-6}/°C$ for aluminum, Hannah and Hillier, 1988). The expansion of the I-beam associated with a +1°C change in temperature is equivalent to an apparent contraction of the micrometer bar which would occur for a -0.5°C change in temperature at the bar. The bottom plot (c) shows the micrometer measurements against the adjusted temperature of the bar; improved linearity is seen over the range tested.

The total station measurements performed at close range are summarised in Table C1. The distance between the two sockets was determined to be 2.461 m. A linear coefficient of thermal expansion for the bar (CoT) of $22.0 \times 10^{-6}/°C$ was derived from the corrected data with a regression fit of 0.996.

**Uncertainty and error of calibration measurements**

The micrometer manufacturer states that the micrometers are rigorously tested and calibrated at a typical laboratory temperature of 20°C, and that they have a general accuracy no greater than ±3μm. Temperature effects on the micrometer itself will be negligible in comparison with the variation of the 3m aluminum rod, and were therefore not considered.

The temperature transducers were calibrated prior to testing against a thermostat controlled oven with an accuracy of approximately ±1°C. The transducers themselves have an accuracy of ±0.5°C (as given by the manufacturer). These errors equate to a potential measurement error at the micrometer no greater than ±0.06mm over a 2.5m span.
The span distance measured using the total station varied by ±1.3mm. This relatively poor precision results from using two different station locations; small deviations from the vertical of the BRE sockets will influence the apparent span distance depending on the location of the instrument.
Appendix D
Borehole mapping using inclinometer torpedo and temporary inclinometer tubing

Introduction
Borehole mapping was undertaken for all subsurface instruments deeper than 10m with the exception of the spade cells. The mapping was performed for two reasons: (a) it is advantageous to know the deviation of borehole from the surface position to aid understanding asymmetry in response, and (b) mapping performed after removal of the electrolevels can provide verification of the instrument performance. In fact, the inclinometer mapping results have provided critical information on the possible mechanisms of ground response; these are discussed in Chapter 7 of this dissertation.

In this appendix the data are in two categories. The first is the verticality survey results determined from the surveys immediately after instrument installation. The second is vertical profiles horizontal movements, both $u$ (parallel) and $v$ (perpendicular) to the tunnel axes. The second borehole mapping survey was performed about 1.5 years after eastbound tunnel construction and over 2 years after the westbound tunnel was excavated.

Equipment
The instrument used for mapping was a Geotechnical Instruments MKIII shown schematically in Figure D1a. It has a 0.5m gauge length (i.e. the distance between the upper and lower sets of guide wheels) and uses a single uniaxial gravity sensing transducer to measure the instrument tilt over the gauge length.

Mapping procedure
At each inclinometer the permanent tubing was mapped. The tubing installation is discussed in Chapter 4. For the extensometer and piezometer holes, temporary inclinometer tubing lengths were joined together at the surface and fed down the completed borehole; plastic spacers positioned at either end of each length maintained the tube approximately at the centre of the hole.

The probe was lowered inside the tubing to full depth and a traverse was made by holding the probe stationary every 0.5m and digitally recording the depth and inclination reading. The survey was repeated with the probe guide wheels in the second pair of grooves, 90 degrees from the first.

When mapping the permanent tubing for the inclinometers, the probe was rotated through 180 degrees at each groove pair and the traverse repeated at identical measurement locations. This procedure allows for the determination of instrument face errors which are discussed later in this appendix.
Verticality survey results

Measured deviations from the true vertical for each measured length (i.e. the instrument gauge length) are summed over the entire tube length assuming that one end of the tube is fixed as shown schematically on Figure D1b. In this appendix, the hole top was assumed as zero. The cumulative deviations from vertical with depth are given in Figures D2-D4; the key figure on each plot shows the sign conventions used. The maximum deviation are summarised in Table D1.

Ax and Cx, bored 4m to either side of the westbound centre-line at the ground surface, both show a negative cumulative deviation trend in the plane transverse (faces C and D) to the tunnel with depth. Since negative deviation is towards the tunnel for Cx and away from the tunnel for Ax, it appears that at the westbound tunnel axis level borehole Cx could be nearly 1.2m closer to the tunnel extrados than borehole Ax.

Inclinometer Ai bored 4m from the westbound centre line at the ground surface, shows negative cumulative deviation with depth transverse to the tunnel (faces C and D). Inclinometer Ci, 4m from the tunnel centre-line on the opposite side, shows very little cumulative deviation. Thus borehole Ai is likely to be between 0.4m and 0.5m further away from the tunnel at the axis level.

Near the eastbound tunnel, the profiles for extensometer holes Gx and Ex (approximately 5m to either side of the eastbound tunnel centre line) indicate that the extensometers at tunnel level in Gx could be a further 0.5m closer to the centre line than at ground surface.

Displacement profiles after construction of both tunnels

The final displacement profiles measured in the inclinometer holes after electrolevel removal are shown in Figures D5 and D6 for both the parallel and orthogonal profiles; key diagrams on each plot show the sign conventions used. Details pertaining to these surveys are discussed in more detail in Chapter 7.

Uncertainty and sources of error

An accuracy of the ±0.1mm over the 0.5m gauge length for the gravity sensor is indicated by the manufacturer, although this clearly depends on the relationship between the mounted sensor and the instrument axis when in the guiding slots of the inclinometer tubing. This potential face error is minimised by averaging the measured deviation and the deviation measured at same level with the instrument rotated 180°. An average face error was determined from all measurements where both instrument faces were used as 0.0005m and 0.0007m for the initial borehole surveying traverse. For surveys where only a single instrument face was used, the measurements were adjusted assuming a 0.0006m face error. For the second survey (inclinometer tubing only) the face errors were between +0.0002m and -0.0001m.

Tubing twist could potentially introduce error but Dunnicliff (1988) suggest that twist surveys are to be considered if difficulties during installation occurred, or for tubing depths
greater than 60m. Care was taken to ensure the tubes were oriented correctly at the surface before letting them 'set' into position, and twisting is thus assumed to be minimal. Systematic twist of inclinometer tubing on manufacturing may be as large as 1° per 3m length of tubing and can be additive depending on assembly techniques.

Errors in measurement can result from drift of the gravity-sensing transducers calibration, zero offset or bias drift for true vertical reading, and changes in azimuth orientation which result from difference in orientation between the axis of the transducer and the wheel assembly on the probe. Temperature effects on the gravity sensing transducer are minimised by allowing equalisation periods with the instrument downhole prior to starting a measurement traverse.
Appendix E

Schedules of borehole drilling, instrumentation coordinates, orientations, gauge lengths, calibrations, and zero readings

This appendix contains summaries of the instrument locations installed at the control site in Tables E1-E5. The origin of the coordinate system is SMP no.3 located on the westbound centre-line. The $y$ axis is perpendicular to the westbound tunnel centre-line, the $x$ axis approximately parallel to the tunnel alignment, and the $z$ axis is vertical and positive upward.

Table E6 summarises the contractor’s levelling point locations in St. James's Park. The chainages to the monitoring lines and the offset of each point from the centre-line of both tunnels are tabulated. Figure 5.5 in Chapter 5 shows the location of the monitoring lines in the park and gives the depths to tunnel axis at each line.

Table E7 tabulates the drilling progress at the site and includes specific comments on instrument installation details. All tables can be found at the beginning of volume 2 of this dissertation.
Appendix F
Schedules of surveys

This appendix comprises schedules of surveys performed at St. James's Park from March 1995. The tables are found at the beginning of volume 2 of this thesis and are divided as follows:

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<th>Table</th>
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Appendix G
Subsurface horizontal displacements in the empirical framework

The subsurface measurements of vertical (w) and horizontal (v) displacement at St. James's Park are presented on Figure G1 as vectors of displacement after construction in the transverse plane for (a) westbound and (b) eastbound tunnels. For both tunnels, large components of horizontal movement are seen at the surface, where the vectors of displacement are focused at a point above the tunnel axis (see Figure 7.9). At the level of the gravel-clay boundary, the vectors are aimed below the tunnel axis, while at deeper levels the displacement vectors become more focused on the tunnel centre-line.

In this appendix a new parameter is presented which may be implemented within the current empirical framework for predicting ground displacements due to tunnelling in clay. This empirical parameter, \( \zeta \), accounts for the variations in displacement vector directions at different levels in the subsurface to predict more closely the due to excavation above tunnels in London Clay. The formulation for \( \zeta \) is derived from the assumption that the vectors are aimed at the tunnel centre-line, and that the soil response is undrained during tunnelling.

Comparisons are made between the two tunnels at St. James's Park at different depths, and recommendations are offered on the use of \( \zeta \).

Defining \( \zeta \)

The parameter \( \zeta \) is based on the assumption that the vectors of displacement are aimed at the tunnel centre-line so that

\[
\frac{v_{(0,z)}}{w_{(0,z)}} = \frac{y}{(z-z_0)} = \frac{1}{\zeta}
\]  

(G1)

where \( v_{(0,z)} \) and \( w_{(0,z)} \) are horizontal and vertical movements at the y offset from the tunnel centre-line and \( z_0 \) is the depth to the tunnel axis level. \( w_{(0,z)} \) can be calculated using the empirical relations presented in Chapter 2. In this form, the larger the value of \( \zeta \), the smaller the horizontal displacement component of movement is calculated; vectors are aimed at the tunnel centre for \( \zeta = 1 \).

In Chapter 2, two equations for the variation in transverse trough width parameter, \( i_y \), with depth were described which fit the available data on subsurface settlement profiles in clays. Mair, Taylor and Bracegirdle (1993) presented the following relationship

\[
\frac{i_y}{z_0} = 0.175 + 0.325(1 - \frac{z}{z_0}) \quad \text{or} \quad i_y = i_0 - 0.325z
\]  

(G2)

where \( i_0 \) is the trough width parameter at the ground surface (and is equal to 0.5\( z_0 \), after Rankin, 1988). To satisfy the undrained response assumption (i.e. zero volume change) Taylor (1995) concluded that the vectors of displacement at any subsurface horizon must be aimed at
a fixed point approximately 0.175z_0/0.325 below the tunnel axis level. However, this pattern is not supported by the measurements at St. James's Park (see Figure G1) particularly at levels near the tunnel crown.

Heath and West (1996) offered an alternative curve form which also the field data presented by Mair et al. (1993).

\[ i_x = i_0 (1 - \frac{z}{z_0})^{1.2} \]  \hspace{1cm} (G3)

It can be shown that satisfying the undrained response assumption using this relation necessitates that the displacement vectors are aimed at a depth below ground level defined by the relationship

\[ z_{\text{focus}} = z + 2(z_0 - z) \]  \hspace{1cm} (G4)

This is schematically shown on Figure G2 and more closely matches the pattern of response seen at St. James's Park on Figure G1. Solving the trigonometric relationship between the ratios of \( v_{y,z} \) and \( w_{y,z} \) with respect to \( y, z \) and \( z_0 \) in this figure yields the following formula:

\[ \frac{v_{y,z}}{w_{y,z}} = \frac{y}{2(z_0 - z)} \]  \hspace{1cm} (G5)

Equating this formula to Equation G1 yields a value for \( \zeta \) of 2. This implies that the displacement vectors are focused at a depth below the horizon considered equal to twice the distance from the horizon of interest (z) to the tunnel axis level (z_0).

**Field measurements at St. James's Park**

The displacement vectors above the westbound tunnel (presented in Chapter 7) at different subsurface horizons were evaluated to determine the mean focus depth for vectors at each level. The mean values of \( \zeta \) determined for each level are plotted on Figure G3 against normalised depth, z/z_0. Also shown is the \( \zeta = 2 \) line with symbols plotted at each level considered.

At shallow depths (z/z_0<0.3, z<9m in predominantly granular material) the measured values of \( \zeta \) range between 0.5 at the surface to about 4 at the gravel-clay interface. The small value of \( \zeta \) at the surface mirrors the shallow vector directions demonstrated on Figure 7.9.

In the clay, the measured \( \zeta \) values vary linearly with normalised depth; values of Figure G3 decrease from over 4 near the gravel-clay interface to about 3 near the tunnel axis. The ratios of measured to calculated \( \zeta \) also shown of Figure G3 demonstrate that the measured values are between 1.5 and 2 times the value given by Equation G5, implying that the observed horizontal displacements are smaller than would be predicted using Equation G5.

The data from eastbound tunnel construction are presented in the same manner as for the westbound tunnel on Figure G4. The response in the granular material is broadly similar to that observed for the westbound tunnel; asymmetry in \( \zeta \) is seen between the north and south
half-troughs (note: no assessment of asymmetry was made in the clay). In the clay the values of $\zeta$ are again linear with normalised depth, but for the eastbound tunnel they show a tendency to increase with depth. This difference in trend with depth between the two tunnels may be the product of the eastbound tunnel being constructed in ground disturbed by the westbound tunnel construction, or differences in the ground response mechanisms highlighted in Chapter 7. The ratios of measured to calculated $\zeta$ shown on Figure G4 are still larger than one in the clay, again implying that the observed horizontal displacements are smaller than would be predicted using Equation G5.

**Case history data for tunnels in clay**

Measurements above both tunnels at St. James's Park have demonstrated that calculating horizontal displacements in the clay using Equation G5 underpredicts the measure values of $\zeta$ and therefore overestimates the measured ratios of horizontal to vertical movements for both tunnels at St. James's Park. This is conservative from the point of horizontal displacement and strain calculations.

Figure G5 shows $\zeta$ values recorded at St. James's Park along with those estimated from: (a) displacement vectors presented by New and Bowers (1994) from the Heathrow Express trial tunnel in London Clay; and (b) centrifuge model tests of 2-dimensional tunnels in overconsolidated kaolin (Mair, 1979). Equation G5 appears to be a conservative limit for these data, except near surface level.

**Movements near the ground surface**

The calculated $\zeta$ values in the granular material near the surface St. James's Park are nearly all less than the value of 2 given for Equation G5. This result is likely to be related to the stiffness of the material near the surface. The increased horizontal displacement component at the ground surface can be compared with the straining of an idealised beam (Burland and Wroth, 1977) resting on the gravel-clay boundary as shown on Figure G6; the beam represents the thickness of granular material overlying the clay. A restraining effect is realised at the gravel-clay interface where the stiffness of the 'beam' is largest. Burland and Wroth argued that in hogging deformation this effect lowers the neutral axis within the idealised beam, but in sagging deformation that neutral axis remains at the middle.

When deflected, straining is focused at the top of the 'beam' in hogging; this would probably result in large horizontal displacements for granular material which is unable to resist tensile strains. In sagging deformation, compressive straining is anticipated at the top of the 'beam' (i.e. the ground surface); the low stiffness of the soil (related to the low confining stresses) would probably allow compressive straining, leading to larger horizontal movements.

The magnitudes of straining for the beam analogy are directly related to the relative deflection, $\Delta L$ (Burland and Wroth, 1977). At St. James's Park the relative deflections are larger above the eastbound tunnel (see Table H1), which might be responsible for the smaller values of $\zeta$ observed in the granular material. However, at the ground surface, the values above
both tunnels coincide at about 0.25.

For the model tests, a similar pattern is seen near the surface which demonstrates that larger horizontal movements at the surface could also be expected for clay soils.

Conclusions

Subsurface horizontal displacements above tunnels in London Clay appear to be reasonably bounded by the relationship

\[
\frac{\nu_{j,z}}{w_{j,z}} = \frac{y}{\zeta = (z - z_0)}, \quad \zeta = 2
\]

The relationship may be applicable to tunnels in normally consolidated clays, although this has not been evaluated. At shallow depths the relationship appears not to be valid, particularly for granular material overlying clay.
Appendix H
The form of construction and long-term transverse displacement profiles observed at St. James's Park

In Chapter 2 a thorough review of the empirical methods for prediction of transverse settlement troughs above bored tunnels was presented. The empirical framework is developed around the assumption that the profile follows an inverted bell-curve or Gaussian distribution. Both the centre-line settlement ($w_0$) and the distribution of settlements about the centre-line are defined by two parameters: the trough volume ($V_t$) or volume loss ($V_1$, equal to the trough volume but represented as a percentage of the excavated tunnel volume), and the trough width parameter ($i_j$). For a Gaussian curve $w_0$ is equal to $V_t/(\sqrt{2\pi} i_j)$.

When back-calculating the appropriate values from field data, the trough volume can be solved numerically (e.g. integration by parts). The trough width parameter can be derived in a number of ways, all of which assume the field data follow a Gaussian distribution.

Method 1
Field data of Gaussian form plotted as the natural logarithm of normalised settlement, $\ln(w/w_0)$ against offset from the centre-line squared, $y^2$, yield a straight line for which the slope is equal to $-1/(2 i_j^2)$. A limit is applied for the linear fit at $\ln(w/w_0) = -3$ (i.e. 95% of the centre-line settlement) to minimise the influence of measurement scatter at the tail ends of the trough.

Method 2
$i_j$ conveniently coincides with both the offset to the point of inflection in the settlement trough, and the offset to the point of maximum slope for a Gaussian curve form. The former is more difficult to ascertain from field data. For the latter, closely spaced monitoring points are necessary for the point of maximum slope to be adequately well defined. For this method $i_j$ is always the offset where the deformation mode changes from sagging to hogging (see Figure H3).

Method 3
The ratio of settlement at an offset, $y$, from the centre-line equal to $i_j$ is 0.61 for a Gaussian-shaped profile, irrespective of the value of $i_y$.

Method 4
A statistical curve fitting exercise performed by minimising the square of the residual errors on the fitted Gaussian curve to the field data.

Method 1 is the procedure used in this thesis for comparing the two observed ground responses at the site, and comparing these with other field data which derive the trough width parameter in the same manner (see, for example, Lake et al., 1992).

The observed troughs at St. James's Park are not exactly Gaussian, and therefore the values of $i_j$ derived from the different methods given above can be significantly different. This appendix briefly compares the trough width parameters obtained for the transverse surface
settlement profile observed after westbound tunnel construction at St. James’s Park by using the different methods above. These comparisons suggest which method of assessment is most appropriate for these data to calculate the most realistic potential building response.

Profile comparisons above westbound tunnel

Values of \( i \), determined from the observed profile for westbound tunnel construction using the above methods are summarised in Table H1; they range from 10.4 to 12.9 m. The calculations of \( i \), for Methods 1 and 2 are presented in Section 6.1.2. Gaussian trough shapes calculated from the different trough width parameters are compared in normalised form \((w/w_0)\) on Figure H1, along with the observed trough data.

- Method 1 gives a wider calculated curve shape than observed. Method 2 \((i_y = \text{point of maximum slope})\) yields the most narrow trough shape and the worst fit to the normalised field data; the calculated values are smaller than those observed at nearly all offsets. Method 3 \((i_y \text{ at } 0.61w_0)\) gives a close fit to the field data at offsets within about 15 m from the centre-line, and slightly underestimates values further out. The least squares curve fit is slightly wider for offsets from the centre-line to about 20 m.

These results are initially surprising, as Method 1 is considered a more thorough fitting procedure than Method 3 but gives a poorer fit to the data. In retrospect, Method 3 should give a better back-calculated fit to data between \( y = 0 \) and \( y = i_y \), as the normalised values at the centre-line and at the inflection are fixed to conform to the field settlement data \((i.e. w/w_0 = 1 \text{ at centre-line. } = 0.61 \text{ at } y = i_y)\), while the Method 1 is derived from interpretation or smoothing of the field measurements. The least squares analysis appears to give the best overall fit to the normalised trough shape.

Using the trough volume determined from the field measurements \((i.e. V_1 = 3.3\%\) and the relation between \( w_0, i_y, \) and \( V_1 \) given earlier, the centre-line displacements are calculated for each method and are summarised in Table H1. The values range from -19 to -23.5 mm, a variation of less than \( \pm 3\% \) from the observed value.

Calculated displacement profiles for the four different methods of determining \( i_y \) are given on Figure H2. Method 1 underestimates the measured movements while Methods 2 and 3 overpredict. Method 4 \( (\text{least squares analysis})\) yields the trough which agrees most favourably across the transverse profile.

Further comparisons are made between the different methods in Table H1 in terms of maximum slope and maximum deflection ratio \((\Delta/I)\) for the hogging portion of the trough, these are schematically shown on Figure H3. Maximum slope is a convenient preliminary assessment of risk of building damage due to tunnelling. Burland and Wroth (1977) highlight the deflection ratio as a key parameter in relating observed deformation to potential building damage, particularly in hogging-mode deformation for masonry structures.

In Table H1 Method 1 is seen to underestimate the observed value of maximum slope by about 20\%, while Method 2 over-predicts by 22\%. Methods 3 and 4 show better agreement, both being within \( \pm 6\% \) of the observed maximum slope. It is worth noting that the positions
of maximum slope for the troughs estimated from Methods 1, 3 and 4 are at larger offsets than actually observed.

The deflection ratios in Table H1 were calculated over a 20m span length extending from the calculated trough width parameter offset. The $\Delta/L$ from the field data is representative of the largest measured deflection ratio for a 20m span in the hogging zone; in this case, the span extends from 12.5 to 32.5m offset.

Method 1 underestimates the field value by about 16%. The others overestimate the observed deflection ratio: 47% for Method 2, 17% for Method 3, and 4% for Method 4. Since $\Delta/L$ is directly related to the critical bending and shearing strains calculated in a stage 2 assessment (see Chapter 2) as shown on Figure H4, good prediction of the deflection ratio is essential for assessing building performance.

Profile comparisons above eastbound tunnel

Values of $i_d$ determined from the less-disturbed north half-trough observations above the eastbound tunnel are summarised in Table H1; these range from between 6.6 to 8.2m. Calculated profiles presented in normalised form and compared with the field data on Figure H5 show a wider trough shape for Method 1, while the other three methods are grouped very closely near the observed profile.

Using the trough volume determined from the field measurements (e.g. $V_1 = 2.2\%$ for the north half-trough) and the relation between $w_0$ and $V_s$ given above, the centre-line displacements are calculated for each method and included in Table H1. The values range from -19.1 to -23.9mm. The largest difference between calculated and observed settlement is for Method 2 (nearly 15%).

The calculated settlement troughs for the four methods are presented on Figure H6. Method 1 underpredicts the settlement values for offsets to about 8m, but shows closer agreement to the field data at larger offsets. The other three curves show close agreement across the half-trough.

The calculations for maximum slope show a similar pattern between the methods as seen for the westbound tunnel data. Method 1 underestimates the observed value by 34%, while Method 2 overpredicts by 13%. Methods 3 and 4 again show better agreement, both being within ±5% of the observed maximum slope. The positions of maximum slope for the troughs estimated from Methods 1, 3 and 4 are again at larger offsets than actually observed.

The largest $\Delta/L$ from the field data for a 20m span in the hogging zone was determined for the length from 6 to 26m offset. The differences for the different methods are as follows: Method 1 underestimates the field value by about 30%; Method 2 overpredicts by 8%; Method 3 under-predicts by 1%; and Method 4 gives a value 6% less than observed.
Long-term settlement profiles above westbound tunnel

Trough width parameters for the long-term displacement data presented in Chapter 4 were determined using Method 1 which assumes that the long-term profiles are Gaussian in form. As a check on the selection of $i_\alpha$, the transverse profiles at about 5m below ground level at the end of Period 3 (250 days after westbound construction and just before eastbound tunnel), and part way through Period 5 (1 year after eastbound tunnel construction) are reviewed in a similar manner as the construction profiles above.

Table H2 summarises the results for the different calculation methods after each tunnel. 250 days after the westbound drive, the calculated values of $i_\alpha$ range from 13m (estimated at the point of maximum slope) and 16.7m. The normalised curve shapes for each method and the field data are presented on Figure H7. The field data are nearly Gaussian in form and the least squares best-fit provides the best representation for the normalised profile. As observed for the construction movements, Method 1 predicts a wider trough.

The calculated transverse displacement profiles for the four methods are given on Figure H8 along with the field data and the observed profile during westbound tunnel construction. The least squares fit agrees most favourably with the field data. It is worth remembering that $w_0$ is calculated from the value of $V_\alpha$ and $i_\alpha$ for the Methods 1 to 3, while for the least squares fit (Method 4), $w_0$ is calculated to best-fit the data.

For the eastbound tunnel, the estimated trough width parameter ranges from 7m (the estimated point of maximum slope) to 17.9m. The large variation reflects the tendency for the profiles to be less Gaussian than the construction profiles shown earlier in this appendix. Interestingly, the trough volume (represented as a volume loss) has climbed to over 9% after 1 year.

Figure H9 shows the normalised profiles for the four different trough width parameters. All curves give a poor fit to the normalised field data as a whole, although some methods agree over short sections of the profile. The same pattern in seen for the displacement profiles; note that Method 2 ($i_\alpha$ @ maximum slope) has been omitted as the centre-line settlement calculated (see Table H2) is nearly twice that observed and obviously gives a very poor fit to the field data.

Maximum slope and relative deflection in hogging portion of the troughs are calculated in the same way as for the construction settlements (see Figure H3). Table H2 shows that the least squares fit to long-term data after westbound tunnel construction underpredicts the maximum slope by 17%; Method 1 gives a maximum value 42% smaller than observed. In contrast, Method 2 overpredicts by 14°, and Method 3 is correct to within 2%. The same pattern is observed for the four methods when considering the deflection ratio in hogging, with the best estimate coming from Method 3. Method 1 underestimates $\Delta/L$ over 35%.

The poor fit for the Gaussian curves to the field data 1 year after the eastbound tunnel construction shown on Figures H9 and H10 give highly variable comparisons for maximum slope and deflection ratios. The best agreement is seen for Method 1 which underestimates the maximum slope by 26° but rather fortuitously gives a deflection ratio in hogging within 2%.

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of the observed value.

Conclusions

The profiles on Figures H2 and H6 demonstrate that, for most purposes, the construction profiles for both tunnels at St. James's Park are reasonably Gaussian and can be described adequately as such.

The summary data in Tables H1 and H2 illustrate that the calculated values for deflection ratio and slope are quite sensitive to the trough width parameter used, and that selection of the trough width parameter must be considered carefully.

For the construction profiles, the curve-fit derived from method 1 underestimates both the maximum observed slope and the deflection ratio in hogging, and therefore the method may not be the most appropriate for assessing the potential risk to structures. Conversely, using the point of maximum slope as \( i_y \) consistently overpredicts both maximum slope and hogging deflection ratio. The best representation for the construction profiles comes from determining both \( i_y \) and \( w_0 \) from a least squares best-fit to the data, assuming a Gaussian curve form. Maintaining the empirical relation between \( i_y \), \( V_y \) and \( w_0 \) changes the nearness of the fit slightly, but the method is by far the best for assessing the potential deformation parameters.

The long-term settlement above the westbound tunnel are usefully described as Gaussian, as shown on Figure H8; in this instance Method 3 provides the best fit to the damage parameters. A more suitable result for the least squares fit (which underestimates both the observed maximum slope and deflection ratio by about 15%) can be obtained by only fitting the data at offsets within 1 tunnel depth. In contrast, the eastbound tunnel shows very poor comparisons with the Gaussian curve form, a result which reflects the different patterns of subsurface response shown in Section 7.4.3.

As a final comment, comparisons can be made between the deformation parameters determined direct from the field measurements of long-term profiles (Table H2) and construction profiles (Table H1). The maximum slope and maximum deflection ratio in hogging observed above the westbound tunnel are both seen to decrease (i.e. reduce in the potential for damage to an overlying structure). Above the eastbound tunnel the slope is seen to increase from construction with time from the values observed during construction, while the deflection ratio in hogging decreases slightly. Thus, the assessment of potential time-dependent movements and their impact on overlying structures must considered individually for each different tunnel and for every structure.
Appendix I

Observations relating to long-term tunnel lining performance

A site visit was made on 5-June-98 to investigate the performance of the linings since completion of the running tunnels through St. James’s Park. This appendix summarises the observations during this visit for each tunnel; the comments are ordered from south to north (increasing ring no, decreasing chainage) from Westminster Station, with zero chainage about 100m beyond the Step Plate Junction at Green Park Station. In Chapter 7 these observations are put into context with the long term settlement observations.

Westbound tunnel

From Westminster Station to the instrumented site (@ chainage 880m, ring no.980) the tunnel is fairly dry. Damp patches are seen on both sides of the tunnel (averaging about every 2nd or 3rd ring) around the keys at knee level only.

From chainage 670m (~ ring no.1185) the patches become more frequent, and the damp areas on each ring appear larger, extending from the track bed/key level upwards around the ring to axis level.

A fairly sharp transition is seen at about chainage 550m to a ‘wet’ section of tunnel; between chainages 550m and 430m (~ ring nos. 1310 to 1430) where the rings are much wetter at knee level, occasionally extending up to axis level, with water ingress reaching a drainage trench which runs along the edge of the track bed.

From chainage 430 to 0m (ring nos.1430 - 1860) the pattern gradually reverts back to damp patches at around the keys at knee level, with occasional wet rings with water reaching the drainage trench.

Eastbound tunnel

From Westminster to Storey’s Gate Shaft (@ chainage 1000) the rings are damp at key level, and occasionally wet, with water leakage reaching the drainage trench; rings become wetter with progress northward. At Storey’s Gate several rings are overgrown with moss.

Chainage 830m (ring no.1120) marks the start of a transition where the rings become drier; at chainage 1085m (ring no.1155) the rings are only damp around the keys. At chainage 760m (ring no. 1210) the lining rings are almost entirely dry, with only very occasional (and small) damp patches to chainage 0m.