IMPACTS OF TUNNELS IN THE UK
A report prepared for High Speed Two (HS2) Limited:

High Speed Two (HS2) Limited has been tasked by the Department for Transport (DfT) with managing the delivery of a new national high speed rail network. It is a non-departmental public body wholly owned by the DfT.

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Executive summary

The number of people travelling by train has doubled over the last decade. Demand for inter-city journeys, commuting and freight rail transport is rising fast and will continue to do so in the future. This means that Britain’s railways are already overstretched and will get more and more overcrowded over the next 10 to 20 years. High Speed Two (HS2) will tackle this problem by building a new railway line – the first line north of London for 120 years. Phase One will run between London and Birmingham and tackle congestion and overcrowding on the West Coast Main Line. Phase Two will do the same for the East Coast and Midland Main Lines from Birmingham to Manchester and Leeds.

The proposed HS2 railway network currently includes up to 62km of twin bore tunnels, 6.3km of single bore tunnels and 9.6km of twin cell cut and cover tunnels, with associated cross passages and shafts. Installing the railway in a tunnel removes direct impacts on people and buildings of running the railway at the surface; however, there may be associated effects. Firstly, the ground above the tunnel may settle during its construction. Secondly, noise and vibration from trains during operation may sometimes be noticeable.

This report addresses the issues of settlement, or ground movement, and ground-borne noise and vibration. The report describes recent UK case histories of building rail tunnels. It reports on the impacts that these have had on people and properties, and describes measures that can be taken to avoid perceptible noise and vibration.

Tunnelling-induced ground movement

Ground movements affecting buildings occur during tunnel excavation or shortly thereafter. There is relatively little, if any, effect on people and buildings in the longer term. The excavation of the tunnel causes a shallow depression, or ‘settlement trough’, at the surface along the line of the tunnel. This report identifies the factors that affect ground movement and its key parameter of ‘volume loss’ – that is, the trough volume as a percentage of the excavated volume of the tunnel. The ground movements associated with deep open cut excavations and shafts are also presented and described based on published experience.

The impact of ground movements on buildings is assessed through an established three-stage process to determine whether there is a risk of building damage. The assessment allows buildings to be categorised on a scale of damage severity from zero to five.
Measures for mitigating settlement are set out. For the vast majority of buildings within the zone of influence of tunnel construction, mitigation beyond current practice is not required.

The report describes case studies including London Underground’s Jubilee Line Extension, Heathrow Airport tunnels, High Speed 1, Crossrail, National Grid cable tunnels, Lee Tunnel and Dublin Port Tunnel. Adopting a cautious approach during design has meant that observed settlements have generally been lower than predicted maximum values, and have typically ranged between 5mm and 25mm, depending on tunnel depth, diameter and other factors. Other tunnel case studies – namely, Ramsgate Harbour tunnel, Abbey sewer, Southwick Hill tunnel, Bristol relief sewer, and Airdrie and Coatbridge sewer tunnel – show that settlement did not create significant issues during tunnelling.

As tunnelling performance has improved over the past 20 years, ground movements have lessened. The associated volume loss has also been reduced – from upper values of between 2% and 3% in the 1990s down to generally less than 1% today for tunnels constructed in London using modern tunnel boring machines. While the vast majority of tunnelling projects are successful, with very low recorded ground settlements, occasionally an incident occurs that results in much higher localised ground settlements or subsidence – in extreme cases, this has led to ground instability. In the past 20 years, there have been three notable incidents in the UK: at Heathrow Airport during construction of the Heathrow Express; on a sewer tunnel in Hull (wastewater flow transfer tunnel); and during the construction of the Channel Tunnel Rail Link (now called High Speed 1 (HS1)) at Lavender Street in East London. Although the exact cause of such incidents can be hard to determine, the lessons learned from these past incidents are used to improve tunnelling practice. The British Tunnelling Society has summarised a number of methods for the control of ground settlement and ground stability, as well as a rigorous risk management approach in underground design and construction. HS2 will adopt the BTS guidance.

**Ground-borne noise and vibration**

This report explains the various sources of noise and vibration from tunnel construction and operation and how these can be reduced.

The impact of ground-borne noise and vibration depends on several factors, including:

- the land use and its sensitivity to noise and vibration;
- the observer’s distance from the source; and
- the way in which the soil and/or building itself transmits noise or vibration.

A steel wheel running on a rail produces some level of vibration, no matter how smooth the rail is. Some of this vibration will be transmitted through the track, the tunnel lining and then the surrounding ground – the ‘transmission path’ – and can cause vibration in buildings. Usually, the resulting vibration and noise levels are very low and generally do not cause annoyance; indeed, the average person may not even notice them.
The first rail tunnels, built over 150 years ago, often caused noise and vibration in buildings overhead. Since then, significant improvements have reduced the impact of noise and vibration. Many have been driven by the need to improve the trains’ running performance. The main improvements include:

- better quality track;
- straighter rail alignments;
- smoother running surfaces on the rails;
- fewer rail joints (reducing the dynamic loads and consequently the wear and tear on the rolling stock); and
- better suspension on the trains (which improves passenger comfort, as well as reducing the impact forces on the track).

The most substantial improvements have been made in the last 30 or so years. See Figure 0.1

For high speed trains, the need for better performance ensures that the track is maintained to a very high standard.

The process of calculating noise and vibration from rail tunnels is well understood and the effects can be accurately predicted. Where excessive noise and vibration levels are considered to be an issue, well-tried mitigation measures are available.

Rail technology has improved and modern railways produce far less noise and vibration than older systems. Recent projects, such as the Jubilee Line Extension and HS1 tunnels under London, have shown that modern railways can run in tunnels under large residential areas without noise and vibration affecting the people who live there or disturbing other highly sensitive non-residential uses.

Noise and vibration due to tunnel boring during construction have been assessed based on previous experience at Dublin Port Tunnel, HS1 and Crossrail. In general, the levels are low and occur for a limited period only.
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<th>Definition</th>
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<tr>
<td><strong>A-weighting</strong></td>
<td>A frequency weighting curve that follows the sensitivity of the human ear to sound, which is less sensitive at low and high frequencies. Denoted by suffix A or (A), as in dBA, dB(A) or LAeq.</td>
</tr>
<tr>
<td>AITES</td>
<td>Association Internationale des Tunnels et de L'Espace Souterrain</td>
</tr>
<tr>
<td>ART</td>
<td>Airside Road Tunnel</td>
</tr>
<tr>
<td>ASCE</td>
<td>American Society of Civil Engineers</td>
</tr>
<tr>
<td>BAA</td>
<td>British Airport Authority</td>
</tr>
<tr>
<td>BPM</td>
<td>Best practicable means</td>
</tr>
<tr>
<td>BTS</td>
<td>British Tunnelling Society</td>
</tr>
<tr>
<td>CIRIA</td>
<td>Construction Industry Research and Information Association</td>
</tr>
<tr>
<td><strong>Closed face</strong></td>
<td>In closed face tunnelling, the excavated tunnel face is retained by pressure applied by compressed air, slurry or excavated material. Pressure is commonly maintained using a TBM with pressurised chamber at the face, sealed against the tunnel lining. The Slurry TBM uses bentonite slurry (or other type) and the EPBM uses an excavated material to maintain closed face conditions.</td>
</tr>
<tr>
<td>CTA</td>
<td>Central terminal area</td>
</tr>
<tr>
<td>CTRL</td>
<td>Channel Tunnel Rail Link</td>
</tr>
<tr>
<td>DART</td>
<td>Dublin Area Rapid Transport</td>
</tr>
<tr>
<td>dB</td>
<td>Decibel</td>
</tr>
<tr>
<td>EIA</td>
<td>Environmental Impact Assessment – an assessment of the possible positive or negative environmental, social and economic impacts that a proposed project may have.</td>
</tr>
<tr>
<td>EPB</td>
<td>Earth pressure balance</td>
</tr>
<tr>
<td>EPBM</td>
<td>Earth pressure balance machine</td>
</tr>
<tr>
<td>ES</td>
<td>Environmental Statement – a document that sets out the likely impacts on the environment arising from the project.</td>
</tr>
<tr>
<td>FOI</td>
<td>Freedom of Information</td>
</tr>
<tr>
<td>FoS</td>
<td>Factor of Safety</td>
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<tr>
<td>FRA</td>
<td>Federal Railroad Administration</td>
</tr>
<tr>
<td>FST</td>
<td>Floating slab track</td>
</tr>
<tr>
<td><strong>Groundborne vibration</strong></td>
<td>Noise caused by vibrations born transmitted through the ground noise (this is generally only significant within a building).</td>
</tr>
<tr>
<td>hr</td>
<td>Hours</td>
</tr>
<tr>
<td>HS1</td>
<td>High Speed 1</td>
</tr>
<tr>
<td>HS2</td>
<td>High Speed 2</td>
</tr>
<tr>
<td>HSE</td>
<td>Health and Safety Executive</td>
</tr>
<tr>
<td>Hz</td>
<td>Hertz</td>
</tr>
<tr>
<td>ICE</td>
<td>Institution of Civil Engineers</td>
</tr>
<tr>
<td>ID</td>
<td>Inside (internal) diameter</td>
</tr>
<tr>
<td>IStructE</td>
<td>Institution of Structural Engineers</td>
</tr>
<tr>
<td>ITA</td>
<td>International Tunnelling and Underground Space Association</td>
</tr>
<tr>
<td>JCOP</td>
<td>Joint Code of Practice</td>
</tr>
<tr>
<td>JLE</td>
<td>Jubilee Line Extension</td>
</tr>
<tr>
<td>K</td>
<td>Trough width parameter</td>
</tr>
<tr>
<td>Kv</td>
<td>Kilovolt</td>
</tr>
<tr>
<td>km</td>
<td>Kilometre</td>
</tr>
<tr>
<td>$L_{pA_{Smax}}$</td>
<td>The maximum A-weighted sound</td>
</tr>
<tr>
<td>Acronyms</td>
<td>Definition</td>
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<td>---------------</td>
<td>---------------------------------------------------------------------------</td>
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<tr>
<td>LPT</td>
<td>London Power Tunnels</td>
</tr>
<tr>
<td>LUL</td>
<td>London Underground Limited</td>
</tr>
<tr>
<td>m</td>
<td>Metres</td>
</tr>
<tr>
<td>mbgl</td>
<td>Metres below ground level</td>
</tr>
<tr>
<td>mm</td>
<td>Millimetres</td>
</tr>
<tr>
<td>NATM</td>
<td>New Austrian Tunnelling Method</td>
</tr>
<tr>
<td>NG</td>
<td>National Grid</td>
</tr>
<tr>
<td>NTS</td>
<td>Non-technical summary</td>
</tr>
<tr>
<td>OD</td>
<td>Outside diameter</td>
</tr>
<tr>
<td>open face</td>
<td>Open face excavation is carried out with the tunnel face at atmospheric pressure. The term applies to tunnels excavated without support shields, with open shields or TBM where the chamber at the face is not pressurised</td>
</tr>
<tr>
<td>Pa</td>
<td>Pascals</td>
</tr>
<tr>
<td>PPV</td>
<td>Peak particle velocity of an oscillating vibration velocity waveform. Usually expressed in millimetres/second</td>
</tr>
<tr>
<td>PSC</td>
<td>Professional services consultant</td>
</tr>
<tr>
<td>PSF</td>
<td>Professional service framework</td>
</tr>
<tr>
<td>RESS</td>
<td>Required excavation support sheets</td>
</tr>
<tr>
<td>rpm</td>
<td>Revolutions per minute</td>
</tr>
<tr>
<td>SAM</td>
<td>Scheduled ancient monument</td>
</tr>
<tr>
<td>SCL</td>
<td>Sprayed concrete lining</td>
</tr>
<tr>
<td>settlement</td>
<td>Downward vertical movement of the ground surface</td>
</tr>
<tr>
<td>SGI</td>
<td>Spheroidal graphite iron</td>
</tr>
<tr>
<td>SPL</td>
<td>Sound pressure level</td>
</tr>
<tr>
<td>T5</td>
<td>Terminal 5</td>
</tr>
<tr>
<td>TBM</td>
<td>Tunnel boring machine</td>
</tr>
<tr>
<td>TRL</td>
<td>Transport Research Laboratory</td>
</tr>
<tr>
<td>VAMC</td>
<td>Vibration additional mitigation case</td>
</tr>
<tr>
<td>VDV</td>
<td>Vibration dose value. A cumulative measure of vibration over a defined time period. This is defined in BS6472:2008</td>
</tr>
<tr>
<td>VRC</td>
<td>Vibration reference case</td>
</tr>
<tr>
<td>XLPE</td>
<td>Cross linked polythene (cable system)</td>
</tr>
<tr>
<td>ZOI</td>
<td>Zone of Influence (zone located within 1mm greenfield settlement contour)</td>
</tr>
</tbody>
</table>
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Introduction

1.1 Background

1.1.1 In January 2012, when the Government gave the go-ahead to High Speed Two (HS2), a commitment was made to publish an assessment of the United Kingdom’s history of building tunnels beneath properties. This included a commitment to “publish a clear, thorough and fully evidenced assessment of:

- the UK’s recent history of building such tunnels;
- the actual impacts that these have had on the properties and people above them; and
- the measures that will be taken to ensure that perceptible vibration impacts can be avoided”.

1.1.2 This report fulfils this commitment made by HS2.

1.2 Methodology

1.2.1 Data used as a basis for this report have been collected from a variety of projects. Examples throughout the report have been chosen because of their similarity to the HS2 project in terms of project scale, location, ground conditions and construction methods. The following sources of data have been used:

- ground movement data provided by project teams on current tunnel construction projects, including Crossrail, National Grid London Power Tunnels, Thames Water and the Lee Tunnel;
- ground movement data published in papers and conferences; and
- ground-borne noise and vibration data published on tunnel projects.

1.2.2 Almost all of the UK tunnel projects with similar features to HS2 during the last 30 years have been implemented in London and the South East. The British Tunnelling Society (BTS) has published a database of tunnels in the United Kingdom on its website (www.britishtunnelling.org.uk). An excerpt from this database, showing tunnels constructed since 1980, with some augmentation based on the knowledge of the authors, is included in Appendix D. Examination of the database shows that no tunnels greater than 4m diameter have been constructed in the types of rock (such as Mercia Mudstone or Coal Measures) expected in the northern sections of the HS2 routes during the last 30 years. Furthermore, there is little data available about ground movement above the smaller tunnels, as movements in the harder rock were so small as to be of little concern.

1.2.3 A search of international literature was made and enquiries issued via members of the International Tunnelling Association (ITA) to identify relevant international projects for comparison with the tunnels in rock. These enquiries provided little data which could be used.

1.2.4 The ground conditions in the Midlands and North of England are different from those of the London area. Many of the ground conditions comprise stronger rock than the soft ground experienced in the London Basin (see section 3), although parts of the Mercia Mudstone are likely to be of similar stiffness to London Clay. The case studies for Dublin Port Tunnel and the North Downs Tunnel, presented in section 5 and following, cover rock types of similar stiffness to that expected in the Midlands and North of England.
1.3 **Outline**

1.3.1 In section 2, this report presents the HS2 scheme and the underground elements currently proposed.

1.3.2 In section 3, the expected ground conditions along the proposed HS2 route are summarised for the various underground structures.

1.3.3 Section 4, together with Appendix A of the report, reviews excavation-induced ground movements and their effects on third-party infrastructure. Building response to ground movement is initially discussed, including details of the process by which properties are assessed.

1.3.4 In sections 5, 6 and 7, together with Appendices B and C, the report explains the ground movement mechanisms associated with bored tunnels, cross passages, shafts and cut and cover structures.

1.3.5 Results from the UK and Ireland’s recent projects are presented, demonstrating the ability to manage and control the effects of ground movement, which limits the impacts on the people and assets located above the infrastructure. Current practice and potential mitigation measures are discussed, as well as lessons learned from previous projects.

1.3.6 In section 8, ground-borne noise and vibration from railway tunnels are described, along with the impacts that these can have. Ground-borne noise and vibration during construction and in operation are also covered. Results from past projects are presented and explained, as well as potential mitigation methods.

1.3.7 A non-technical summary (NTS) has been written in association with this technical report and is available at www.hs2.org.uk/Assessment of the Effects of Rail Tunnel Construction and Operation in the UK.
Figure 2.1 | Proposed High Speed Two (HS2) rail network
2 HS2 project

2.1 Phase One and Phase Two

2.1.1 The HS2 project provides a high speed railway network, increasing connectivity across the country. It consists of two phases. Phase One links London and the West Midlands, as well as connecting to HS1. Phase Two branches into a Y shape, with lines running on to Manchester, Leeds and beyond. The full network is shown in Figure 2.1 on the previous page.

2.1.2 Under current design, up to 62km of twin bore tunnels, 6.3km of single bore tunnels and 9.6km of twin cell cut and cover tunnels are planned along the route, with associated cross passages and shafts. The design of Phase One is further advanced than that of Phase Two; hence, greater detail about the tunnels and structures is available.

2.1.3 The bored tunnels in Phase One are at the Euston HS1-HS2 link, Old Oak Common, Northolt, Chiltern, Long Itchington Wood and Bromford tunnels. The cut-and-cover tunnels (including ‘green’ tunnels) include South Heath, Wendover, Greatworth, Chipping Warden, Long Itchington Wood (along part of its length) and Burton Green. Underground station connections are proposed at Euston. A station box is proposed at Old Oak Common and a crossover box is proposed at Victoria Road, near Old Oak Common.

2.1.4 The bored tunnels in Phase Two are: (on the Leeds route) East Midlands, Hoyland and Ardsley, and (on the Manchester route) Whitmore; Madeley; Crewe; and Manchester. The cut-and-cover tunnels (including ‘green’ tunnels) in Phase Two include: (on the Leeds route) Red Hill and Shelley; and (on the Manchester route) Hopton.
3 Ground conditions overview

3.1 Ground categories

3.1.1 Table 3.1 below lists the ground types. Some key examples for the HS2 alignment are identified, along with some typical properties associated with tunnelling. This table is provided for reference further through the document.

<table>
<thead>
<tr>
<th>Ground type</th>
<th>Examples</th>
<th>Properties (typical)</th>
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<tbody>
<tr>
<td>Post-glacial</td>
<td>Terrace Gravels</td>
<td>Located close to the surface</td>
</tr>
<tr>
<td></td>
<td>Langley Silt, Peat, Alluvium</td>
<td>Often requires ground improvement for tunnel construction</td>
</tr>
<tr>
<td>Glacial</td>
<td>Glacial tills: Lodgement tills/deformation tills (boulder clay)</td>
<td>Ground properties are extremely variable with hard and soft deposits often intermixed</td>
</tr>
<tr>
<td>Pre-glacial</td>
<td>London Clay Formation, Lambeth Group, Thanet Sand Formation, Bagshot Beds, Gault Formation</td>
<td>Ground conditions for London basin tunnelling, Granular soils are usually water bearing</td>
</tr>
<tr>
<td>Rock</td>
<td>Chalk Group, Upper Greensand Formation, Great Oolite Group, Rutland Formation, Inferior Oolite Group, Marlstone Rock Formation, Dyrrham Formation, Whitby Mudstone Formation, Charmouth Mudstone Formation, Penarth Group/Branscombe Mudstone Mass Formation/Mercia Mudstone Group, Sherwood Sandstone Group, Lenton Sandstone Formation, Edlington Formation, Kidderminster Formation, Tile Hill Mudstone, Coal Measure Sandstones, Mudstones &amp; Siltstones (including Mexborough Rock Sandstone)</td>
<td>Can vary in strength with some acting more like engineering soils, Others are harder rock and stand up for extended periods</td>
</tr>
</tbody>
</table>

3.1.2 The expected ground conditions for the Phase One tunnels/underground structures are as follows:

- Euston, HS1-HS2 link and Old Oak Common tunnels, Old Oak Common station box, Victoria Road crossover box – London Clay
- Northolt Tunnel – London Clay/Lambeth Group/Chalk
- Chiltern Tunnel – Chalk
- Long Itchington Wood and Bromford Tunnels – Penarth Group/Mercia Mudstone Group (including dolomite, Siltstone, and Sandstone bands)
- South Heath & Wendover Tunnels – Chalk
- Greatworth Tunnel – Glacial till, Great Oolite Group, Inferior Oolite Group, Whitby Mudstone Formation, Rutland Formation (Limestone/Lias Clay)
- Chipping Warden Tunnel – Lower Jurassic Marlstone Rock Formation, Dyrrham Formation, Charmouth Mudstone Formation
- Burton Green Tunnel – Upper Carboniferous Tile Hill Mudstone.
The expected ground conditions for the Phase Two tunnels are as follows:

- East Midlands and Crewe tunnels – Mercia Mudstone Group
- Whitmore tunnel – Kidderminster Formation – Sandstone, Mudstone, conglomerate
- Madeley & Manchester tunnels – Mercia Mudstone Group, Siltstone and Sandstone
- Hoyland tunnel – Coal Measures, Sandstones, Mudstone and Siltstone
- Ardsley tunnels – Coal Measures, Sandstones, Mudstone and Siltstone, including Mexborough Rock Sandstone
- Hopton Tunnel – above-ground “green” tunnel
- Red Hill – Mercia Mudstone Group and Siltstone – Branscombe Mudstone Formation
- Strelley tunnel – Glacial till, Sherwood Sandstone Group comprising Lenton Sandstone Formation over Edlington Formation.
4 Effects of ground movement on buildings and structures

4.1.1 Buildings respond to ground movement in different ways, but often this response goes unnoticed. This is because most buildings are generally good at tolerating small ground movements of the magnitude produced by modern tunnelling. The effect on a building depends on the amount of ground movement, the difference in this movement across the building, and the building structure itself.

4.1.2 In order to determine whether there is a risk of building damage from ground movement, engineers carry out assessments. A description of the methods for assessments used, with experience from recent projects, is included in Appendix A.

4.1.3 The process of assessing the impact of excavation-induced ground movements is simple and robust, resulting in conservative assessments of potential damage and the need for protective measures. The process is also widely recognised throughout the world as a suitable approach to assessing excavation-induced potential damage.
5 Tunnel construction and ground movement

5.1 Sources of ground movement due to tunnelling

5.1.1 When a tunnel is constructed, the ground moves towards the excavation face. This results in relaxation ahead and around the tunnel, which propagates towards the surface. A shallow depression or ‘trough’ results, following the tunnel alignment. The movements start some distance ahead of the tunnel face and continue for some time after excavation. The vast majority of these types of movement occur within a few days of the tunnel face passing.

5.1.2 In addition to movement of the ground, movement of groundwater occurs towards the tunnel, which causes changes in pore water pressures and potentially results in consolidation settlement – this is more of a long-term effect. The initial movements of ground as the tunnel is constructed are generally larger and are more localised than the consolidation settlements discussed in sections 5.2.1 and following.

5.1.3 Volume loss ($V_L$) is the parameter typically used to assess the performance of ground movement from tunnelling. It is defined as the volume of ground loss as a proportion of the final tunnel volume, and is measured in the plane of the tunnel (Dimmock & Mair, 2007). For the purpose of calculating $V_L$, the ground loss is taken to be the area of the settlement trough (B) measured from ground movements at the surface and expressed as a percentage of the excavated tunnel cross-section (A). In some cases dilation of the ground can result in not all of the volume loss at the tunnel face being transferred to the surface, in which case settlements are smaller.

5.1.4 This ground movement causes the trough-like profile shown in Figure 5.1. This creates a characteristic three-dimensional ground movement profile, as shown in Figure 5.2.

5.1.5 Tunnel construction can be carried out with an open or closed face. Open face tunnelling excavation is carried out with the tunnel face at atmospheric pressure. The term applies to tunnels excavated without support shields, with open shields or TBMs where the chamber at the face is not pressurised.
Closed face tunnelling is a method where the excavated tunnel face is retained by pressure applied by compressed air, slurry or excavated material. The pressure is commonly maintained using a TBM with pressurised chamber at the face, sealed against the tunnel lining. The Slurry TBM (Figure 5.3) uses bentonite slurry (or other type) and the EPBM (Figure 5.4) uses excavated material to maintain closed face conditions.

5.1.6 The International Tunnelling and Underground Space Association (ITA) AITES Report 2006 on Settlement induced by tunnelling in Soft Ground (Leca & New, 2007) identifies four areas (see Figure 5.5) contributing to ground movement due to shielded tunnelling:

- ground movements above and ahead of the face;
- ground movements over the length of the shield;
- ground movement at the tail of the shield; and
- ground movement due to lining deformation.

5.1.7 Movements above and ahead of the face in closed shield tunnelling are affected largely by control of the face pressure – the confining pressure in the chamber of the shield. This pressure can be controlled by operation of the shield (rate of screw operation on an EPBM or slurry flow rates of a tunnel boring machine).

5.1.8 Ground movements over the length of the shield occur as the ground moves to fill the annulus around the shield. It depends on the speed of advance, stiffness of the ground and the degree of overcut. The cut profile of the tunnel is controlled by TBM geometry, including the setting of the cutter teeth.

5.1.9 Ground movements at the tail of the shield develop from the gap between the ground and the outer face of the tunnel lining. This comprises the clearance gap outside the shield, the thickness of the tailskin and the gap inside the tailskin filled by the seals. Effective and timely grouting to fill the gap is critical in reducing movement of this nature.
5.1.10 Ground movements due to lining deformation are generally very small, as the lining is designed to resist high long-term and short-term loads and, compared with the ground, is relatively stiff.

5.1.11 According to Leca and New (2007), typically these losses are in proportions as follows:

- 10% to 20% at face;
- 40% to 50% along the shield; and
- 30% to 40% at the tailskin.

5.1.12 For underground excavations such as required for the shorter tunnel lengths, some shafts, underground station connections and cross-passages, sprayed concrete lining (SCL) techniques are likely to be adopted. SCL works involve excavation with an open face and using a staged excavation sequence. Figure 5.6 shows a typical simple excavation sequence that could be used for the construction of an SCL tunnel, with a sprayed shotcrete lining applied after each excavation stage.

5.1.13 The ground movement sources for SCL works are as follows:

- ground movements above and ahead of the excavation face;
- ground movements associated with the installation of the temporary ground support (SCL and associated ground support items); and
- ground movements associated with sequencing of excavation (partial or full-face excavation).

5.1.14 Excavation face stability is critical in controlling ground movements when using the SCL technique. Measures such as ground treatment, canopy support and bolting in the face can control face stability in challenging ground conditions.

5.1.15 Installation of the temporary ground support and closure of the ring as soon as possible behind the excavated face are key to minimising ground movements.

5.1.16 Careful sequencing to avoid unsupported faces will also reduce ground movements.

5.2 Ground movement due to consolidation

5.2.1 Consolidation is a long-term effect, occurring as pore water pressures in the ground change to their long-term equilibrium values. The greater the change in pore water pressures, the greater the consolidation effects in terms of ground movement.

5.2.2 Changes in pore water pressure can be associated with loading and unloading of the ground or lowering of the water table associated with tunnel excavation and drainage through a porous lining.

5.2.3 Consolidation is not a sudden effect, and can occur gradually over years and often over an extensive area (Mair & Taylor, 1997).

5.2.4 The ground conditions are a significant factor affecting the amount of consolidation ground movement. Consolidation effects in fine-grained silts, clays and peats are often significant and are greater than in coarse-grained soils.
5.2.5 In general, consolidation tends to have little effect on structures above the tunnel; provided that there are no abrupt changes in the underlying geology, the differential ground movements across structures are minimal.

5.2.6 Consolidation effects are generally small where the tunnel lining is reasonably watertight (with permeability significantly lower than the surrounding ground). Relative permeabilities between the ground and the tunnel lining are key to the observed consolidation effect (Mair, 2008).

5.2.7 In all cases, the HS2 tunnels will be designed as substantially watertight. However, linings may be relatively permeable compared with some clay, and long-term changes of pore water pressure may occur. The Old Oak Common station box is likely to be designed as a drained structure. Depending on the ground conditions, there may be some consolidation settlement in these areas.

5.3 Factors affecting ground movement

5.3.1 The key factors that affect the magnitude and profile of ground movements due to tunnelling are as follows:

Tunnel diameter and shape

5.3.2 Ground movements are proportional to the volume excavated (and square of the diameter) assuming all other parameters are equal. The larger the tunnel, the greater the expected ground movements. A circular tunnel is the ideal profile and for a given area will minimise ground movements, but sometimes the space requirements dictate that alternative shapes are used. Larger SCL tunnels can sometimes be constructed using partial drifts or bench and heading, in order to limit the size of open face.

Ground cover to surface

5.3.3 Shallower tunnels tend to have narrower settlement troughs with greater maximum settlement. Deeper tunnels have wider settlement troughs, but the maximum settlement is significantly reduced, assuming all other parameters remain the same. Differential ground movements and settlement impacts are thus likely to be lower for deeper tunnels.

Tunnel construction method

5.3.4 Construction with a tunnel boring machine (TBM) tends to result in reduced ground movements compared to an equivalent tunnel with an open face or using SCL techniques. Using an Earth Pressure Balance Machine (EPBM), slurry TBM or compressed air typically has the best performance in terms of ground movements. The continual balancing of the earth pressure at the face limits ground movements. However, this must be continually controlled to ensure the face is not over-pressurised, inducing soil heave.

5.3.5 Shield excavation with an open face typically has more ground movement than closed face TBMs, however where the face is stable and the lining is erected quickly behind the excavating head ground movements may be comparable to those for closed face TBM.

5.3.6 SCL is likely to produce greater ground movements as the face is open and not pressurised. The time taken to close the ring (the full circumference) of the lining is crucial because this introduces stiffness into the support. Careful construction sequencing is vital during excavation, in order to ensure support is provided to the ground and minimise the size of open faces at all stages. SCL excavation in stages tends to reduce overall ground movement and with special measures movements can be very small.
**Ground properties**

5.3.7 The ground properties govern the profile of settlement, as well as the construction method of the tunnel.

5.3.8 Coarse-grained materials typically have narrow but deeper settlement troughs. Fine-grained materials tend to have wider, shallower settlement troughs.

5.3.9 Some ground, such as clay, peats and silts, may be subject to consolidation settlement with changes of water table level, generally widespread and long-term. Rocks, such as Mercia Mudstone, Coal Measures (on the Phase Two route) or Chalk, are stiffer and tunnelling in these materials generally causes little ground movement at the surface risks. There is an inherent risk of greater settlement where tunnels pass through interfaces at faults, between types of ground or where there are voids; this can be managed and dealt with through construction methodology based on information obtained through geotechnical investigation.

5.4 **Assessment of ground movements**

5.4.1 The empirical method commonly used for predicting ground movements assumes that the profile of the settlement trough is that of a Gaussian distribution – see O’Reilly and New (1982). The approach does not take into account stiffness of any structures and is based on monitoring recorded from generally greenfield sites. It is almost universally used for first-stage assessment of ground movement associated with tunnels.

5.4.2 Figure 5.7 shows the standard Gaussian profile, along with the associated equations. This settlement profile is a conservative estimate of settlement values for an urban environment, as buildings or sub-surface structures are normally stiffer than the ground. The soil structure interaction tends to reduce maximum settlement values, but can extend the width of the trough.

\[
\begin{align*}
S &= S_{\text{max}} e^{-\frac{y^2}{2i^2}} \\
Vs &= \sqrt{2\pi} i S_{\text{max}} \\
i &= K z
\end{align*}
\]

Figure 5.7 | Settlement profile as proposed by O’Reilly and New, 1982 – soft ground tunnelling

---

*S = vertical ground movement where a positive value is settlement (in metres (m))
*S_{\text{max}} = maximum vertical ground movement (m)
*C = cover from ground level to crown of tunnel (m)
*D = tunnel diameter (m)
*z = tunnel depth from ground level to tunnel axis (m)
y = offset from tunnel centre line (m)
i = corresponds to the point of inflexion of the curve and is the standard deviation of the curve (m)
*K = trough width parameter, an empirical constant based on ground properties in the range of 0.25 and 0.7. For cohesive soils K is typically 0.4 to 0.5.
*V_s = volume loss per metre advance (m^3)

Figure 5.7 Settlemnt profile as proposed by O’Reilly and New, 1982 – soft ground tunnelling
5.4.3 Ground movement in rock – Mercia Mudstone, Coal Measures as found on the Phase Two route, or Chalk – is generally much less than for soft ground. The Gaussian distribution of settlement is less applicable, but may be used with low volume loss assumed.

5.5 Historical tunnel ground movements

5.5.1 In this section, current and existing projects are presented, along with ground movement data and information. Representative data have been combined (where possible for these projects) and compared to produce trends. The aim is to demonstrate the typical ground movements expected from tunnelling and confirm the statements made in section 5.3. The projects include the Jubilee Line Extension, the Channel Tunnel Rail Link (now HS1), the Heathrow Tunnels, Crossrail, London Power Tunnels, the Lee Tunnel (Thames Tideway Project) and Dublin Port Tunnel.

Jubilee Line Extension

5.5.2 Ground monitoring on the JLE (see following page for JLE project details) was used extensively, with vast amounts of published material about building movements based on this. A wide range of scenarios were monitored and analysed, including greenfield analysis and a variety of structural monitoring.

5.5.3 Data are provided in Mair, 2003. Assessment values prior to construction for volume loss on JLE were between 2% and 3%. Typical observed values of volume losses were for open face shielded tunnels:

- St James’s Park measurement site – eastbound 2.4%, westbound 3.3% (the high values of volume loss on the westbound drive are explained by a tunnel excavation method which allowed face unsupported for a long time and allowed high values of overbreak – Dimmock and Mair, 2007);
- St James’s Park north of lake – between 1.5% and 2.0%; and
- Generally for open face approximately 1.5%.

For closed face EPB machines:
- Southwark Park (Lambeth Group) – 0.3-0.7%; and
- Keetons Estate (stiff clays) – 1.2-1.7%.

The learning curve excavated materials consistency correct for transfer through the screw of the TBM. Some of the impacts on buildings are presented in Table A.5.

5.5.4 Compensation grouting was widely implemented to reduce impacts of ground movement on sensitive buildings. The process is described in section A.7.10. The effect is to reduce any ground movement at critical locations, such as building foundations.

5.5.5 The JLE tunnels potentially influenced a large number of properties and sensitive structures, notably buildings around Westminster Square and Network Rail viaducts east of Waterloo Station, but no significant impacts were experienced.

Channel Tunnel Rail Link (High Speed 1)

5.5.6 The settlement performance of the CTRL London tunnels has been covered in a number of published papers (Bowers, Woods & Mimnagh 2005, Institution of Civil Engineers (ICE) Proceedings Special Issue 2007, Mair 2008, Leca & New, 2007). Various figures for volume loss relating to design are quoted: 2% and 1% for initial assessments; 1% contractual requirement for TBM performance.
Case study: Jubilee Line Extension (JLE)

Scheme: London Underground’s Jubilee Line Extension runs from Green Park to Stratford in east London. Completed in 1999, its route includes 16km of twin bore 4.5m ID, 4.9m outside diameter (OD) running tunnels and 11 major station developments, shared between 11 construction contracts. The main tunnelling contracts were:

- **Contract 102 – Westminster and Waterloo stations and running tunnels from Green Park to Waterloo**
  The 4.45m diameter running tunnels were constructed in London Clay using a roadheader cutting boom mounted in a Howden shield, and lined with expanded concrete segmental tunnel linings. Cover 25m-35m.

- **Contract 103 – Southwark station and adjacent running tunnels**
  The 4.45 m diameter running tunnels were constructed in London Clay using a roadheader cutting boom mounted in a Dosco shield, running 25m beneath 100-year-old brick railway viaducts. The station tunnels included 7m ID platform tunnels, 9.2m diameter concourse tunnel, inclined tunnels, shafts and adits with spheroidal graphite iron (SGI) linings.

- **Contract 104 – London Bridge**
  A complex underground station with 600m of running and station tunnels in London Clay under a range of structures, some of which were very sensitive. Constructed partly in SGI lining segments and partly in SCL.

- **Contract 105 – Bermondsey station and running tunnels from London Bridge to Canada Water**
  Two Kawasaki EPBMs bored the twin 4.45m ID with cut diameter of 5.03m tunnels through challenging, varied ground conditions of the Lambeth Group (sands, clays and gravels) and underneath sensitive Railtrack structures. Compensation grouting was designed to limit settlement to 20mm maximum, which was largely achieved.

- **Contract 107 – running tunnels from Canada Water to Canary Wharf**
  Over this section the running tunnels were constructed through Thanet Sand using two closed face Herrenknecht TBMs. The tunnels were lined with 4.45m diameter bolted concrete segmental linings.

- **Contract 110 – running tunnels from Canary Wharf to Canning Town Portal**
  The running tunnels for this section were driven using Lovat TBMs capable of tunnelling in both open and closed modes, through ground consisting of London Clay between Canada Water and Canning Town, and through the Lambeth Group and into the Thanet Sand from there to Canary Wharf.
CASE STUDY: Channel Tunnel Rail Link (CTRL) – now HS1

Scheme: The Channel Tunnel Rail Link (CTRL) connects London St Pancras with stations in the South East and the Channel Tunnel. The scheme was the first high speed line in the UK and is now known as High Speed 1.

The London tunnels consist of 36km of tunnels driven by EPB TBMs. The majority of the tunnels were constructed under densely populated areas of north and east of London. The London tunnels were split up into the following contracts:

- C220 Stratford to St Pancras;
- C240 Stratford to Barrington Road; and
- C250 Dagenham to Barrington Road.

Tunnel details: The tunnels are 7.15m ID, 8.15m OD lined with fibre reinforced concrete bolted segments.

Geotechnical setting: The London tunnels were driven through mixed soft ground conditions that included:

- Lambeth Group (stiff clay, sands and gravels);
- London Clay;
- Thanet Sand;
- Thames River Terrace Gravels;
- Upper Chalk;
- Alluvium; and
- Peat.

The majority of tunnelling was carried out through the Lambeth Group and Thanet Sand, occasionally dipping into the Upper Chalk. These formations are water bearing and extensive de-watering of the project area was carried out before construction.
5.5.7 Figure 5.8 summarises observed performance for the three London Tunnels contracts. Volume loss of 0.25% to 0.75% was experienced in London Clay (typically equivalent to maximum settlements of 5mm to 15mm at surface), with values towards the lower end of this range for Thanet Sand. An average volume loss was quoted as 0.5%. There were local incidents, often at the commencement of drives, where volume loss was greater than the 1% specified value for particular reasons, which include tunnelling through areas of disturbed ground and testing and trialling in EPB mode. (The incident at Lavender Street in Stratford is discussed in a ‘case study’ box in section 5.6).

5.5.8 In total, 36km of bored tunnel was constructed in East London through challenging, varied ground, with approximately 3,000 residential properties, 67 bridges, 12km of railways and eight operational tunnels within the zone of influence. There was, in general, insignificant settlement impact. Passing under the Central Line with 4.5m clear ground and achieving a volume loss of 0.25% demonstrated that with careful operation, exceptional performance can be achieved where it is needed for sensitive infrastructure.
**CASE STUDY: Heathrow Tunnels**

**Scheme:** A number of major tunnelling infrastructure projects have been completed at Heathrow. These include:

<table>
<thead>
<tr>
<th>Tunnel</th>
<th>Diameter</th>
<th>Length</th>
<th>Ground conditions</th>
<th>Construction method (running tunnels)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Piccadilly line extension to Heathrow Terminals 1, 2, 3</td>
<td>3.81m ID</td>
<td>4.9km Twin bore</td>
<td>London Clay</td>
<td>Twin bore Shield with boom and roadheader</td>
</tr>
<tr>
<td>Piccadilly line extension to Heathrow Terminal 4</td>
<td>3.81m ID</td>
<td>6km</td>
<td>London Clay</td>
<td>Shield with roadheader</td>
</tr>
<tr>
<td>Heathrow Express</td>
<td>5.7m ID</td>
<td>5.4km Twin bore</td>
<td>London Clay</td>
<td>Open face Dosco shield with roadheader boom</td>
</tr>
<tr>
<td>Heathrow Airside Road Tunnel (ART)</td>
<td>8.1m ID</td>
<td>1.26km Twin bore</td>
<td>London Clay</td>
<td>EPB TBM with compressed air applied to face</td>
</tr>
<tr>
<td>Terminal 5 (T5) Piccadilly Line Extension</td>
<td>4.5m ID</td>
<td>1.6km Twin bore</td>
<td>London Clay</td>
<td>Open face TBM</td>
</tr>
<tr>
<td>T5 Heathrow Express Extension</td>
<td>5.7m ID</td>
<td>1.4km and 1.8km</td>
<td>London Clay</td>
<td>Open face TBM</td>
</tr>
<tr>
<td>Post-T5 Baggage Transfer System</td>
<td>5.1m ID</td>
<td>1.9km</td>
<td>London Clay</td>
<td>Open face TBM</td>
</tr>
</tbody>
</table>

**Tunnel details:** The majority of the works at Heathrow, with exception of the Airside Road Tunnel, were carried out either with open face TBM tunnelling or open face shielded tunnelling. Lining was segmental lining or SCL.

**Geotechnical setting:** Heathrow’s tunnelling works have mainly been constructed within London Clay. The geology of the area consists of up to 2m of made ground, 4m to 6m of Terrace Gravels and approximately 60m depth of London Clay.
CASE STUDY: Crossrail

Scheme: The Crossrail route will link existing Network Rail services from Maidenhead and Heathrow in the west, and Shenfield and Abbey Wood in the east, with new underground stations at Paddington, Bond Street, Tottenham Court Road, Farringdon, Liverpool Street, Whitechapel, Canary Wharf and Woolwich.

21km of new twin-bore tunnels are being constructed to deliver the new rail tunnels through which the Crossrail trains will operate.

Tunnel details: Eight tunnel boring machines are being used to construct the new tunnels, each of which weigh approximately 1,000 tonnes. They are up to 140m in length, with an external diameter of 7.1m. This allows for an inside tunnel diameter of 6.2m once the bolted concrete tunnel segments are in place.

The five lengths of running tunnels to be constructed are:

- **Royal Oak Portal to Farringdon** – 6.4 km
- **Limmo Shaft to Farringdon Station** – 8.3 km
- **Pudding Mill Lane Portal to Stepney Green Shafts** – 2.7 km
- **Limmo Shaft to Victoria Dock Portal** – 0.9 km
- **Plumstead Portal to North Woolwich Portal** – 2.6 km

At the front of the TBM is a full-face cutter head which rotates at around 1 to 3 revolutions per minute (rpm). As the TBM advances, the cutter head excavates the ground. The loosened material is removed from the cutter head via a screw conveyor or a slurry circuit, which moves the material through the back of the TBM and out of the tunnel via a conveyor belt.

Geotechnical setting: There will be two different types of TBM to reflect the differing ground conditions along the Crossrail route. Six will be EPBMs, which will be used for the main running tunnels between Royal Oak Portal and Pudding Mill Lane Portal. These will pass through ground which is predominantly London Clay, sand and gravels. The Thames Tunnel (Plumstead Portal to North Woolwich Portal), which is predominantly constructed through chalk, will use two Slurry TBMs.
The Airside Road Tunnel is an 8m diameter tunnel constructed in 2002 with a minimum cover of 5m. Part of the alignment runs under stands and taxiways. The paper by Sam, Rock & Audureau (2003) describes aspects of the project's construction. The EPBM, with a capability of maintaining pressure on the face using compressed air, was specified with the aim of minimising settlement. The specified maximum settlement was 15mm, equivalent to a volume loss of 0.4%. The observed ground movements indicated an average volume loss of 0.2% and a maximum of 0.35%. This project indicates that when stringent measures are taken, exceptional settlement performance can be achieved for sensitive infrastructure.

Crossrail

At the time of writing, the Crossrail tunnel drives from Royal Oak Portal to Farringdon, Limmo Shaft to Farringdon and Plumstead Portal to North Woolwich Portal were underway, with five of the eight final TBMs tunnelling beneath London. Ground movement information for monitoring points is provided from Royal Oak Portal up to Hyde Park, where the TBMs had reached at the time of writing. The first TBM constructing the westbound tunnel was approximately 500m ahead of the second TBM constructing the eastbound tunnel. No data have yet been published, but monitoring data were kindly provided by Crossrail Ltd for inclusion in this report.

Crossrail Ltd is undertaking a vast amount of monitoring across London. All tunnelling drives are being monitored thoroughly as the works progress. Listed buildings and those assessed to be in need of mitigation works are also included in the monitoring.

Figure 5.9 shows the records of ground movements along the tunnel centreline. Volume losses are generally less than the 1.0% value used to predict settlements, and range between 0.4 and 0.9%. Notably, the second bore to be driven causes a greater settlement than the first. The two are, in general, approximately 15m apart on centreline and the greater settlement on the second drive may be partly due to relaxation of the ground around the first one, which permits settlement when the second TBM passes.

National Grid London Power Tunnels

At the time of writing, the National Grid London Power Tunnels (LPT) were still under construction. No data had been published, but monitoring data were kindly provided by National Grid for inclusion in this report.

Figure 5.9 | Ground movement (settlement) against tunnel ring number for the data currently gained on the Crossrail tunnel drives from Royal Oak Portal (to 22 February 2013)
CASE STUDY: National Grid – London Power Tunnels

**Scheme:** The London Power Tunnels comprise cable tunnels, with other ancillary works, between the electricity substations at Wimbledon and Kensal Green and the electricity substations at Hackney, St John’s Wood and Willesden. The tunnels are designed to house two 400 kilovolt (kV) cross-linked polyethylene (XPLE) circuits and up to three 132kV circuits between Hillmarton Road Junction chambers and St Pancras Substation shaft.

**Tunnel details:** The project involves the construction of 3m and 4m ID diameter soft ground tunnels, using pressurised face tunnelling equipment (in open and closed modes) and bolted and gasketed segmental linings.

- **Hackney to St John’s Wood** – 12.4km (4m internal diameter)
- **Willesden to St John’s Wood** – 7.1km (3m internal diameter)
- **Wimbledon to Kensal Green** – 12.4km (3m internal diameter)

**Geotechnical setting:**
- **Hackney to St John’s Wood** – London Clay/Lambeth Group/Thanet Sand
- **Willesden to St John’s Wood** – London Clay
- **Wimbledon to Kensal Green** – London Clay and potentially Lambeth Group
5.5.15 Figure 5.10 (a) and (b) shows two arrays located approximately perpendicular to the direction of the tunnel at chainages 980 and 4210 on the Eade Road to St John's Wood drive. These display results similar to the characteristic settlement troughs as shown in Figure 5.1 and Figure 5.2. Low settlements were seen and measured; therefore, measurement error can have a larger impact on the results and may be the cause of the more angular settlement trough in Figure 5.9 (b).

\[\text{Figure 5.10} \quad (a) \text{ Ground movement trough at Chainage 980 of the Eade Road to St John's Wood drive, 4m ID, London Clay & Lambeth Group} \\
(b) \text{ Ground movement trough at Chainage 4210 of the Eade Road to St John's Wood drive, 4m ID, London Clay}\]

5.5.16 National Grid used 1.5% volume loss to assess ground movements. This equates to 4mm maximum settlement in Figure 5.7 (a) and 5mm maximum settlement in Figure 5.9 (b) based on depths of approximately 53m and 46m and a K value of 0.5.

**Lee Tunnel (Thames Tideway Project)**

5.5.17 The Lee Tunnel was under construction at the time of writing. Data for the first section of tunnel between the Beckton overflow shaft and the Beckton connection shaft (c. 800m) were kindly provided by Thames Water and the Lee Tunnel project team.

5.5.18 Figure 5.11 shows the settlement with advance of the Slurry TBM. Settlements are less than 5mm. These low values are expected because the tunnel is being excavated in Chalk, a weak rock, and volume loss will be lower than in soft ground such as clay or sands; and because of the great depth (approximately 70m) which results in a relatively wide and shallow settlement trough.

5.5.19 The maximum ground movement of approximately 4mm, as shown in Figure 5.11, is equivalent to 0.5% volume loss based on a depth of 70m, $K = 0.4$ and tunnel diameter of 8.8m. For the majority of the drive to date, volume losses are less than 0.25% based on the movements measured.

\[\text{Figure 5.11} \quad \text{Settlement for chainage 0 to 800m of the Lee Tunnel}\]
CASE STUDY:
Lee Tunnel (Thames Tideway Project)

Scheme: The Lee Tunnel is being built as part of the new Thames Tunnel project. It comprises deep shafts, a large pumping station and a 7km long, 7.2m ID deep tunnel between Beckton Sewage treatment works and Abbey Mills Pumping station. The Lee Tunnel will prevent more than 16 million tonnes of sewage per year from overflowing into the River Lee, the main tributary of the River Thames.

Tunnel details: The tunnel is 7.2m ID and 8.8m excavated diameter. The primary lining for the main tunnel consists of 350mm thick, fibre-reinforced precast concrete segments. The secondary lining consists of 300mm thick, fibre-reinforced in-situ concrete cast within travelling shutters. Double linings have been adopted to ensure watertightness. The tunnel system is very deep and will experience high differential fluid pressures, both due to groundwater pressures when empty and when surcharged by the sewage fluid (Mitchell & Jewell, 2011).

The construction is taking place with a Slurry TBM to balance the high water pressures expected along the drive.

Geotechnical setting: The tunnel is constructed in Chalk, at depths of approximately 60-70m.

Dublin Port Tunnel and North Downs Tunnel

5.5.20 Surface settlement monitoring and prediction were a critical part of this project because of the large number of residential properties under which the tunnel passes. The project was written up by Goto et al. (2004) in a presentation to the Chinese Taipei Tunnelling Association.

5.5.21 The open faced TBM used in the Boulder Clay produced centreline ground movements of between 35mm and 40mm. (The Dublin Port Tunnel is large diameter and relatively shallow compared with many others discussed in this report.) This was equivalent to a volume loss of between 0.5% and 0.7%. The predicted volume loss within the stiff Boulder Clay was 0.8%.

5.5.22 Surface settlements and volume losses are typically more limited in rocks due their greater strength and stiffness compared with soils, and this proved to be the case. Typical surface ground movements measured for the tunnel in Carboniferous Limestone were around 5mm. These were equivalent to a volume loss in the range of 0.07% to 0.12%.
Extensive in-tunnel and out-of-tunnel settlement monitoring was carried out and tunnel volume losses typically varied between 0.04% and approximately 0.5%. Maximum settlements of between 1mm and 45mm were measured along the tunnel drives. A distinction was observed between settlement with shallow cover (less than between 1.7 and 2.2 times diameter) and deep cover. The highest face losses were observed in areas with low overburden, weak Chalk and where advance was slow.

The paper by Watson et al. (2001) discusses settlement above the tunnel. In shallow cover areas, face loss was in the range 0.18%–0.5%. In deep cover areas, there was very little impact at ground level, with no more than 3mm settlement and apparent face loss of less than 0.05%. They concluded that in areas of deep cover, the ground above the tunnel forms a stable arch which takes the load.

The case studies for Dublin Port Tunnel and the North Downs Tunnel demonstrate that, for tunnels in rocks of similar stiffness to those expected for the Phase Two tunnels in Northern England, ground movement impacts at the surface are likely to be insignificant.

### CASE STUDY: Dublin Port Tunnel

#### Scheme: Dublin Port tunnel is a twin tube road tunnel that diverts traffic away from the busy Dublin city centre and is part of the M50 motorway. It was officially opened in December 2006.

#### Tunnel details: The tunnel comprises twin 4.5km-long tubes containing two lanes of carriageway and a walkway for emergency exit and maintenance; 2.6km of each tunnel is bored, with the remainder in cut and cover. The bored tunnel was constructed using two different 11.8m Herrenknecht TBMs; a Hard Rock machine for 2.25km of each tunnel in limestone and an open faced shield for 300m in Boulder Clay. Both of these TBMs had been used before and were fully refurbished and modified prior to the start of tunnelling.

Both tunnels are lined with a primary lining comprising pre-cast concrete segments and an in situ concrete secondary lining. The segmental lining consists of six segments and key; each are 1.7m wide and 350mm thick. It is designed to support full ground and water pressures. The finished internal diameter (ID) is 10.29m and cover varies between approximately 7m and 35m.

#### Ground conditions: The bored tunnel falls within ground described as Boulder Clay in the north and Carboniferous Limestone over the southern section of the route. The Limestone is typically categorised as strong to very strong.
CASE STUDY: Channel Tunnel Rail Link (CTRL) – North Downs Tunnels

Scheme: The Channel Tunnel Rail Link (CTRL) connects London St Pancras with the Channel Tunnel and the Continent. The scheme is the first high speed line in the UK and is now known as HS1.

The North Downs tunnel takes the twin tracks of HS1 beneath the Chalk hills of the North Downs in Kent. It is 3.2km long and up to 100m below the ground. This was the first major NATM tunnel constructed since the incident at Heathrow.

Tunnel details: The internal dimensions are 13.06m by 10.5m, giving an excavated cross-sectional area of 166m². The primary lining generally consists of 250mm shotcrete lining, a waterproof membrane and a secondary lining of 350mm reinforced concrete.

Geotechnical setting: The North Downs tunnel is constructed in the Lower Lewes Chalk, the New Pit Middle Chalk and the Holywell Middle Chalk.
5.6 **Improvements in tunnelling practice**

5.6.1 Tunnelling is essentially a safe construction activity with most work carried out under factory conditions. However, there have been incidents in the past connected with underground construction. This section describes some of the important lessons learned from incidents, together with the additional measures that have been implemented to make tunnel construction safer and reduce risks. The resulting developments in TBM and other equipment technology and improvements in management of schemes have reduced the impacts on the public and third parties.

5.6.2 This section discusses some of the issues relating to tunnel construction, with case studies where appropriate. It also describes the resulting developments which have improved tunnelling practice.

**Risk management**

5.6.3 A significant development in recent years has been the publishing of the *Joint Code of Practice for Risk Management of Tunnelling Works in the UK* (JCOP) by the Association of British Insurers and British Tunnelling Society in 2003. This document was prepared in response to assessing and managing insurers’ risk in tunnelling projects following a number of high-profile losses, notably at Heathrow and at Hull, as well as in other locations worldwide. An international edition of this document was also published by the International Tunnelling Association in 2006 and updated in 2012.

5.6.4 The JCOP seeks to “promote and secure best practice for the minimisation and management of risks associated with the design and construction of tunnels, caverns, shafts and associated underground structures”. It sets out requirements for managing underground construction projects which would provide assurance to potential insurers. The intention was that projects which comply with the JCOP standard for managing risk would be able to find insurance at a reasonable cost.

5.6.5 The principal requirements in the JCOP include:

- risk assessment which is undertaken and documented in risk registers at every stage of the project, from inception through to construction;
- competence of the client, designer and contractor;
- comprehensive ground investigation; and
- use of ground reference conditions or a geotechnical baseline report in contract documents to ensure fair allocation of risk.

5.6.6 The effective management of risk and better understanding of the tunnelling process has led to improvements in practice which are discussed in the following sections.

5.6.7 There is anecdotal evidence, as cited by Mellors (2011), to indicate a reduction in insured losses on tunnelling projects since the introduction of the JCOP. In the 1990s, insurance of tunnelling works was running at a loss ratio typically in excess of 500%. This was unsustainable. The fact that tunnelling projects now have constructive dialogue with the insurance industry and can be insured implies that improvements have been made and sustained within the industry. For more on risk management in context, please see “Heathrow Express Rail Link tunnel incident – 1994” on page 28.
5.6.8 The International Tunnelling Insurance Group (ITIG) published a subsurface risk report in the *Tunnelling Journal* (April/May 2013), in which leading insurers presented an assessment of the world’s current market conditions. On Europe, the report notes: “Central London remains an active area for tunnelling where robust risk management procedures are being applied.”

**Continuous monitoring**

5.6.9 Monitoring of tunnelling works is standard practice. However, the interpretation and assessment of the monitoring of data require thorough understanding. Sometimes the quantity of data makes it difficult to interpret. The setting of realistic and appropriate trigger levels, with associated actions, has greatly improved monitoring processes in recent years. Daily meetings of suitably qualified staff to review monitoring data and agree approaches to excavation and lining for the next day have been introduced as standard on UK tunnelling projects – this includes issue of ‘permits to dig’ (required excavation support sheets – RESS) for underground construction works. These meetings ensure that account is taken of trends in observations and that appropriate action follows.

## CASE STUDY: Hull Humbercare sewage tunnel collapse – 1999

**Scheme:** The 10.5km flow transfer tunnel was part of Yorkshire Water’s sewage collection and processing scheme. The tunnel is segmentally lined and 3.6m ID, consisting of six trapezoidal concrete segments, 250mm thick. Rings were assembled with the use of temporary build bolts, which were removed once they were clear of the TBM train. Ground conditions were comprised mainly of normally consolidated glacial clays in the west of the project and medium dense glacial sands and gravels to the east, “adjacent to the River Hull, a localised band of alluvial sands and clays entered the tunnel horizon, and these were instrumental in a subsequent collapse of the tunnel”. (Machon & Stevens, 2004).

**Collapse:** The collapse occurred in a section built 1 to 2 weeks previously on 16 November 1999. The centre of the collapse was within a few metres of a 7.5m shaft. The result was a crater which was 60m in diameter and approximately 2.5m deep. Fortunately, the area surrounding the collapse was being used as a surface car park. The nearest buildings suffered relatively little damage.

**Causes:** “It was established that the two primary factors leading to the collapse were: (i) the presence of substantial volumes of fine sand, under considerable water pressure, adjacent to the tunnel; and (ii) a leak through the lining large enough to allow sand to wash through.” (Grose & Benton, 2005).
**Unforeseen ground conditions**

5.6.10 A thorough ground investigation is vital to underground works, both in terms of detecting obstructions and defining ground conditions for tunnelling.

5.6.11 Fractures within the ground, permeable lenses and unexpected levels of ground interfaces can all cause incidents when tunnelling. Due to the nature of site investigation, the properties of the ground can never fully be known. However, the implementation of a thorough and extensive ground and site investigation, together with appropriate interpretation of the data, minimises these risks.

5.6.12 Attention should also be paid to the ground conditions at the start of construction on site (i.e. from shaft sinking and piling). These can offer an opportunity to further corroborate the pre-construction site investigation. In addition, probing ahead of the tunnel advance can be used in suitable ground conditions and tunnelling methods.

5.6.13 The Hull Humbercare sewage tunnel is one case in which ground conditions were a major reason for the collapse. This tunnel was constructed in difficult and variable ground, with layers of compressible peat above the tunnel horizon, and has shown the need for thorough site investigations, particularly in areas known to have variable geology. Following the collapse, the British Tunnelling Society (BTS) set up a Closed Face Working Party to examine this collapse and identify lessons to be learned by the industry. For more on unforeseen ground conditions in context, please see “Hull Humbercare sewage tunnel collapse – 1999” on page 26.

**Underground obstructions and voids**

5.6.14 Encountering underground obstructions can cause delays in construction and, at worst, a significant incident. The discovery of structures such as abandoned wells and utilities can have significant effects and often requires immediate actions.

5.6.15 Previous incidents of this kind have reinforced the need for comprehensive and thorough desk studies of old records and ground investigations, including exploratory measures for discovering underground obstructions, which is a key part of the lead-up to underground constructions. Emergency procedures must be in place before the start of construction and immediate assessment, but with the health and safety of workers and those potentially affected of paramount importance. For more on underground obstructions and voids in context, please see “Channel Tunnel Rail Link, Stratford – 2003” later in this section.

**BTS closed face tunnelling machines and ground stability**

5.6.16 Following collapses at Hull and CTRL, the BTS set up a Closed Face Working Party to examine similarities between this and another collapse in Portsmouth, identifying lessons to be learned by the industry. The principal recommendations published in the Closed Face Tunnelling Guidelines (2005) included more extensive site investigation using a range of methods, and use of annular grouting where close control is needed.
CASE STUDY: Heathrow Express Rail Link tunnel incident – 1994

Scheme: Heathrow Express trains use the fast tracks on the Great Western Main Line from Paddington. The 8.5km airport branch leaves the main line close to Stockley Bridge and swings southwards into twin tunnels which pass first under the M4 and then under the runways to reach the passenger terminals. The scheme included 10km of tunnels and two new stations: one serving Terminals 1 and 2 and the central terminal area of Terminal 3; the other Terminal 4.

Construction involved three different tunnelling methods:
- 5.675m diameter running tunnels were driven with open face Dosco roadheader shields and lined with expanded concrete linings;
- the 800m long portal structure was formed by cut-and-cover construction in a landfill area known to contain leachate, methane and other landfill gases; and
- 7.8m wide station platform tunnels, together with railway junctions and vent adits, were constructed using sprayed concrete for primary support and in some cases for secondary lining. Construction also involved the use of compensation grouting.

Incident: On 20-21 October 1994 a collapse occurred within Heathrow Airport’s Central Terminal Area (CTA). Construction was still going on minutes before the first collapse, with repairs to critical parts of the tunnel lining in one tunnel and another tunnel still being advanced. However, no one was injured during the collapse. Heathrow Airport saw major disruption and the Heathrow Express Rail Link project suffered a severe setback and major delays.

Causes: A 2000 report by the Health and Safety Executive concluded that the failure and collapse of the tunnel was a result of a chain of events:
- substandard construction in the initial length of the CTA concourse tunnel over a period of some three months;
- tunnel excavation used the sidewall or partial drift method and repairing a defective invert joint was one of the contributing factors to the collapse;
- grout jacking that damaged the same length of tunnel plus inadequately executed repairs to it some two months before the collapse;
- the construction of the parallel tunnel in failing ground; and
- major structural failure in the tunnels, progressive failure in the adjacent ground and further badly executed repairs during October 1994.

The potential hazards with using thin shell linings and compensation grouting were not identified by all parties. Management systems and a failure to fully interpret the instrumentation and monitoring data were also contributing factors.
5.7 Summary of tunnel-induced ground movements

5.7.1 Following a review of a range of projects undertaken in the UK as listed in the BTS database (see Appendix D), as well as international projects, data have been collected from current project teams and published material. Based on the data, case studies relevant to the HS2 project have been presented. Ground movement data from projects selected on the basis of relevance to the HS2 project tunnels are plotted on Figures 5.12 and 5.13.

- Post T5 Baggage Transfer Tunnel 5.5m OD
  - London Clay
  - Open face TBM

- NG LPT 4m ID – London Clay/Lambeth Group
  - Open & Closed Mode EPBM

- West Ham Flood alleviation 2.8m
  - Lambeth Group
  - Closed Mode EPBM

- Lee Tunnel 7.2m ID – Chalk
  - Closed Face Slurry TBM

- JLE (Old Jamaica Road, Southwark Park and Niagara Court Reference sites) 5.0m OD
  - Lambeth Group
  - Closed Mode EPBM

- Crossrail Royal Oak Portal to Farringdon up to Hyde Park
  - 6.2m ID, 7.1m OD
  - River Terrace deposits above London Clay
  - EPBM

- CTRL C220/240/250 – 7.15m ID, 8.15m excavated
  - Terrace Gravel, London Clay, Lambeth Group, Thanet Sand, Chalk
  - Open and Closed Mode EPBM

Figure 5.12 | Combined project data for settlement against depth over the tunnel centre line

Figure 5.13 | Combined project data for settlement over diameter squared against depth over tunnel centre line

Figure 5.14 | Timeline of project design volume losses and typical volume losses achieved with closed face tunnelling (EPBM) for normal running (Bowers et al., 2005) (Mair, 2003)
CASE STUDY: Channel Tunnel Rail Link Lavender Street, Stratford – 2003

Scheme: Boring of 7.15m ID twin tunnels for the high speed railway in the Lambeth Group (mixed sands/clays) with a Herrenknecht EPBM as part of Contract 240 from Stratford to Barrington Road.

Incident: A number of difficulties were experienced at the beginning of the drive, following a highly successful crossing below the LUL Central Line tunnels. Unexpected foundations were encountered at one point. Then, some 100m later, a large depression opened up in the gardens between two rows of terraced houses. The volume of the void was assessed at 450m³ from concrete and grout records. Nobody was injured and there was little damage to properties; however, the result was a stoppage in tunnelling – for two and a half months for the non-incident tunnel and three and a half months for the incident tunnel – to investigate causes. Once tunnelling recommenced, the remainder of the 4.5km drives, which passed under approximately 800 properties, progressed without further incident.

Causes: The incident the subject of intensive investigation. A major area of investigation was the possibility that there was an existing void in the ground, due to the existence of a well or other cause. The conclusion of the Health and Safety Executive was that vibration from the TBM had caused the collapse of a pre-existing void which had spread upwards to the surface. Following this, the British Tunnelling Society Closed Face Working Group prepared guidelines for risk management best practice.
to 5.14.

5.7.2 The most significant current project data comes from Crossrail, the National Grid London Power Tunnels and Thames Water’s Lee Tunnel. Other data sources include the Heathrow Tunnels, the Channel Tunnel Rail Link, the Dublin Port Tunnel and the Jubilee Line Extension. Most of these projects relate to London and particular care was taken to find others in the Midlands and the North of England. However, it should be noted that:

- nearly all tunnels constructed away from the South East in the last 30 years have been less than 4m in diameter; and
- ground movement due to tunnelling in material such as the Mercia Mudstone and Coal Measures has been so little that it has not been noted or recorded in detail. For example, the references to settlement are scarce in papers on rock tunnels, but there is one reference for ground movement due to tunnelling in Mercia Mudstone for the Abbey Sewer in Leicester (World Tunnelling, 1994; Tunnels and Tunnelling International, 1997). The tunnel was 2.44m in diameter, depth approximately 3m to 8m and the settlement is quoted as 5mm maximum. The ground movement was apparently very small, but the depth of tunnel at that location is unknown.

5.7.3 It is concluded that the selected data from the London projects in addition to Dublin (Carboniferous Limestone) and North Downs Tunnel (Chalk) represent ground movements which are larger than anything which would generally be expected in the soft/hard rock conditions of the Midlands and North of England.

5.7.4 Figure 5.12 shows a range of data from projects with settlement plotted against depth; it shows settlement above the centreline only. It can be seen that settlement generally increases with size of tunnel and that the range of settlement values is greater for shallow tunnels than deep tunnels. Settlement values for open face tunnelling are significantly higher than those for closed face tunnelling at similar depths and are higher for soft ground when compared to rock. The trends are generally in line with the discussion in section 5.3.

5.7.5 The settlements due to construction of the Heathrow Post T5 Baggage Tunnel stand out as the highest group. This tunnel was constructed with an open face at depths below ground ranging from 13.4m to 21.5m. The maximum settlement measured was 25mm – not great enough to affect the structures within its influence, nor affect operation of the airport.

5.7.6 The small settlements (less than 5mm) at the right of the plot achieved on Lee Tunnel reflect the good performance expected for closed face tunnelling in rock.

5.7.7 The Crossrail data show that for larger-diameter tunnels, settlements can be limited to a range of less than 15mm with closed face extraction.

5.7.8 In general, the maximum settlement for open face tunnelling is approximately 25mm. For closed face tunnelling it is approximately 15mm. Many properties and assets would generally not require any mitigation for ground movements of this level. However, each property would be assessed on an individual basis if ground movements of this magnitude were predicted.
5.7.9 The plot shown as Figure 5.13 shows ground movement/diameter$^2$ on centreline plotted against depth of tunnel, and was made so that tunnelling performance can be compared more easily between tunnels of different diameter. The lines shown represent ground movement for a Gaussian profile of greenfield settlement (see section 5.4.2) with different values of volume loss.

5.7.10 The comparison of the plotted points with the curves for the Gaussian settlement profiles validates the empirical assumptions made in settlement assessment:

- the collection of points for each project tend to lie along a line representing a value of volume loss;
- the volume loss for all projects is generally less than 1.5%; and
- the volume loss for closed face tunnel excavation is generally less than 1% and mostly around 0.5% – almost always less than open face tunnelling.

5.7.11 The data from National Grid London Power tunnels represent a mixture of open and closed face tunnelling, dependent on the varying ground conditions. In some cases, the points fall below the 1.5% curve for volume loss, but maximum settlements are less than 10mm. These 4m diameter tunnels are relatively small compared with rail tunnels and there may be scale effects, particularly for open face tunnelling, for which volume losses of between 0.5% and 2.0%.

5.7.12 Figure 5.14 shows a decreasing trend in project design volume loss values for EPBM tunnelling. There have been developments in understanding how the measures available can be employed effectively to improve settlement performance in tunnel construction. Better management of risk in tunnelling has produced demonstrable results.

5.7.13 From the survey of ground movement data, it can be concluded that ground movements have been:

- too small to affect most structures;
- predictable; and
- steadily reducing in magnitude over the years.
5.8 Current practice for minimising ground movements due to tunnelling

5.8.1 The following are brief summaries of measures regularly used to ensure that ground movements from tunnelling remain within an acceptable range. Some of these have been developed from past incidents and are now current practice. Details of measures to reduce the impact of tunnelling are given in section A.7.

Control of face pressure

5.8.2 Careful control of face pressure in the TBM chamber to balance the combined earth and water pressures at the face is one of the most important ways of minimising ground movements. The balance of pressure is achieved by ensuring that the removal of material from the chamber is equivalent to the TBM advance. Automated monitoring, weighing of excavated material and 24-hour working discussed below all contribute to effective control of face pressure.

Tailskin grouting

5.8.3 Tailskin grouting is performed to reduce the surface settlements behind the TBM and is an inherent part of the TBM process. This consists of grouting the void on the outside of the segments in a segmentally lined tunnel. Continuous grouting generally takes place as the TBM advances on its drive.

5.8.4 Grouting can be performed either by injection through the segments or through the trailing shield. Grout and injection pressures must be continually monitored to ensure that the void is fully filled, but that the tunnel lining is not over-pressurised.

5.8.5 This mitigation method reduces volume loss and ground movements associated with the annular space created behind the shield. The grouting may be linked to TBM advance by an interlock to ensure that the void is adequately filled.

Figure 5.15 | Methods for tailskin grouting; (a) Grouting through the trailing shield (b) Grouting through the segments (Images – Control of Ground Settlement in EPB Tunnelling – Denys Slinchenko – LOVAT Inc)
Annular grouting

5.8.6 Grouting of the annulus surrounding the TBM shield has been found to be effective in reducing settlement (volume loss). The void around the shield is filled with slurry injected at the head and prevents closure of the gap between shield and the excavated face. This was employed at places on High Speed 1 where settlement performance was critical. Annular grouting increases friction resisting head rotation and slows progress, but for places where it is important to minimise settlement (volume loss) it is a valuable tool.

Automated monitoring

5.8.7 Automated monitoring of the tunnels and relevant areas of the ground above them is now current practice in tunnelling, particularly in urban areas. This allows the pressures at the face and advance rate to be adjusted to optimise performance including ground movements.

5.8.8 The use of appropriate trigger levels with associated actions ensure rigour in the review of monitoring data and ensures that any issues are identified as underground construction works progress.

5.8.9 The British Tunnelling Society’s Monitoring Underground Construction – a best practice guide offers guidance for the industry. This document is appropriate to all underground construction, including new tunnelling works, modification and repair of existing tunnels and other deep excavations processes, such as shafts and deep box excavations.

Weighing of excavated material

5.8.10 Weighing and measuring the volume of excavated material is one way to ensure the face is not being over-excavated. Over-excavation can lead to increased ground surface movements (volume loss). In EPB tunnelling, the same volume of material should be extracted from the face as is being excavated. This ensures that the volume supporting the face is maintained.

5.8.11 Simple methods of monitoring the volume and weight being removed from the face include counting the number of cars used to carry away the excavated material compared to the advance of the tunnel. More modern technologies such as mechanical belt scales, laser volume scanners and nuclear density meters can be used. This significantly increases the accuracies of the excavated material measurement.

Continuous working

5.8.12 Continuous working is now regularly used in tunnelling with machines operating 24/7, including Crossrail and National Grid London Power Tunnels. This is particularly relevant to work in closed face tunnelling, but also to SCL and open face tunnelling.

5.8.13 By continually tunnelling this means the ground is not allowed to fully relax at and around the machine face. It also means that in closed face tunnelling, a constant pressure can be applied to the face to offer stability and support for the ground. Both of these aspects tend to reduce ground movements (volume loss).
Comprehensive ground investigation

5.8.14 Comprehensive ground investigations are key to the success of current and future underground construction work. Developments are continually being made in ground investigation practice to improve the knowledge of the ground and groundwater conditions before construction. They identify ground properties to assist in the design and assessment and reduce risk by investigating uncertainties.

TBM specification

5.8.15 Appropriate TBM specification along with comprehensive ground investigations can be used to limit ground movements (volume loss) and increase productivity. For large scale projects, such as HS2, TBMs would be specified and commissioned specifically for this project. Individual ground conditions would be taken into consideration during the specification stage hence producing a machine most capable and suitable for the HS2 project.
6 Shaft construction and ground movement

6.1 Introduction

6.1.1 For the purpose of this report, a shaft is either a circular excavation or a rectangular excavation with plan dimensions comparable to the depth. An excavation with a length significantly greater than the width is covered under open cut sections. In principle, ground movement due to shaft excavation is three-dimensional, whereas open cut excavations are two-dimensional. Movement around circular shafts in the horizontal plane is less because radial movement is resisted by the ground and the relatively stiff walls.

6.1.2 Ground movements for shafts can often be limited to the construction site in which the shaft is constructed and the land that has been acquired to build it. Thus other parties are often not affected. As a result of this, few accounts detailing ground movements associated with shafts are published. Consolidation effects due to groundwater table movements may be more widespread; however, such effects have less impact as movements result in usually relatively insignificant gradients or distortions across structures.

6.2 Ground movements due to shaft construction

6.2.1 Categories of ground movement, factors that affect it and the basis for assessment of ground movements due to shaft construction are described in Appendix B.

6.3 Historical shaft ground movements

6.3.1 There is far less literature on ground movements associated with shafts than on movements associated with tunnels. Some recent and historic results are shown below.

**Heathrow Express Trial Tunnel – Shafts**

6.3.2 The results in Figure 6.1 are from the New and Bowers, 1994, paper from the shaft as part of the Heathrow Express Trial Tunnel. The results shown are those used to develop the formula

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**CASE STUDY: Heathrow Express Trial Tunnel – Shafts**

**Scheme:** The Heathrow Express Trial Tunnel was the first case of the New Austrian Tunnelling Method (NATM) (now known as SCL) being used in London Clay. The British Airport Authority (BAA) decided to invest in this project prior to using this method in the station areas of the Heathrow Express project. The trial consisted of a 10m length of transition tunnel and then three test sections each of approximately 30m in length with differing excavation sequences. An 11m diameter, 26m deep shaft was also constructed using caisson and underpinning methods lined with bolted precast concrete segments as part of these works. Few if any published accounts of shaft movements in London Clay had been produced before this monitoring.
**CASE STUDY: National Grid – London Power Tunnels – Shafts**

**Scheme:** The National Grid project includes 13 shafts across London, to act as launch shafts, excavated material away-points, and permanent access/exit points for cables and personnel during construction and operation.

**Shaft details:** The shafts vary in diameter from 6m to 15m and in depth from 31m to 47m. Construction technique varies across locations due to ground conditions and final use of the structure. Underpinning, SCL and caissons were all used.

**Geotechnical setting:** The geotechnical setting varies from site to site. The base of the shafts is generally founded in London Clay with some in the Lambeth Group. The shafts generally pass through layers of made ground, as well as (potentially) River Terrace Deposits, Alluvium and Harwich/Lambeth Sand.

Presented in B.3.1. Although these results may be limited to the particular conditions, the derived formulas have commonly been used by designers to calculate an estimate for settlements due to shaft construction with justified modifications made.

6.3.3 The S line represents monitoring points away from the tunnelling works, whilst the T line are monitoring points above the tunnel axis, with results shown as the shaft was approaching full depth and tunnel works were just commencing (New and Bowers, 1994). Settlement results shown are limited to approximately 13mm at 3m from the shaft with negligible movements at 20m-25m from the shaft.

**National Grid – London Power Tunnel Shafts**

6.3.4 The Willesden Shaft was approximately 13m excavated diameter and constructed by underpinning with segments to a depth of 20m followed by SCL for a further 9m, giving a total depth of 29m.

6.3.5 The results of settlement monitoring are shown in Figure 6.2 below. The ground movements are greatest adjacent to the shaft but are nowhere greater than 10mm. They are negligible at 30m from the shaft. It can be seen that they are in general less than predicted values using the New and Bowers assessments and the values predicted by methods of the CIRIA Paper No 580 “Embedded Retaining Walls – guidance for economic design”.

![Figure 6.2 | Willesden Shaft ground movements McNicoll, 2013; with New & Bowers, 1994](image-url)
6.3.6 The Channel Gate Road Shaft (excavated diameter 13m) was constructed mainly by SCL. Segments are constructed to a depth of 8m and then SCL for a further 25m to a total depth of 33m. Ground movement data are shown in Figure 6.3 below.

Crossrail – Whitechapel Station – Cambridge Heath Shaft

6.3.7 The Channel Gate Road Shaft appears to have greater settlements, possibly due to the greater depth of the SCL. Both examples shown have peak values of around 10mm to 20mm at approximately 4m to 5m from the shaft. At 10m away from the shaft, settlements tend to be less than 10mm. Figure 6.4 below shows the ground movements recorded to date at the Cambridge Heath Shaft up to the excavation at 23 metres below ground level (mbgl) (to end of March 2013). The total excavated depth of the shaft prior to casting the base will be in excess of 30m. The diaphragm wall panels were excavated between April 2012 and August 2012. During this period, it can be seen that up to 6mm settlement was observed.
CASE STUDY: Crossrail – Whitechapel Station – Cambridge Heath Shaft

**Scheme:** The Crossrail route will link existing Network Rail services from Maidenhead and Heathrow in the west, and Shenfield and Abbey Wood in the east, with new underground stations at Paddington, Bond Street, Tottenham Court Road, Farringdon, Liverpool Street, Whitechapel, Canary Wharf and Woolwich. In total, 21km of new twin-bore tunnels are being constructed to deliver the new rail tunnels through which the Crossrail trains will operate.

**Shaft details:** The shaft is formed within 1.2m thick diaphragm walls and was constructed from April 2012. The diaphragm walls forming the shaft are 40m in length. The shaft has an internal diameter (between diaphragm wall panels) of 28.2m. Total excavated depth will be in excess of 30m prior to casting the base slab.

**Geotechnical setting:** Ground conditions comprise of around 7m of superficial deposits comprising made ground and River Terrace deposits overlying London Clay to approximately 31m below ground level. De-pressurisation is proposed to be used ahead of excavating the shaft from 23m below ground to the base level.
6.3.8 Pilot SCL tunnels were excavated nearby to the shaft in late 2012 and movements associated with these excavations have been removed from the movements presented in the Figure 6.4 below. Despite this, some further settlement was reported between completion of the diaphragm wall panels and start of the shaft excavation (up to 6mm close to the diaphragm walls, but becoming less with increasing distance from the shaft). It is thought that some consolidation-related movements may have arisen during this time. The excavation of the shaft started on 28 January 2013. Very little excavation-related movement is recorded.

**Lee Tunnel – Shafts**

6.3.9 Figure 6.5 opposite shows the Abbey Mills Reception shaft in plan. The shaft is 25m in diameter and 66.8m deep to internal finished floor level (and 72.9m to the base of the excavation).

6.3.10 Figure 6.6 and Figure 6.7 show the vertical movement of monitoring points 6000 to 6003 and 6100 to 6103 between May 2012 and January 2013 as the shaft is excavated. A maximum of 6mm of ground movement was measured at approximately 6m from the wall of the shaft. Some ground movements could have been caused by construction of the diaphragm walls but are not captured in the monitoring data shown.
CASE STUDY: Lee Tunnel – Shafts

**Scheme:** The Lee Tunnel is being built as part of the new Thames Tunnel project. It comprises deep shafts, a large pumping station and 7km long, 7.2m ID deep tunnel between Beckton Sewage treatment works and Abbey Mills pumping station.

**Shaft details:** Four shafts are constructed as part of the project; three located on the Beckton site and one at the Abbey Mills site. Internal diameters vary between 20 and 38m. The shafts are between 66.8 and 77.8m deep (from ground level to internal floor finish level) constructed using diaphragm walls.

**Geotechnical setting:** The geology of the Beckton site consists of made ground, River Terrace Deposits, London Clay, Harwich Formation, Lambeth Group, Thanet Sand all overlying Chalk. The top of the Chalk layer is approximately 40m-55m below ground level.
6.3.11 Figure 6.8 shows the plan of the Beckton Pumping Shaft and Connection Shaft. The Pumping Shaft is 38m diameter and the Connection Shaft is 25m in diameter. Both are approximately 80m in depth. Figure 6.9 shows the settlement data from array 5101-5104 across the 38m diameter Pumping Shaft. A maximum ground movement of 13mm was measured approximately 2m-3m from the shaft wall.

6.3.12 Maximum settlements of 13mm were measured in all the arrays shown in Figure 6.5 and Figure 6.8, with monitoring points located within 5m of the shaft wall. Despite the shafts being very deep, the ground movements are relatively small and are far smaller than the New and Bowers (1994) equation would predict. Diaphragm walls form a stiff structure which limits movement during excavation and in addition the Chalk is a relatively stiff material.

6.4 Current practice methods for shafts

6.4.1 Selection of the method(s) of shaft construction depends on the project requirements and the ground and groundwater conditions. In general, significant ground movements around shaft excavations are localised and in many cases restricted to the shaft site area. Consolidation settlements due to drainage of groundwater are likely to be widespread, but uniform, and therefore less significant. Shaft locations can be selected where possible to minimise effects of settlement on surrounding infrastructure.
6.4.2 Where there may be an issue, ground movements due to shaft construction may be reduced by:

- installing ground support before excavation using diaphragm walls, secant or contiguous piles;
- using top-down construction to provide greater stiffness to the structure; or
- avoiding effects on the groundwater table by constructing a watertight shaft and, if using caissons, sinking in the wet, rather than dry.
Open cut/cut and cover construction and ground movement

7.1 Introduction

Building HS2 will involve the construction of several open cuts as well as tunnels. Open cuts will be constructed both as temporary works (e.g. for the construction of an underground station, cut-and-cover tunnel, etc.) and permanent works (e.g. for the construction of a tunnel portal). In areas with abundant construction space, unsupported sloping open cuts can be considered. However, in an urban environment or where there is lack of available space, the open cuts will be vertical – these will need to be supported with an earth-retaining structure that will help to control and reduce ground movement. The basis for assessment of ground movement due to open cut excavation is discussed in Appendix C.

7.2 Case history data

Short-term movement

7.2.1 There is a significant amount of published data related to excavation-induced ground movements, including empirical approaches for estimating ground movements.

7.2.2 M. Long (2001) in his “Database for retaining wall and ground movements due to deep excavations” has collected and presented some 300 case histories of wall and ground movements due to deep excavations worldwide (of which around 60 are in the UK). He divided the data into four sets according to the particular characteristics of each case. These are illustrated in Figure 7.1 (where ‘FoS’ refers to the Factor of Safety against base instability). Sets 1 and 4 are most relevant to the majority of conditions anticipated along the HS2 project where open excavations will be built.

Figure 7.1 | Subdivision of data, after M. Long database (2001)

7.2.3 From this analysis for Set 4, Long concluded that for propped retaining walls mainly in stiff soils (SET 1), normalised maximum lateral movement values are frequently between 0.05% and 0.25% of the excavation depth and normalised maximum vertical settlement values are usually even lower: between 0% and 0.20% of the excavation depth (Figure 7.2). Hence, for a 10m deep excavation, settlement would be expected to be less than 20mm behind the retaining wall.

7.2.4 If the data that refer to retaining structures constructed in the UK is extracted from the database, the vertical settlements average to approximately 0.15% of the excavation depth and lateral movements to 0.18% of the excavation depth (Figure 7.3).
For cantilever walls (SET 4) the normalised maximum lateral movements are relatively modest around 0.36% of the excavation depth (Figure 7.4) and are independent of the excavation depth and system stiffness (i.e. the stiffness of the retaining wall props and soil in the passive zone).
7.2.6 Fernie and Suckling’s publication in 1996 describes their approach to estimating lateral wall movement of embedded walls due to excavation in UK ground conditions, which was based on semi-empirical approaches to lateral wall movement predictions (developed in the United States, Clough and O’Rourke, 1990) for propped walls and applied for UK stiff soils. It was concluded that lateral movement of propped embedded walls in UK stiff soils is likely to be 0.15% of the retained height and will not exceed 0.3% of the retained height.

7.2.7

Figure 7.5 and Figure 7.6 are reproduced from CIRIA Report C580, showing a collection of data from Clough and O’Rourke (1990), Thompson (1991), Carder (1995) and Carder et al. (1997) of ground surface movements (horizontal and vertical) due to bored pile and diaphragm wall installation. The majority of recorded movements are small (less than 0.02% of wall depth, i.e. less than 5mm for a 25m deep retaining wall). Larger movements have usually been due to the pile bore being left open for an extended period (e.g. Bell Common Tunnel).

7.2.8 These diagrams show that for both bored piled walls and diaphragm walls installed in stiff clays horizontal movements tend to become insignificant at distances greater than 1.5 times the wall depth (i.e. for a 15m deep wall no horizontal movement at 22.5m away from the wall and at 7.5m away from the wall horizontal movement is as little as 3-5mm). With regards to the vertical settlements none were recorded at distances greater than 2 times the wall depth for bored piled walls and 1.5 times the wall depth for diaphragm walls (i.e. for a 15 deep diaphragm wall near to zero settlements are recorded at a distance 22.5m away from the wall and at 5m away from the wall settlements can be as little as 3mm).
7.2.9 Observations (over a period of less than one year) from cantilever walls in Glacial Tills in Dublin show that the walls perform extremely well with lateral movements less than 0.1% of the excavation depth for excavation depths up to 8m (O’Brien, 2010).

Figure 7.5 | Ground surface movements due to bored pile wall installation in stiff clay (CIRIA C580)

Figure 7.6 | Ground surface movements due to diaphragm wall installation in stiff clay (CIRIA C580)
CASE STUDY: Heathrow – Airside Road Tunnel

Scheme: The Airside Road Tunnel (ART) project at Heathrow Airport includes a 1.3km long twin bored tunnel and two earth-retaining structures which form the approach portals. It links the Central Terminal Area – Terminals 1, 2, 3 (East Portal) to the western edge of the airport (West Portal) and also provides a link to Terminal 5. Construction of this road tunnel was undertaken by British Airports Authority (BAA) between 2001 and 2005, when it became operational, in order to improve the airside traffic operations.

Open cut details: The portals included the launching and receiving chambers for the TBM, which were approximately 30m wide by 20m long by 17m deep, straight sections of cut and cover tunnel approximately 40m long, and long, curving ramps approximately 150m long with variable retained height. The West Portal is located above part of the LUL Piccadilly Line Tunnel.

Geotechnical setting: The portals were constructed in ground which included 4m-5m thick Terrace Gravel deposits (dense to very dense sandy medium to coarse gravel) followed by London Clay with a top weathered layer (0.5m to 1.0m thick) above the unweathered layer (firm to stiff with partings of silt and bands of claystones up to 50m thick). A thin layer of made ground (up to 1m thick) was also present.

Key features: Monitoring of ground movements during construction of the portals allowed for a construction sequence to be adopted that enabled all strutting to be eliminated. This lead to considerable programme and material savings.
Long-term movements

7.2.10 Selected walls have been instrumented and monitored for many years whilst in service. The Transport Research Laboratory has produced a report on the long term performance of embedded retaining walls covering wall instrumentation during construction that has continued to be read over many years post construction (Transport Research Laboratory (TRL) Report 381, Carder and Darley, 1998). These walls include a cantilever diaphragm wall in Reading, a propped bored piled wall in New Malden and an anchored diaphragm wall in Neasden. Observations of measurements behind and in front of the wall are summarised in Table 7-1 below.

<table>
<thead>
<tr>
<th>Wall Type</th>
<th>Reading</th>
<th>New Malden Carriageway</th>
<th>Neasden Carriageway</th>
<th>Long Cut &amp; Cover Tunnel Bell Common</th>
</tr>
</thead>
<tbody>
<tr>
<td>Construction Year</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1972</td>
<td></td>
<td></td>
<td>1972</td>
<td>1982</td>
</tr>
<tr>
<td>Instrumentation Installation Year</td>
<td></td>
<td></td>
<td>1984</td>
<td>1982</td>
</tr>
<tr>
<td>1984</td>
<td></td>
<td></td>
<td>1988</td>
<td></td>
</tr>
<tr>
<td>Duration of monitoring</td>
<td>12 years</td>
<td>11 years</td>
<td>8 years</td>
<td>11 years</td>
</tr>
<tr>
<td>Ground</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Conditions</td>
<td>Terrace Gravels over London Clay (very stiff below 14m with sand bands)</td>
<td>Made ground over London Clay (very stiff below 7m)</td>
<td>Stiff London Clay foundation</td>
<td>London Clay</td>
</tr>
</tbody>
</table>

Table 7.1 | Summary of case histories of open cut instrumentation whilst wall in service (TRL Report 381, Carder and Darley, 1998).

Heathrow – Airside Road Tunnel

7.2.11 With regards to the Airside Road Tunnel West Portal TBM launch chamber wall performance there is a significant number of inclinometer readings that were recorded (Hitchcock A, et al. 2004). Wall convergence to first cut level (approximately 4m below top of wall) was small, in the order of 3mm. From excavation below that level the walls converged a further 15mm up to the point where the blinding strut was cast and a further 5mm occurred up to base slab installation. For the top down TBM launch chamber a maximum wall displacement of 17mm occurred at 2/3 depth for an excavation depth of 16m below ground level.

7.2.12 For the Airside Road Tunnel East Portal ramp, the cantilever walls converged approximately 5mm at capping beam level after excavation to first cut level (3m below capping beam level), followed by a further 8mm to 10mm convergence resulting from excavation of the preceding bays. During excavation to formation and casting of the blinding strut, a further 5mm convergence was recorded. Due to the curved alignment of the ramp, the wall displacements were not symmetrical, but in general for a section of the cantilever wall monitored, a maximum wall displacement of approximately 10mm was observed for an excavation depth of 8m.
7.2.13 The aforementioned observations of wall deflections for the ART tunnel portals compare favourably to the case history data for lateral wall movements in stiff clays presented in sections 7.2.3 to 7.2.9 of this report. It was therefore confirmed that the controlled excavation sequence and rapid installation of the blinding struts contributed to keeping wall movements low.

7.3 Mitigation measures

7.3.1 Burland et al. (1979) highlighted that it is impossible to avoid ground movement when an excavation (i.e. unloading) takes place, and that the retaining structures will transfer and reduce but not eliminate ground movement. It has been shown that the differential movements are the most significant to the potential risk of damage to buildings and structures, and both vertical and horizontal movements need to be considered. Thus, controlling the ground movement to acceptable levels is the key issue for the design and construction of open cut excavations in urban areas.

7.3.2 A wide range of measures have been successfully used for several decades in order to control and minimise ground movements. Some of the most common methods are outlined in CIRIA C580 “Guidance for Economic Design of Embedded Retaining Walls”, which refers to:

- the provision of adequate embedment for satisfactory vertical and lateral stability;
- the reduction of the first-stage cantilever excavation depth and installation of the first level of props (or anchors) as early as possible;
- the reduction of the extent of the dig beyond the level of the support;
- the reduction of delays to the construction sequence (which applies for both the wall and the excavation);
- the avoidance of over-excavation at any stage;
- the protection of any clay berms that provide support;
- the reduction of removal of fines during dewatering; and
- the reduction of drawdown outside the excavation.

7.3.3 Most of the measures are associated with the construction methodology, especially the timing and sequence of installing supports to the retaining wall. These will be taken into account in the analysis and design stage of the project.

7.3.4 Instrumentation schemes during construction offer a valuable means to monitor ground movements and the response of structures and to compare these against predictions. If large deviations are observed between the values measured and predicted, actions can be implemented for design verification (e.g. back analyses), construction mitigation (e.g. increase in number of props) and building protection against damage (e.g. provision of supports).
7.3.5 There is no reason to consider that all ground movements from open cut excavations are detrimental unless they may affect the structure itself or nearby buildings, services or tunnels. It is generally recognised that for embedded retaining walls, excessive movements are most likely to be due to poor understanding of geological and hydrogeological conditions of the site, poor design and construction details and poor workmanship, or construction control. Experience indicates that current design practice is conservative and leads to relatively small wall and ground movements, provided that construction is undertaken properly.

7.3.6 An open cut excavation can be carried out safely with minimal risks when a thorough ground investigation and interpretation has been undertaken in the design; appropriate and relevant case history data have been referred to; quality of construction is ensured; and a thorough monitoring scheme is in place.
8 Ground-borne noise and vibration from rail tunnels

8.1 Introduction

8.1.1 This section of the report discusses the technical background to ground-borne noise and vibration as a result of underground works during construction and operation.

8.1.2 Results from the UK’s recent projects are presented in this report, demonstrating the ability to manage and control the effects of ground-borne noise and vibration, hence limiting the impacts on the people and assets located near the infrastructure.

8.1.3 This section presents current practice mitigation measures and considers lessons learned.

Noise and vibration

8.1.4 The terms ‘sound’ and ‘noise’ are used in this section. ‘Sound’ is the neutral term used to describe the fluctuating pressure waves in the air that stimulate the sense of hearing. Sound only becomes noise when it exists in the wrong place or at the wrong time, such that it causes or contributes to some harmful or otherwise unwanted effect, like annoyance or sleep disturbance.

8.1.5 Without mitigation, ground-borne vibration created by either construction activities or train services can propagate through the ground to surrounding buildings where it may result in the vibration of floors, walls and ceilings; which could also be heard as a low frequency ‘rumbling’ sound (called ground-borne sound).

Ground-borne noise

8.1.6 Absolute criteria, rather than sound change criteria, apply for ground-borne sound for four main reasons, as follows:

• The character and nature of ground-borne sound differs from other ambient sound heard inside buildings;

• The body of experience and research available with regard to human response to ground-borne sound has mostly been based on the assessment of the maximum sound level for each train pass-by (i.e. an absolute sound level);

• Ground-borne sound can affect any room in a property so the criteria consider situations where existing internal background sound levels are at their lowest for a particular classification of receptor (e.g. rooms on a quiet façade of a residential receptor or new build concert hall or broadcast facility); and

• There is rarely any appreciable existing ground-borne sound at a receptor.

Ground-borne vibration

8.1.7 Without mitigation, ground-borne vibration could cause the following types of significant adverse effect:

• There might be a risk of cosmetic damage to buildings (e.g. cracking in plaster) at very high levels of vibration, which very rarely occur adjacent to modern railways;

• Perceptible vibration in residential buildings; and

• Low levels of vibration that would be imperceptible to people can adversely affect buildings where low ambient vibration is critical to operations (e.g. nanotechnology laboratories).
Source of ground-borne noise and vibration

8.1.8 The following are potential sources of ground-borne noise and vibration:

- Construction (Temporary) sources: e.g. tunnel boring machine(s) and their supporting temporary construction railways, some types of piling and vibrocompaction; and

- Operational (Permanent) sources: train operation and to a lesser extent other rail systems such as infrastructure maintenance depots.

8.1.9 It is current practice during construction to use ‘best practicable means’ to control and mitigate temporary construction noise and vibration effects consistent with legislation. ‘Best practicable means’ include consideration of working methods, working hours, selection of plant, logistical planning physical barriers and proactive community engagement. The framework for determining such mitigation on a site-by-site basis is usually set out in the Code of Construction Practice for the project.

8.1.10 For the operational railway, significant ground-borne noise and vibration effects may be reduced or removed through, for example, the performance specification and design of the rolling stock and infrastructure (especially the track system).

Units

8.1.11 The ground-borne sound level is reported as $L_{pA}^{S\text{max}}$ in decibels, where:

- $L_p$ denotes the Sound Pressure Level;
- $A$ denotes A-weighting, that represents the sensitivity of the human ear;
- $S$ denotes Slow time response over 1 second; and
- $\text{max}$ denotes the maximum value during a single train pass-by.

8.1.12 ‘Vibration’ can be defined as oscillation of matter about a fixed reference point. Units of displacement, velocity or acceleration are used to describe and quantify vibrations.

8.1.13 The effect of vibration on people is described in terms of a level of annoyance caused by the cumulative effects of the vibration over a day. This is expressed as a Vibration Dose Value (VDV).

8.1.14 The Peak Particle velocity (PPV) is the highest instantaneous vibration velocity. This has been found to give a good predictor of potential damage to buildings. However, as the level of vibration required to cause even very minor damage to buildings is many times higher than would be acceptable to inhabitants, these limits are not relevant when considering vibration from the construction and operation of new railway tunnels.
8.2 Impact criteria for ground-borne noise and vibration

**Ground-borne noise**

8.2.1 Airborne noise is usually assessed by considering the predicted levels during operation with the current noise levels. However, for ground-borne sound, there are no relevant national or international standards to set criteria for the acceptable levels. The requirements are generally specified for each project and may identify specific buildings of types of building that have different requirements. The impact criteria in general use are set out in Table 8.1 and Table 8.2. These have been drawn from similar projects in the UK and Ireland (e.g. Crossrail, the Jubilee Line Extension, DART Underground, Dublin Metro North and HS1). These projects assess ground-borne sound in terms of the absolute level of sound generated by a train passing by.

<table>
<thead>
<tr>
<th>Impact classification</th>
<th>Ground-borne sound level dB $L_{PASmax}$</th>
<th>Description*</th>
<th>Existing example (where there are similar levels of ground-borne noise)*</th>
</tr>
</thead>
<tbody>
<tr>
<td>Negligible</td>
<td>&lt; 35</td>
<td>The passage of trains may be audible to particularly sensitive people during quiet periods of the day in rooms with low background noise. Very unlikely to cause complaint.</td>
<td>Recent rail tunnels such as Jubilee Line Extension and HS1.</td>
</tr>
<tr>
<td>Low</td>
<td>35–39</td>
<td>The passage of trains may be audible particularly during quieter periods of the day such as evening or early morning. Level of annoyance is likely to be low with few complaints.</td>
<td>Ground floor room 20–70 metres from London Underground Limited tunnel. Levels dependent on tunnel depth, ground-type and train speed.3</td>
</tr>
<tr>
<td>Medium</td>
<td>40–44</td>
<td>The passage of trains is likely to be audible regardless of the time of day. Levels likely to give rise to some annoyance regardless of time of day. There may be some complaints.</td>
<td>Ground floor room 10–40 metres from London Underground Limited tunnel. Levels dependent on tunnel depth and ground-type.3</td>
</tr>
<tr>
<td>High</td>
<td>45–49</td>
<td>Noise from the passage of trains will tend to be prominent and give rise to annoyance regardless of time of day. It is likely that there will be some complaints.</td>
<td>Directly above some atypical existing London Underground Limited lines (e.g. shallow tunnel with poor quality jointed rails).</td>
</tr>
<tr>
<td>Very high</td>
<td>&gt;49</td>
<td>During the passage of trains ground-borne noise will probably dominate above noise from other sources (road traffic etc). Considerable annoyance likely throughout the day and night. There may be some sleep disturbance. Complaints very likely.</td>
<td>Directly above some exceptional sections of existing London Underground lines (e.g. extremely shallow tunnel with very poor quality jointed rails).</td>
</tr>
</tbody>
</table>

Table 8.1 | Ground-borne sound impact criteria for residential receptors with illustrative descriptions

1. The descriptions relate to the possible subjective impression of these levels of ground-borne noise as a result of the operation of a new railway.

2. The examples are made on the basis that LUL ground-borne noise is of a similar spectral character to that predicted for the passage of trains on tunnelled sections of HS2.

3. The examples for low and medium categories assume an underground railway of similar construction to the London Underground Central Line.

<table>
<thead>
<tr>
<th>Category of Building</th>
<th>Impact criterion dB $L_{PASmax}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Theatres/large auditoria and concert halls</td>
<td>25</td>
</tr>
<tr>
<td>Sound recording/broadcast studios</td>
<td>30</td>
</tr>
<tr>
<td>Places of meeting for religious worship/courts/cinema/lecture theatres/museums/small auditoria or halls</td>
<td>35</td>
</tr>
<tr>
<td>Offices/schools/colleges/hospitals/hotels/libraries</td>
<td>40</td>
</tr>
</tbody>
</table>

Table 8.2 | Ground-borne sound impact criteria for non-residential receptors
8.2.2 No matter how quiet dwellings may be, the background levels will never be silent, even at night; indeed, absolute silence can become oppressive. However, the criteria for non-residential receptors reflect that some buildings – such as world-class concert halls, theatres or broadcast studios – are based on backgrounds that are practically silent. The residential and non-residential criteria presented in Table 8.1 and Table 8.2 have been subject to parliamentary and public inquiry (Channel Tunnel Rail Link, Parliamentary inquiries 1994 – 1995; Crossrail Bill Parliamentary inquiries 2005 – 2007; Forth Crossing Bill Parliamentary inquiries; and DART Underground, oral hearing – 2010 – 2011) and are set out in recent international guidance, such as that published by the US federal transit and railroad authorities.

Ground-borne vibration

8.2.3 Ground-borne vibration is generally assessed against two criteria: annoyance to people and damage to buildings.

8.2.4 BS6472:2008 Part 1 gives advice on human response to cumulative levels of vibration through the day and night. The level of annoyance is very subjective. The vibration level is expressed as a Vibration Dose Value (VDV).

8.2.5 Some non-residential properties may be more susceptible to vibration. These may include scientific laboratories using electron microscopes or sensitive manufacturing facilities. These are typically dealt with on a case-by-case basis.

<table>
<thead>
<tr>
<th>Impact classification</th>
<th>Vibration Dose Value(^1) (m/s(^{1.75}))</th>
<th>Description(^2)</th>
<th>Existing example (where there is a similar magnitude of ground-borne vibration)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Day (0700-2300)</td>
<td>Night (2300 – 0700)</td>
<td></td>
</tr>
<tr>
<td>Negligible</td>
<td>≤ 0.2</td>
<td>≤ 0.1</td>
<td>Adverse comment not expected(^2). Individual events may be perceptible to some people at some times of the day/night.</td>
</tr>
<tr>
<td>Minor</td>
<td>&gt; 0.2 – 0.4</td>
<td>&gt;0.1 – 0.2</td>
<td>Low probability of adverse comment(^2). Individual events likely to be perceptible to some people at some times of day/night.</td>
</tr>
<tr>
<td>Moderate</td>
<td>&gt; 0.4 – 0.8</td>
<td>&gt; 0.2 – 0.4</td>
<td>Adverse comment possible(^2). Events will be perceptible to most people.</td>
</tr>
<tr>
<td>Major</td>
<td>&gt; 0.8</td>
<td>&gt; 0.4</td>
<td>Adverse comment probable. Events will be perceptible to all.</td>
</tr>
</tbody>
</table>

Table 8.3 | Vibration impact criteria for the disturbance (annoyance) of occupants and building users with illustrative descriptions

1. As defined in BS 6472-1 (2008).
2. The descriptions are as defined in BS 6472-1 (2008) and relate to the possible typical subjective impression of these magnitudes of ground-borne vibration as the result of the operation of a new railway. BS6472 includes a VDV category above 1.6 where adverse comment is “very likely”.
3. This project was assessed, designed and delivered in accordance with the 1992 version of the British Standard BS 6472, where the calculated or measured vertical VDV is approximately half that calculated or measured using the 2008 version of the standard.
4. The examples for low and medium categories assume an underground railway of similar construction to the London Underground Central Line as measured in the 1990s (London Underground has replaced trains and been progressively improving track and maintenance since this time and hence it is likely that current exposure is reduced from that measured in the 1990s).
Effect of vibration on buildings

8.2.7 As noted in ISO 14837-1, "Extremely high levels of ground-borne vibration or a large number of vibration cycles of high magnitude can, in unusual circumstances, give rise to risk of damage to building structures either through direct stress/strain on building components or through induced settlement in cohesion-less soils and fill. The vibration levels required are of the order of 10 to 100 times larger than those associated with human perception and thus levels of vibration sufficient to damage a building, even cosmetically, would be intolerable to occupants".

8.3 Factors affecting noise and vibration

Sources of noise and vibration

8.3.1 There are two distinct phases in the lifetime of a tunnel when noise and vibration need to be considered: construction and operation.

8.3.2 During the construction phase, perceptible ground-borne noise and vibration can be generated as the TBM excavates the tunnel and from fixed equipment within the tunnel. The noise from the TBM depends largely on the type of ground being excavated.

8.3.3 The second source during construction will be from the temporary railway laid inside the tunnel to transport material and personnel to the TBM. These temporary railways are laid in short lengths as the TBM advances; therefore, there will be joints in the rails. The noise and vibration from this railway is controlled by the design and quality of the rolling stock and track, as well as the levels of track maintenance.

8.3.4 Experience in London and other locations has shown that the passage of a TBM is likely to generate perceptible ground-borne noise and vibration impacts, but only at receptors within a close distance of the centreline of tunnel being constructed, and only for short periods of time, such as a few days. In general, the deeper the tunnel, the lower the impact. The perceptible noise and vibration would increase as the TBM approaches and diminish as it moves away. Vibration from TBMs has been shown to present no risk of building damage.

8.3.5 The effects on building occupants will be short term – a matter of days – and hence are not considered significant. Projects such as HS1 have shown that proactive and advanced community relations in advance of the TBM passing under properties avoids any surprise and allays concerns.

8.3.6 The passage of the TBM may be noticeable for a few days. The temporary railway will be in operation for a longer period – this depends on the rate of tunnelling and the distance from the access point. As described in the case studies later in this section, earlier projects such as HS1 provide lessons in controlling ground-borne noise and vibration from the temporary construction railway – for example, by maintaining the temporary track. Projects such as Crossrail are being delivered with the aid of these lessons and also with latest technology in construction railway rolling stock. Crossrail's construction is showing that significant noise and vibration effects arising from use of the temporary railway can be avoided by a design and maintenance specification.

8.3.7 Once a railway is in operation, the passage of trains generates vibration in the ground. The level of noise and vibration in neighbouring buildings is dependent on the forces applied by the train to the rails and the attenuation between the rails and the buildings. This attenuation can be calculated and the level of mitigation can be designed to achieve acceptable levels of noise and vibration in the buildings.
Calculation procedure for noise and vibrations from trains

8.3.8 The levels of noise and vibration are calculated using a range of methodologies proven by external scrutiny and successful project delivery, although there is no national method defined. The methodologies available can also be used to assess the performance of ground-borne noise or vibration reduction measures where these are required.

8.3.9 The framework for current practice in prediction methodologies including a structured process for demonstrating validation and verification is provided in the International Standard ISO 14837 part 1.

8.3.10 The calculations typically consider the vibration at the source (the contact between wheel and rail) and the losses along the path to the receptor as indicated diagrammatically in Figure 8.1 above.

8.4 Control of ground noise and vibration for major infrastructure projects

8.4.1 Over the last 20 years or so, tunnelled rail schemes have been successfully delivered and now operate with no ground-borne noise or vibration impact or a minimised level of impact. In part, this is due to the introduction of the Environmental Impact Assessment Directive and the associated UK regulations in 1988. The control stems from assessment and management throughout the development of the project that can be divided into three distinct but integrated stages: planning, approval and delivery.

Planning stage

8.4.2 During this stage, an assessment of ground-borne noise and vibration will be undertaken as part of the early scheme development. The assessment forms part of an Environmental Impact Assessment (EIA), the outcomes of which are published in an Environmental Statement (ES). The ES is submitted to the decision makers for the process to seek powers to construct and then operate the proposed scheme (e.g. application for an Order under the Transport and Works Act or a Bill placed before Parliament). The ES is published publicly at the time of its submission. The assessment will involve calculations of ground-borne noise and vibration, using calculation methods either published by organisations such as the Federal Railroad Administration or developed and verified by practitioners employed by the promoter of the proposed scheme following the guidance in ISO 14837-1. The calculated levels of ground-borne noise and vibration are compared with impact criteria as part of the evaluation of likely significant effects on the range of receptors located over the proposed tunnel. Examples of impact criteria are given in section 8.2.
8.4.3 The ES sets out envisaged mitigation measures to avoid or reduce any significant effects that are identified. The ES will also present any likely residual significant effects – these are significant effects that are likely to arise taking account of the envisaged mitigation.

8.4.4 The application for powers to construct and operate the proposed scheme is usually supported by a number of other documents such as environmental policies, a Code of Construction Practice, and environmental design aims.

**Approval stage**

8.4.5 Any challenge to the findings of the ES may form an objection or petition made by an interested party against the application submitted to the decision maker. The decision making process invariably includes a Public Inquiry that will consider objections/petitions that have not been resolved before the Inquiry. For a Bill, a Parliamentary inquiry, or inquiries, are held by a Select Committee, or Select Committees, specifically established by Parliament and comprising Members of Parliament or Lords who have no interests in the proposed scheme. The inquiry hears evidence in respect of petitions against the Bill as well as from the proposed scheme’s promoter.

8.4.6 Taking account of the evidence heard at any inquiry, the decision maker may reject the application for powers, grant the application or grant the application with various modifications and commitments. Modifications may include additional mitigation or protective measures. These modifications, as well as any measures proposed as part of the application for powers, are made binding either by Conditions imposed on an Order or by Undertakings and Assurances made before Parliament.

8.4.7 The application, if granted, usually provides outline planning permission. The submission to the decision maker will usually therefore also include a mechanism by which detailed planning has to be secured. Commitments often also cover a range of other secondary consents such as control of ground-borne noise and vibration from construction under Section 61 of the Control of Pollution Act. Whilst secondary consents will include matters such as noise barriers, they do not extend to the approval of track work as it would not be appropriate for a local authority to take responsibility for such a safety critical item. Instead it is common for there to be a commitment for the deliverer to continue technical engagement with the local authorities.

**Delivery stage**

8.4.8 The detailed design and construction methodologies for the proposed scheme must be developed in compliance with the commitments imposed at the approval stage. The organisation delivering the proposed scheme (termed the ‘nominated undertaker’ if it is a Bill) will have to seek the required secondary consents. The consents will generally have to be secured before construction is started. The decision makers for the secondary consents, often local authorities and other statutory bodies, have a responsibility to assure the deliverer’s compliance based on information submitted to them.

8.4.9 Compliance is generally determined by the secondary consents and any other permits required. This is because it is critical that the required mitigation is designed into the scheme and delivered during construction as it is not practicable to provide additional mitigation after the start of operation.
8.5 Historical performance of noise and vibration

Historical background

8.5.1 Early rail tunnels caused significant levels of noise and vibration in properties above them. The main reasons were:

- Poor rail alignment: It was difficult to control the exact position of the top of the rail, resulting in additional variations of force between the wheel and the rail. Modern techniques ensure that the rail is positioned far more precisely during construction.

- Rolling stock suspension: The suspension design and operation of early trains meant that most or all of the mass of the locomotive/carriage would contribute to the dynamic forces at the wheel/rail interface, increasing ground-borne noise and vibration. Modern train suspension isolates much of the vehicle's mass from the dynamic forces that cause ground-borne noise and vibration.

- Rail joints: Originally, the rails were made up from 60-foot-long (18.3m-long) sections, connected together with fishplates. These joints produced the 'clickety-clack' noise heard from a surface railway and the additional impact as each wheel crossed the joint generated an impact force that was transmitted into the ground. Modern track is formed from continuous welded rails, removing these impact forces.

- Poor track and wheel maintenance: Much of the noise and vibration is generated because of roughness in the rail head and wheel surface. Even though a rail may appear shiny and smooth, there are variations in level that impart additional forces. Modern monitoring equipment and rail and wheel grinding techniques are used to maintain both surfaces to a high standard in order to reduce the noise from roughness. A high standard of maintenance is also necessary to ensure passenger ride comfort at high speed. These measures reduce wear and tear as well as ground-borne noise and vibration.

- Rigid track supports: The rails were rigidly fastened to sleepers which were supported on a relatively thin layer of ballast in the tunnel. Modern track includes resilient pads directly under the rails to help isolate the vibrations. The track itself is often separated from the tunnel lining to reduce the transmitted vibration. An example of vibration isolation is shown below in Figure 8.2 below.

Thanks to the improvements in track and rolling stock design, London Underground receives substantially fewer complaints from building occupiers above the newer tunnels, such as the Jubilee Line, than from the older lines.

Figure 8.2 | Resilient pad beneath rail fastening to isolate vibration.
Modern track

8.5.2 Many of the causes of vibration and ground-borne noise experienced by underground metro trains do not arise with modern high speed trains.

8.5.3 Magnitudes of vibration and ground-borne noise levels are directly proportional to the dynamic forces created by a moving train. In simple terms, a smooth-running train on smooth rails will produce lower vibration excitation forces and hence lower vibration and ground-borne noise levels.

8.5.4 From a ground-borne noise and vibration perspective, the differences are considerable. Many of the noise-generating sources found in older trains and tunnels no longer exist in high speed trains and modern tunnels.

8.5.5 For many reasons, the operating speeds of high speed trains require precise controls that are not required in low speed metro stock. For example, out-of-balance wheels or wheel flats, sometimes found on older metro stock, would be unacceptable during high speed operation, as would be the large, sudden train movements from poor tracking and guidance problems found in old tunnels and rolling stock. The sophisticated measures used in modern rolling stock to control car body movements and ensure directional stability are non-existent in low speed metro stock. Some of the controls in place on high speed lines that affect vibration reduction include computer-controlled air suspension, variable dampers, traction and anti-locking braking systems, very large radius curves (with radii measured in kilometres rather than metres), wheel dampers, high precision laser aligning equipment for track laying, and resiliently mounted continuous rails.

8.5.6 Travelling at speeds of 250mph and above requires wheels that are smooth, true and accurately balanced. The modern technology in the latest wheel-grinding equipment ensures that the wheels are true and finely balanced, so that vibration-inducing contact force fluctuations from unbalanced and out-of-round wheels are reduced to insignificant levels.

8.5.7 High speed track must be very carefully maintained. Routine inspections, in conjunction with onboard computer monitoring, will identify signs of abnormal wear patterns and defects. The much higher standards and tighter tolerances will ensure that the appropriate corrective action is taken.

8.5.8 The rails will be continuously welded and smoothly ground so that there are effectively no joints in the rail. In the past, jointed rails have been a major contributor to ground-borne noise and vibration.

Historical improvements in noise performance

8.5.9 The improvements in rail tunnel performance are shown in Figure 8.3. Since each tunnel affects a large number of properties, the value given is only approximate. The figure shows the date of opening for each tunnel and the noise level in a building above it. (Points 1, 2 and 4 represent recent measurements taken for old tunnels.) The introduction of the Environmental Impact Assessment process in 1988 is shown by a blue line. Schemes designed after that date have a design aim to maintain $L_{pA;Max}$ below 40dB(A).

8.5.10 The information shows that the ground-borne noise from tunnels built more than 100 years ago is sufficiently loud to cause complaints (45dB-55dB). More recent rail tunnels have achieved much lower noise levels where the use of mitigation has been justified. For the Docklands Light Railway Lewisham and Woolwich Arsenal
Impacts of tunnels in the UK | Ground-borne noise and vibration from rail tunnels

extensions, the Jubilee Line Extension and High Speed 1 tunnels (points 12, 13, 14 and 15) the sound levels are negligible (<35dB).

8.5.11 The case study panel for CTRL (Now HS1) provides the key parameters for the construction and operation of CTRL.

8.5.12 The following sections set out how ground-borne noise and vibration impacts were evaluated, managed and either avoided or reduced during the development and delivery of the scheme, drawing on the number of publications (Allett et al, 2002) (Greer et al, 2004).

The planning stage (1991 to 1994)

8.5.13 CTRL was the subject of what was then the largest Environmental Impact Assessment ever undertaken in Europe. The Environmental Statement, which was submitted in support of the Channel Tunnel Rail Link Bill lodged with Parliament in November 1994, included three specialist studies related to noise and vibration effects: airborne noise from the operation of the line; airborne noise from the construction of the line; and ground-borne noise and vibration from the construction and operation of the line.

8.5.14 When the project was conceived, there were no standardised or accepted method for predicting or assessing noise and vibration from the operation of railways. The promoter – first the British Railways Board, then Union Railways – funded the development and validation of prediction methods for all forms of noise and vibration specifically for high speed rail. Assessment criteria were developed based on available guidance, project experience and complaint histories available for the underground lines in London.

8.5.15 The prediction methods for ground-borne noise and vibration from the tunnelled section of the CTRL, were developed by analysing thousands of measurements obtained over tunnels on the French and German high speed lines and also over many railways in the UK (Greer, 1999). This provided methods based on real data that covered the range of train speeds, train types, track types tunnel depths, ground conditions, and building types necessary to consider the proposed CTRL scheme.

8.5.16 The project developed two levels of track mitigation considered in the assessment.
**The approval stage (1994 to 1996)**

8.5.17 For construction, the approval stage centred on the project’s Code of Construction Practice which included sections on community engagement, working hours, overarching management of environmental performance and control of noise and vibration.

8.5.18 In response to a number of petitions against the Bill, the CTRL prediction method for operational ground-borne noise and vibration was further developed and validated.

8.5.19 The revised method for tunnels predicted both ground-borne noise and vibration and was developed using additional data secured from speed trials during the testing and commissioning of the first German high speed line.

8.5.20 The procedure was subject to peer review by a number of leading railway noise and vibration specialists and was scrutinised by the London local authorities affected by CTRL and their specialist advisors. On the basis of the revised prediction method and two commitments by the promoter, the London local authorities agreed to withdraw their collective petition against the CTRL Bill.

8.5.21 The first of the Project’s commitments was for the predicted levels of ground-borne noise impact for the London tunnel to be not materially worse than the 530 ‘low’ impacts (between 35dB and 40dB \( L_{pA,max} \) \)[Table 8-1] and 100 ‘medium’ impacts (between 40dB and 45dB \( L_{pA,max} \) ) accepted by Parliament as being the likely adverse effects of the operation of HS1 London tunnels. The revised prediction method identified that no properties over the London tunnels were likely to experience vibration impacts based on the relevant British Standard in force at the time [Table 8-2]. The final forecasts identified that no ground-borne noise or vibration impacts were likely over the North Downs Tunnel.

8.5.22 The second commitment required the Project to employ Best Practicable Means (BPM) to reduce ground-borne noise levels below 40dB \( L_{pA,max} \) [Table 8-1]. Importantly, it was accepted by Parliament that complying with this commitment would not require changes to the alignment and would not require the use of floating slab track (FST). In the latter case, this was because FST is not proven under high-speed operation and would also have required a substantial increase in tunnel diameter. Given the need to construct the arrangement required by the safety authorities – the twin tunnel with a single track in each tunnel – the increased diameter would have been required for all 40km of tunnel, with a consequent enormous increase in capital cost.

**The delivery stage (1996 to 2003/2007)**

8.5.23 The Government procured, through open competition, a delivery partner to design, build and operate CTRL – London & Continental Railways. Union Railways Ltd became the nominated undertaker, employing through competition all the contractors to deliver the detailed design and construction of the project.

**Construction**

8.5.24 As forecast, the excavation of the tunnels generated perceptible ground-borne noise and vibration in overlying properties for a small number of days before and immediately after the excavation passed under each property. There was no damage caused to property by construction vibration and no calls to the 24/7 helpline where the contractors provided advance warning of the perceptible noise and vibration to residents and business occupiers.
CASE STUDY: Channel Tunnel Rail Link (CTRL) – Now HS1

Scheme: The Channel Tunnel Rail Link (CTRL) is the high-speed rail link between the Channel Tunnel and London. It is 108 km (68 miles) long and runs between St Pancras Station in London to the Channel Tunnel. CTRL carries international and domestic passenger services between 0530h and 2400h, and a growing number of high speed freight services during the day and night. The line was delivered in two sections. Section 1, with a line speed of 300km/h (188mph), runs mainly on the surface from the Channel Tunnel to the River Thames. Construction started in 1998 and operation commenced in September 2003. Section 2, with a line speed of 230km/h (144mph), runs from the River Thames to St Pancras and is primarily in tunnel. Construction started in 2001 and operation started in late 2007. CTRL was delivered on time and within budget. The project includes five ‘green tunnels’ and three major excavated tunnels.

London tunnels
- 18km, twin bore tunnel, excavated around 20m to 40m below ground level through sand, mixed sand & clay, and clay ground types.
- Excavated by tunnel boring machines launched from Stratford and Dagenham.
- Trains run at up to 230km/h on low vibration slab track that was specifically developed for CTRL and which sets a new benchmark for the level of ground-borne noise and vibration reduction for a high speed railway tunnel.
- The tunnels pass under or near 4,000+ homes, as well a number of offices and shops, but do not pass especially sensitive properties, such as hospitals or recording studios.
- No complaint about ground-borne noise or vibration has been recorded since the CTRL entered service.
- The Lands Tribunal has held that there was no injurious affection on the value of property over the tunnels due to noise or vibration.

Thames tunnels
- 3 km, twin bore tunnel under the River Thames (no noise or vibration sensitive receptors).

North Downs Tunnel
- 3km, single bore tunnel, excavated between 35m and 90m below ground level through chalk.
- Excavated in sections by NATM techniques with a shotcrete primary lining and a secondary lining of reinforced concrete.
- Trains run at up to 300km/h on ballast track, with softer rail pads provided for the shallower section of line to reduce ground-borne noise and vibration.
- The tunnel passes under or near several hundred homes, mostly where the tunnel is at its deepest.
- No complaint about ground-borne noise or vibration has been recorded since the CTRL entered service.
8.5.25 The ground-borne noise from the operation of the temporary construction railway, required to support the tunnel boring machines used to excavate the London Tunnels, resulted in a number of calls to the 24/7 helpline. Improved maintenance of the temporary track led to a reduction in ground-borne noise. More recent projects, particularly Crossrail, have developed improved temporary construction railways that avoid the temporary impacts observed during CTRL construction.

**Operation**

8.5.26 For London tunnels the commitments entered into were onerous for a high-speed rail project. Union Railways Limited understood that the ground-borne noise and vibration commitments for the tunnelled sections would require substantial incremental innovation based on the then current high-speed track design. Equally Parliament accepted that it may not be reasonably practicable to avoid low levels of ground-borne noise impact (ground-borne noise between 35dB and 39dB L_{pAmax}) in several areas.

8.5.27 Over a seven-year period, the project worked to identify the most appropriate track type and then develop detailed design to meet the acoustics requirements and to ensure a safe and reliable rail service.

8.5.28 All of the track system reviews identified that a ‘booted sleeper’ track was best placed to meet all of the project’s requirements. Figure 8.4 shows the construction of the booted sleeper system adopted. Two under sleeper pads and 12 lateral pads were installed at every sleeper to isolate the overlying buildings from the train generated vibration. The track system was optimised to provide the highest practicable performance.

8.5.29 For North Downs Tunnel, ballast track was identified as the optimum track solution with softer rail pads in the shallower section of the tunnel to reduce ground-borne noise and vibration.

8.5.30 The likely ground-borne noise and vibration impacts for the railway were reassessed using the final track system design. The final results showed that, for London tunnels, only 100 properties were predicted to experience levels of ground-borne noise impacts above 35dB L_{pAmax}, compared with the 630 properties identified at the time of the parliamentary process. Of these, only one property was predicted to experience medium impacts above 40dB L_{pAmax}, compared to the 100 likely medium impacts identified at the approval stage.

8.5.31 No likely ground-borne impacts were forecast for the North Downs Tunnel, although impacts have occurred at two unusual properties built into the Chalk escarpment directly over the shallowest section of the tunnel.
8.5.32 No ground-borne vibration impacts were forecast for the operation of the ‘as-built’ tunnels.

8.5.33 Consistent with the commitments, the project actively engaged with the local authorities and their technical advisors with an interest in the scheme throughout the design, testing and construction stages, always providing opportunity for check and challenge. The local authorities accepted the project’s ‘as-built’ scheme. There have been no recorded complaints about ground-borne noise or vibration since the railway came into service.

8.5.34 The upper and lower expected levels of vibration at different distances from the Tunnel Boring Machine (TBM) were predicted based on a number of previous studies. With soil properties measured on the site, a more accurate prediction equation was produced. It can be seen from Figure 8.5 that all the measured values lay within the initially predicted bounds. The prediction curve based on the soil properties compares well with measured values.

8.5.35 It should be noted that the PPVs measured during construction were not large enough to cause damage to buildings. The relatively high level of vibration was caused by the hard limestone being cut, which not only required a large cutting force, but was effective at transmitting vibration from the TBM to the houses. With softer ground, the vibrations will be lower.

8.5.36 This example shows that the vibration from a TBM can be predicted with some degree of accuracy, and the predictions can be refined with site-specific soil data.

8.5.37 The existing trackform of cast iron baseplates on timber sleepers was replaced by high resilience baseplates on concrete sleepers on a 120m long section of the Southbound Victoria Line in 2000. These baseplates provide a much softer vertical support to the rail compared with normal baseplates, whilst still providing the necessary support to prevent excessive lateral movement. The vibrations at the ground surface were compared to find the improvement in noise and vibration.

8.5.38 The vibration of the ground surface reduced by approximately 8dB after the installation of the resilient baseplates.

8.5.39 Whilst there is technology available to improve the performance of track in older tunnels this is unlikely to be needed for high speed track.
CASE STUDY: Control of ground-borne noise and vibration on Jubilee Line Extension using resilient rail support

Scheme: The Jubilee Line, an extension to Charing Cross of the former Stanmore branch of the Bakerloo Line, opened in 1979 using traditional rail support resulting in significant levels of vibration and ground-borne noise. The line was extended to Stratford from a new turnout south of Green Park in central London. It passes under Westminster, Southwark and east London in tunnel as far as Canning Town, and opened in 1999. The Jubilee Line extension was the first London Underground scheme to be subject to Environmental Impact Assessment, and the first deep tube system to be designed to targets for ground-borne noise and vibration which would result in no significant effects on occupiers of building above the tunnels.

Tunnel and track details: The tunnel is generally 4.35m diameter lined with concrete segments, with some cast iron sections, at a range of depths from 20m-30m. The trains are London Underground tube trains operating at up to 80 km/h. The standard track support consists of resilient rail baseplates designed to meet a target for ground-borne noise of not more than 40 dB LpASmax, found to be the threshold of complaints following the opening of the Jubilee and Victoria lines. There are some sections with floating slab track where the tunnels pass below buildings with highly sensitive uses or buildings with deep foundations close to the tunnels.

Monitoring: Following the opening of the system measurements of vibration and ground-borne noise showed that the design aims had been met.
CASE STUDY: Control of ground-borne noise and vibration on Crossrail using resilient rail support

Scheme: The Crossrail project includes twin tunnels from Royal Oak near Paddington to portals east of Liverpool Street, in Pudding Mill Lane and Victoria Dock, passing under the residential areas of Bayswater and Mayfair, under the West End and the City of London and under residential areas in east London. Following the precedent of the Jubilee Line Extension, the Environmental Statement included mitigation of vibration and ground-borne noise so as to avoid significant effects both for residential; buildings and many highly sensitive buildings such as the Barbican concert hall.

Tunnel and track details: The tunnel is designed for the operation of mainline electric multiple units at up to 100km/h and is generally 6.2m diameter lined with concrete segments, at a range of depths of up to 40m. The standard track support will be designed with resilient rail support endeavouring to meet a target for ground-borne noise in residential buildings of 35 dB LpASmax and with a limit for predicted levels not to exceed 40 dB LpASmax. There will be some sections with floating slab track where the tunnels pass below buildings with highly sensitive uses or buildings with deep foundations close to the tunnels. For the first time there are also controls on ground-borne noise from the temporary construction railway.

Monitoring: Monitoring of ground-borne noise and vibration from the operating railway will take place following the start of test running of trains in 2018.
CASE STUDY: Dublin Port Tunnel TBM construction vibration

Image credit: National Roads Authority and Dublin City Council

**Scheme:** The Dublin Port Tunnel is a 4.5km tunnel extending north from Dublin Port under the City linking up with the M1/M50 motorways and the rest of the national road network at Santry. Construction started in 2001 and the tunnel was opened to traffic in 2006.

**Tunnel details:** 2.6km of the Dublin Port Tunnel were bored through Carboniferous Limestone using an 11.8m diameter TBM. The tunnel passed some 20m below residential buildings.

**Noise and vibration from TBM:** The noise from the TBM was audible for about three weeks as it passed beneath affected properties. During this time the TBM travelled approximately 200m. At its peak, the noise reached a level of 55dB. Vibration was noticeable for about half the length of time and generally did not exceed 1.5mm/s, well below the level to cause any damage to buildings.
CASE STUDY: Vibration reduction on Victoria Line, high resilient rail fastening

Scheme: The rail fastenings on a 120m long section of the southbound Victoria Line between Oxford Circus and Green Park stations were replaced in 2000. The track had conventional cast-iron baseplates fixed to timber sleepers. It was replaced by highly resilient rail fastenings on concrete sleepers.

Tunnel details: The northbound and southbound tracks run in separate tunnels, around 25m apart and at 20m below ground level. All of the trains passed at approximately 50km/h.

Monitoring: The vibrations of the tunnel structure and the ground surface were measured before and after the track was replaced, as well as nearly a year later. The response from trains on both lines was measured, allowing a direct comparison between the original and replaced trackforms.
8.6  **Current practice for noise and vibration**

8.6.1 Where there are no buildings close enough to be affected, there is no benefit from controlling noise and vibration. Where buildings would experience only low levels of noise, less mitigation is needed to achieve satisfactory results.

8.6.2 It is necessary to identify the level of attenuation necessary to reduce noise levels to an acceptable level. The generally accepted noise levels depend on the use to which buildings are put – for example, there is little benefit in reducing ground-borne noise levels within a noisy factory, but it is essential to be very quiet in a recording studio.

8.6.3 Employing the highest levels of mitigation may produce more environmentally detrimental consequences. For example, if a larger tunnel diameter is required to install a better track, the tunnel volume will increase, leading to increased ground movement, more excavated material requiring disposal and a longer construction period.

8.6.4 Any mitigation measures must also be compatible with the need to provide a reliable, maintainable and safe railway.

8.7  **Conclusions**

8.7.1 Historically, some rail tunnels have been a source of annoyance from noise and vibration. In more recent years, these problems have been addressed and techniques to calculate the noise and vibration have been developed. Measurements have confirmed that these effects can be calculated with reasonable accuracy. The predictions are based on:

- existing knowledge of vibrations caused by trains travelling at a range of speeds;
- knowledge of the performance of buildings, depending on their construction type and height;
- measurement of the ground properties for each section of tunnel; and
- performance of different track types.

8.7.2 Not only are modern tracks constructed more accurately than previously, but monitoring regimes ensure that the rails and wheels are maintained to a higher standard. This ensures that the track produces less vibration than earlier railways.

8.7.3 A range of track forms can be employed to reduce the vibration passed from the rail to the tunnel structure. The track system will be designed and specified to reduce potential adverse impacts caused by ground-borne vibration within the requirements for a safe, reliable and maintainable railway.
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Appendix A: Effects of ground movement on buildings and structures
A.1 Introduction

A.1.1 This Appendix briefly outlines the asset protection process used in current practice for potential damage assessment on from underground construction projects. A risk-based, staged approach is applied in the identification and assessment of existing/proposed below and above third-party infrastructure affected by proposed underground works, including surface buildings, services/utilities, tunnels and highway structures. In the context of building damage assessments, “level of risk” is a commonly used term and is used to refer to the “possible degree of damage”.

A.2 Background

A.2.1 The development of a simple, objective system for classifying damage is a prerequisite for the successful assessment of the risk of damage to structures due to ground movements. It can also provide criteria upon which the design for movement can be based. In the UK, the development of an objective system of classifying damage is proving to be beneficial in creating realistic attitudes towards building damage, and also in providing logical and objective criteria for designing for movement in buildings and other structures.

A.2.2 Burland et al. (1977) concluded that there were essentially three categories when considering limiting movements in buildings, namely

a. aesthetics (visual appearance – finishes);

b. serviceability limit state (function – doors/windows); and

c. ultimate limit state (stability – structure integrity).

A.2.3 The magnitude of the foundation movements and the severity of the corresponding damage increase, in ascending order, for these categories. The detailed damage classification system summarised in Table A.9.1 was subsequently developed, comprising six categories of damage, numbered 0 to 5 in increasing order of severity. In this system, emphasis is placed on the ease of repair of the damage that is visible at the time of inspection. Crack width is given as an additional indicator rather than a direct measure of damage. Table A.9.1 also includes detailed descriptions of what is considered the ‘normal degree of severity’ associated with each category of damage. These descriptions of the normal degree of severity relate to standard domestic and office buildings (i.e. low-rise structures).

A.2.4 Under normal circumstances, Categories 0, 1 and 2 relate to aesthetic damage, Categories 3 and 4 relate to the serviceability limit state whilst category 5 represents the ultimate limit state. Where sensitive finishes are concerned this ranking may not be appropriate. It should be noted that most buildings experience a certain amount of cracking, often unrelated to foundation movement, which can be dealt with during routine maintenance. The division between Categories 2 and 3 is of particular importance in the potential damage assessment process for surface buildings; it represents the important threshold between aesthetic and serviceability damage. Damage up to Category 2 can result from a variety of causes, either from within the structure or associated with the ground; identification of the cause is usually very difficult and may result from a combination of effects. The cause of damage that exceeds Category 2 is usually much easier to identify and is frequently associated with ground movement.
A.2.5 Under normal circumstances aesthetic damage (i.e. damage to normal finishes) is considered acceptable given its ease of repair; it is more cost-effective to repair rather than prevent such damage occurring. Those structures identified as having the potential to sustain Category 2 (Slight) damage (or less) are normally excluded from further consideration; no protective measure provision is deemed necessary.

A.2.6 Foundation movements are not the only cause of distress in buildings; at the 1969 Concrete Society Conference on Design for Movement in Buildings many of the examples of building damage presented resulted from the movement of structural members rather than the foundations. Furthermore, a certain amount of cracking is unavoidable if the building design is to remain economic (Peck et al., 1956); it has also been said that it is not possible to eliminate cracking in buildings induced by shrinkage and creep, etc. Little (1969) described a particular type of building where the cost of preventing cracking was likely to exceed 10% of the total building cost.

<table>
<thead>
<tr>
<th>Category of damage</th>
<th>Normal degree of severity</th>
<th>Description of typical damage (ease of repair is underlined)</th>
<th>Crack width* (mm)</th>
<th>Limiting tensile strain (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>Negligible</td>
<td>Hairline cracks.</td>
<td>&lt; 0.1</td>
<td>0 to 0.05</td>
</tr>
<tr>
<td>1</td>
<td>Very slight</td>
<td>Fine cracks which are easily treated during normal decoration. Damage generally restricted to internal wall finishes. Close inspection may reveal some cracks in external brickwork or masonry.</td>
<td>0.1 to 1</td>
<td>0.05 to 0.075</td>
</tr>
<tr>
<td>2</td>
<td>Slight</td>
<td>Cracks easily filled. Re-decoration probably required. Recurrent cracks can be masked by suitable linings. Cracks may be visible externally and some repointing may be required to ensure weathertightness. Doors and windows may stick slightly.</td>
<td>1 to 5</td>
<td>0.075 to 0.15</td>
</tr>
<tr>
<td>3</td>
<td>Moderate</td>
<td>The cracks require some opening up and can be patched by a mason. Repointing of external brickwork and possibly a small amount of brickwork to be replaced. Doors and windows sticking. Service pipes may fracture. Weathertightness often impaired.</td>
<td>5 to 15 or a number of cracks greater than 3</td>
<td>0.15 to 0.3'</td>
</tr>
<tr>
<td>4</td>
<td>Severe</td>
<td>Extensive repair work involving breaking-out and replacing sections of wall, especially over doors and windows. Windows and door frames distorted, floor sloping noticeably. Walls leaning or bulging noticeably, some loss of bearing in beams. Service pipes disrupted.</td>
<td>15 to 25 but also depends on number of cracks</td>
<td>Greater than 0.3</td>
</tr>
<tr>
<td>5</td>
<td>Very severe</td>
<td>This requires a major repair job involving partial or complete rebuilding. Beams lose bearing, walls lean badly and require shoring. Windows broken with distortion. Danger of instability.</td>
<td>Usually greater than 25 but depends on number of cracks</td>
<td></td>
</tr>
</tbody>
</table>

Table A.9.1 | Building damage classification (developed for brickwork/blockwork/stone masonry buildings)

Notes
1. In assessing the degree of damage, account must be taken of its location in the building or structure.
2. Crack width is only one aspect of damage and should not be used on its own as a direct measure of it.
3. Boscardin and Cording (1989) describe the damage corresponding to the tensile strain in the range 0.15-0.3%, as ‘moderate to severe’. However, none of the cases quoted by them exhibit severe damage for this range of strains. Therefore there is no evidence to suggest that tensile strains up to 0.3% will result in severe damage.

Local deviation of slope, from the horizontal or vertical, of more than 1/100 will normally be clearly visible. Overall deviations in excess of 1/150 are undesirable.

A.3 Potential damage assessment

A.3.1 The widely accepted three-stage approach to potential damage assessment for buildings (Mair et al., 1996) with an increased level of rigour being applied at each stage of the assessment process is to be adopted on the HS2 project. The approach is illustrated graphically in Figure A.1.

A.3.2 The potential damage assessment process is intended to be conservative, such that those structures at risk of sustaining unacceptable damage can be identified and thereby allow more detailed study to be concentrated in problematic areas (Mair et al., 1996). The greenfield surface settlement contours determined as part of this process are not intended to serve as a prediction of the expected effects but should be used as a filter to identify infrastructure that is potentially at risk (Moss & Bowers, 2005).

Stage 1

A.3.3 Stage 1 of the process comprises the production of contours to identify, in the first instance, the number of structures within the zone of influence attributable to excavation-induced ground movement. This zone of influence is usually defined as the 1mm greenfield ground surface settlement contour. A greenfield assessment ignores any positive contribution made by existing structures, both surface and sub-surface, in mitigating the effect of excavation-induced ground movement. Structures out with the 1mm settlement contour are usually not considered further. Generalised criteria, for example a minimum settlement of 10mm or a slope of 1:500 (Rankin 1988), are then applied to eliminate structures from further consideration. Experience on recent tunnelling projects undertaken in the London area has shown that the effects on buildings of ground movements less than 10mm are not significant. However, the criteria should be applied with thought rather than on a purely mechanical basis; exceptions are usually made for Listed Buildings. The existing condition, presence of sensitive features and potential lines of weakness as well as long-term settlement effects can all combine to produce significant damage in structures, which would otherwise be eliminated from further consideration at Stage 1.

A.3.4 The calculations are simple and straightforward adopting the conventional empirical greenfield formulations for settlement estimation, and provide a useful method of identifying structures which may be affected by the potential movements that occur during construction. The empirical greenfield formulations are based on well-established and widely accepted methods determined from the back analysis of case histories of short-term volume loss movements (for example O’Reilly and New (1982), Attewell and Woodman (1982), and New and Bowers (1994). Ground movements due to shaft construction can generally be estimated using the approach proposed by New and Bowers (1994) with suitable allowance made for the anticipated ground conditions, shaft size and the construction methods likely to be employed (see Section 6).
Stage 1 – Preliminary Assessment
Estimation of greenfield ground surface settlement for wished-in-place case

Building/structure with 10mm greenfield ground surface settlement colour and/or subject to 1:500 (or greater) ground slope

No further assessment required. Building/structure subject to negligible risk of damage

Stage 2 – Individual Building Assessment
Deep beam analogy – maximum tensile strain

Potential damage classification
Categories of building damage 0, 1, 2

No further assessment required

Categories of building damage 3, 4, 5

Stage 3 – Detailed Evaluation

Phase 3 iterations:
Iteration 1 – Phase 2 model
Iteration 2 – Phase 2 model input parameters review
Iteration 3 – Excavation sequence modelled etc.

Iterate on potential damage classification

Potential damage classification
Categories of building damage 0, 1, 2

Protective measures not required. No further assessment required.

Categories of building damage 3, 4 & 5

No further iterations possible

Protective measures required

Table A.9.1 | Building damage classification (developed for brickwork/blockwork/stone masonry buildings)
**Stage 2**

**A.3.5** Stage 2 of the assessment process usually involves the consideration of a two-dimensional section through the structure concerned and the determination of the greenfield building strains likely to result from the underground works. Settlements are assessed by either empirical or analytical methods.

**A.3.6** The building is represented by an elastic deep beam and the greenfield ground movement imposed upon it; it is assumed that the structure behaves completely flexibly and that its stiffness has no influence on the greenfield settlement profile, see Figure A.2. This is a conservative assumption as in reality a building’s stiffness may modify the ground movement effects and limit the development of horizontal strain, thus reducing the potential for damage.

**A.3.7** An integral part of this stage of the assessment process is building damage classification. Boscardin and Cording (1989) demonstrated that the damage categories proposed by Burland et al (1977) could be broadly related to ranges of limiting tensile strain thus providing the link between estimated building deformation and the possible severity of damage. Burland (1995) produced an interaction diagram relating damage category to deflection ratio and horizontal tensile strain for an aspect ratio (L/H) of unity Figure A.3.

**A.3.8** Under normal circumstances if damage classification indicates that there is the potential for the building to sustain Category 2 (Slight) damage, or below, no further action is necessary and the assessment process for the particular building concluded. Aesthetic damage is considered acceptable given its ease of repair; it is more cost-effective to repair rather than prevent such minor damage occurring. Exceptions to this general rule can include:

A. the building has shallow foundations and is within a distance from a retained cutting, shaft or box equal to the excavated depth of superficial deposits, i.e. soils such as Made Ground, Alluvium or River Terrace Deposits
B. the building is listed by English Heritage as being of historical significance or heritage value
C. the building has a piled foundation
D. the building contains sensitive equipment
E. the building is in poor condition.
A.3.9 The deep beam analogy adopted during Stage 2 of the potential damage assessment process though considerably more detailed than the assessment carried out during Stage 1 still usually results in a conservative assessment. In the majority of cases the actual level of damage sustained by a building will be less than that assessed because in calculating the tensile strains the building is assumed to have no stiffness and behave perfectly flexibly. In practice the inherent stiffness of the building will be such that its foundations will interact with the supporting ground and tend to reduce both the deflection ratio and horizontal strains. However, the buildings structural form and any existing condition can influence the response of the building to excavation induced ground movement and the associated damage, for some structures strains will concentrate at movement joints, planes of weakness or existing cracks.

A.3.10 Parametric numerical studies, for example Franzius, Potts & Burland, (2006) and Potts & Addenbrooke, (1997), have been undertaken to produce design charts that take account of building stiffness; these charts can be used if appropriate, to produce a modified building damage assessment.

A.3.11 For buildings where there is the potential for Category 3 (Moderate) damage or worse to be sustained as a result of the excavation-induced ground movements, a Stage 3 assessment, possibly involving sophisticated numerical analysis techniques, will be required. Such an assessment will generally involve the development of a building specific model rather than the more generic model form used in Stage 2.

Stage 3

A.3.12 Stage 3 of the assessment process consists of several sub-steps or iterations, each refining the building and tunnel model to a higher degree of sophistication. During this stage both the strain developing within the building and the strain limit which is associated with damage Category 3 are re-assessed.

A.3.13 In Stage 3 each building is considered in detail in contrast to the first two stages. In the first iteration the same model is used as in the Stage 2 assessment. This model is then successively refined in the following iterations. Detailed consideration is given to the structural nature, current condition and fabric of the building, potential lines of weakness and sensitive features as well as potential soil-structure interaction effects. If necessary, the tunnel-excavation-soil-building interaction problem is modelled numerically. Assuming that a building conforms to 'greenfield' ground movements may often be overly conservative. Stiff buildings often experience much less differential settlement than flexible buildings and in such cases protective measures are not necessary (Mair, 2013); this should be taken into account in the Stage 3 analysis.

A.3.14 The division between damage Categories 2 and 3 remains the critical threshold but in this case it is not for determining whether or not to progress to the next stage of the assessment process, rather it is for resolving the need for, and nature of, any protective measures. Consideration of the ease of repair, i.e. the cost effectiveness and appropriateness of the mitigation measures, is a fundamental part of this process.

A.3.15 In terms of protective measures provision, the Category 2 (slight)/Category 3 (moderate) threshold is the governing criterion. Under normal circumstances the provision of protective measures is not generally envisaged where a damage Category of 2 or below is anticipated.

A.3.16 Similar staged approaches are taken for the assessment of the impact of excavation-induced ground movements on existing tunnels, highway structures and services/utilities.
A.4 Evaluation of impacts

A.4.1 A methodology is required for evaluating the overall impact on the Built Heritage. The methodology described above relates to the evaluation of the impact in terms of building damage classification alone. The methodology to be adopted to evaluate the overall impact also needs to incorporate the architectural and historic importance of the building, its finishes and robustness.

A.4.2 The knowledge of existing practice and professional judgement to assess the likely extent and significance of potential impacts should also be considered.

A.4.3 The baseline threshold is defined as whether the proposed works will ‘affect a building’s special (architectural and historic) interest’. This is the test defined by statute to determine whether listed building consent would be required.

A.4.4 This weighting has been reflected in the assessment of impacts. A low magnitude impact is defined as where the effect on the building’s special character would generally not be adverse. For moderate and high magnitude impacts, the broad criteria set out below (from Circular 07/09, PPS5 & NPPF) are applied using professional judgement to categorise the significance. These are defined as:

- The importance of the building
- The particular physical features of the building
- The setting of the building and its contribution to the local scene
- The extent to which the proposed works would bring substantial benefits for the community.

A.4.5 The assessment of the effects of ground movement is primarily influenced by the first two of the above criteria.

A.4.6 Table A.2 shows the evaluation matrix to be used to determine the significance of each impact.

<table>
<thead>
<tr>
<th>Level of Impact</th>
<th>Value of Resource</th>
<th>Value of Resource</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Grade II</td>
<td>Grade I and II*, SAMs</td>
</tr>
<tr>
<td>Low magnitude</td>
<td>NSig</td>
<td>NSig/Sig</td>
</tr>
<tr>
<td>Moderate magnitude</td>
<td>NSig/Sig</td>
<td>Sig/PSig</td>
</tr>
<tr>
<td>High magnitude</td>
<td>Sig/PSig</td>
<td>PSig</td>
</tr>
</tbody>
</table>

Key: NSig: non-significant impact; Sig: significant impact; PSig: impact of particular importance.

Table A.2 | Impact significance evaluation matrix

9.4.7 The same level of national importance has been given to Scheduled Ancient Monuments (SAMs) as Grade I and Grade II listed buildings (as shown in Table A.2).

Listed Buildings

A.4.8 For listed buildings the heritage value of the structure shall be considered in Stage 3 through a consideration of the sensitivity of the building to excavation induced ground movements and its ability to tolerate movement without significant distress. The potential for interaction with adjacent buildings should also be considered.
A.4.9 As part of this process the building can be allocated two scores in relation to its sensitivity to ground movement, both in accordance with the criteria set out in Table A.9. In order that the process of assessing the impacts can be applied to a wide variety of buildings and to record the process, a score is attached to each category. The scores can range from 0 for a robust building with no significant features up to 4 for delicate buildings with very fine finishes.

A.4.10 In assessing the sensitivity scores, due note will be taken of the grade of listing together with the special features of each building. In general Grade I, II* and SAMs will score at least one point more than a Grade II building and therefore will have a higher sensitivity. The scoring is to incorporate the professional judgement of specialists experienced in dealing with the construction and repair of historic buildings together with specialists in excavation-induced ground movements.

A.4.11 The first score is an indication of the overall nature of the building/structure and its ability to tolerate movement without significant distress. The second score relates to particular features within the building or structure and how they might respond to movement. These two scores are summed to provide an indication of the overall sensitivity of the building to ground movement. A score corresponding to the building damage category classification as outlined in Table A.3 is then allocated to the building.

A.4.12 The total score is then used to define the magnitude of the impact on the building as shown in Table A.3. Scores of less than 3 are assessed as a low magnitude impact. These buildings may suffer some fine cracking or minor deterioration of existing defects; this would need minor making-good once the works are complete. Where the total score is 3 or more, the impact is assessed as having either a moderate or high magnitude. In determining the overall score, corresponding impact and the need (or otherwise) for mitigation measure provision, as with the building sensitivity scoring, the professional judgement of specialists experienced in dealing with the construction and repair of historic buildings as well as that of specialists in the estimation of excavation-induced ground movements should be incorporated into the scoring system. The criteria should be applied with thought rather than on a purely mechanical basis. The existing condition, presence of sensitive features and potential lines of weakness can all combine to produce significant damage in structures.

<table>
<thead>
<tr>
<th>Score</th>
<th>Sensitivity of the structure to ground movements and interaction with adjacent buildings</th>
<th>Sensitivity to movement of particular features within the building</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>Masonry building with lime mortar not surrounded by other buildings. Uniform facades with no particular large openings.</td>
<td>No particular sensitive features</td>
</tr>
<tr>
<td>1</td>
<td>Buildings of delicate structural form or buildings sandwiched between modern framed buildings which are much stiffer, perhaps with one or more significant openings.</td>
<td>Brittle finishes, e.g. facias or tight-jointed stonework, which are susceptible to small movements and difficult to repair.</td>
</tr>
<tr>
<td>2</td>
<td>Buildings which, by their structural form, will tend to concentrate all their movements in one location.</td>
<td>Finishes which if damaged will have a significant effect on the heritage of the building, e.g. cracks through frescos.</td>
</tr>
</tbody>
</table>

Table A.3 | Listed buildings sensitivity assessment scoring
Source: LU Ground Movement Guidelines LUL Civil Engineering – Common Requirements
### A.5 Case history: The Mansion House

#### A.5.1

The limiting tensile strain approach (after Burland & Wroth, 1975) assumes that the building is fully flexible and follows the greenfield foundation level settlement profile. This is, in general, a conservative assumption as it is often the case that the stiffness of the building modifies the greenfield settlement profile. Indeed, measurements made during tunnelling works below The Mansion House (Frischmann et al., 1994) indicate that the presence of the building above the tunnel significantly modified the tunnelling-induced settlement profile from the anticipated greenfield ground movement profile, broadening the settlement trough, reducing the maximum settlement and, most importantly, limiting the corresponding differential movement (see Figure A.4 below).

![Figure A.4 | Comparison of actual and predicted settlements along western facade of The Mansion House due to construction of the Waterloo and City Line Link Tunnel (after Frischmann et al., 1994)](image-url)
A.6 Case history: the Jubilee Line Extension

A.6.1 The JLE project is described in section 5.4.

A.6.2 Many buildings across London were monitored as part of the JLE project. The results of this monitoring and the corresponding interpretation of the responses of the buildings are published in CIRIA SP200. A selection of these case histories is presented in Table A.5.

A.6.3 One measure that was widely implemented to prevent the effects of ground movement on sensitive buildings was compensation grouting. This process was used to support and limit the movements and tilting of St Stephen’s (Big Ben) clock tower (now the Elizabeth Tower) due to tunnel construction and the excavation of the main station box during the JLE works at Westminster. This method was also implemented successfully on other structures across the capital during the JLE works, hence reducing the impacts of the works on third parties.

<table>
<thead>
<tr>
<th>Building Details</th>
<th>JLE Infrastructure</th>
<th>Predicted ground settlement &amp; potential damage category</th>
<th>Mitigation</th>
<th>Actual ground movement</th>
<th>Result</th>
</tr>
</thead>
<tbody>
<tr>
<td>The Treasury, Westminster. Ornate classical style building of Portland stone ashlar</td>
<td>Two running tunnels beneath the south west corner of the building – Open Face TBM construction</td>
<td>Maximum 32mm (2% Volume loss, K=0.5) Damage category – slight/moderate.</td>
<td>Compensation grouting after the construction of the first tunnel to limit.</td>
<td>24.5mm after the first running tunnel. 28mm 5 years after completion of construction.</td>
<td>Negligible damage.</td>
</tr>
<tr>
<td>Elizabeth House, Waterloo. 7 &amp; 10 storey reinforced-concrete frame structure</td>
<td>Situated above two running tunnels and a crossover passage connecting them – SCL construction</td>
<td>Maximum 53mm (1.3% volume loss, k= 0.45). Damage category – very slight/slight (greenfield)</td>
<td>None. Detailed analysis showed building would behave in a rigid manner.</td>
<td>48mm at 3.5 years after completion of construction.</td>
<td>Negligible damage. Building behaved in a rigid manner as Detailed analysis suggested. Small increases in existing crack widths and expansion joints.</td>
</tr>
<tr>
<td>Blick House, Rotherhithe. Two blocks of flats on 5 storeys. Constructed of load bearing brickwork with reinforced concrete floors.</td>
<td>Two running tunnels spaced at approximately 25m centre to centre.</td>
<td>Maximum 22mm (2% Volume loss, K=0.5)-24mm (2.3% Volume loss, K=0.43). Damage category – slight</td>
<td>None.</td>
<td>10mm maximum with negligible long term settlements.</td>
<td>No damage occurred as part of the construction works. Building response was almost completely flexible.</td>
</tr>
<tr>
<td>Murdoch House, Moodkee Street. Load bearing masonry.</td>
<td>EPB TBM Running Tunnels passing beneath building</td>
<td>Maximum 8mm (0.75% Volume loss, K=0.45)-Negligible</td>
<td>None</td>
<td>8mm</td>
<td>Negligible</td>
</tr>
<tr>
<td>Neptune House, Moodkee Street. Load bearing masonry.</td>
<td>EPB TBM Running Tunnels passing beneath building</td>
<td>Maximum 7mm (0.75% Volume loss, K=0.45)-</td>
<td>None</td>
<td>6mm</td>
<td>Negligible</td>
</tr>
<tr>
<td>Clegg House, Moodkee Street. Load bearing masonry.</td>
<td>EPB TBM Running Tunnels passing beneath building</td>
<td>Maximum 8mm (0.75% Volume loss, K=0.45)-Negligible</td>
<td>None</td>
<td>8mm</td>
<td>Negligible</td>
</tr>
</tbody>
</table>

Table A.5 | Selected case studies from the Jubilee Line Extension (Viggiani & Standing, 2001) (Standing, 2001) (Withers, 2001a) (Withers, 2001b)
A.7 Mitigation measures

A.7.1 The design of tunnels and underground spaces includes consideration of operational requirements, construction practicalities and optimisation of the design of the temporary and permanent works to minimise the ground movements at source and thus the impact of ground movements on a particular building or facility as far as is reasonably practicable. The design process integrates these elements and balances the requirements. A comprehensive ground investigation to establish a good understanding of the ground and groundwater conditions is required at the design stage.

A.7.2 In tunnel construction selection of an appropriate excavation technique and lining method is important in minimising ground movements, for example the incorporation of a pilot tunnel in the construction sequence. In shaft construction, the methods employed to support the ground as excavation progresses will influence the magnitude of the adjacent ground movements.

A.7.3 For the vast majority of buildings within the zone of influence of the tunnel construction, no mitigation is required to reduce risk of damage to an insignificant level, provided current practice is followed. However, for those that do require mitigation, there are various measures which can be applied in this limited number of cases to reduce the impact of excavation-induced ground movements on buildings and other infrastructure, including the following which are set out in order of consideration:

A.7.4 At-source Measures – are techniques designed to reduce the magnitude of excavation-induced ground movements at source. In general, these comprise the installation of additional and/or stiffer support to the ground at the earliest practicable point within the excavation sequence for example forepoling or soil nailing in the tunnel face.

A.7.5 Strengthening of the ground to provide a layer of increased stiffness below the building foundation level or to prevent volume loss at the tunnel face during excavation. This may be achieved by a variety of means which include:

- **Permeation grouting** – the injection of cement grout into Gravels or sands to bind the soil together and thus redistribute loads and resist local deformations; there would also be a decrease in permeability of the ground. The grout is normally injected through pipes (Tube à Manchettes) from the ground surface or from basements/shafts.

- **Ground freezing** – the temporary artificial freezing of the ground to stabilise it, through the provision of structural support to and/or exclusion of groundwater from an excavation, thus enabling construction.

- **Jet grouting** – the controlled partial replacement of the ground with grout. After treatment the ground is stronger, stiffer and less permeable.
A.7.6 **Strengthening of the building** – to sustain the additional stresses or accommodate the corresponding deformations induced by excavation-induced ground movements without significant distress buildings can be strengthened. These strengthening works may include the use of tie rods, bracing and temporary/permanent propping. However, such work can be very intrusive and may result in greater impacts on the building than simply allowing some cracking to occur which can be repaired subsequently. Consequently these works should be carried out in a manner which does not interfere with or damage any historic features or aspects of a building.

A.7.7 **Underpinning** – the introduction of an alternative foundation system to eliminate or minimise differential movements induced by excavation. If the existing foundations are inadequate or in a poor condition, underpinning may be used to strengthen them and provide a more robust and stiffer support system.

A.7.8 **Mechanical jacking** – this is suitable for new builds and developments only where specific provision has been allowed for this. The insertion of hydraulic or screw jacks at appropriate locations within the structure so that, as the foundations move down as a result of excavation-induced ground movement, the level of the structure above can be adjusted and maintained. The jacks are removed on completion of the works and the integrity of the structure reinstated. This measure may also be used where a building houses sensitive equipment.

A.7.9 **Installation of a physical barrier** – the installation of, for example a slurry trench wall or a row of secant bored piles, between the building foundation and the source of the excavation-induced ground movements. Such a barrier is not structurally connected to the building’s foundation and therefore does not provide direct load transfer. The intention is to modify the shape of the settlement trough and reduce ground displacements adjacent to and beneath the building. It is not effective directly above the tunnel, but only at a specific distance from the tunnel alignment.

A.7.10 **Compensation grouting** – the controlled injection of cement grout into the soil between the proposed tunnel and the affected building in response to observations of ground and building movements during tunnelling. As the name implies the technique is intended to ‘compensate’ for the volume loss. Grout is initially injected prior to tunnel excavation and then in stages as excavation proceeds. This technique is suited to the clay soils underlying most buildings in Central London; it has also been successfully applied to sands and Gravels. Shafts usually have to be constructed to enable the grout to be injected at the appropriate level in the soil. The shaft construction may add to ground movement in some areas of the site. Detailed instrumentation and monitoring is required as part of the process.

Burland (2001) commented that compensation grouting is a very expensive protective measure and that careful consideration should be given to its use. In addition, he noted that the actual level of damage sustained by many of the buildings along the route of the Jubilee Line Extension was less than that anticipated on the basis of the results of the staged assessment process. Burland concluded that compensation grouting was thus not strictly required to the buildings overlying the running tunnel alignments. He also noted that there was little doubt that compensation grouting was necessary above the major underground stations at Westminster, Waterloo and London Bridge but that it was likely that lower volumes of grout could have been used.
9.7.11 **Monitoring** – Monitoring does not mitigate the effects of settlement, but it can be used as appropriate to check that the magnitudes of the anticipated movements are not being exceeded. It can also be used where needed to determine whether reactive mitigation works need to be implemented.

9.7.12 **Making good** – If some minor cracking does occur with residual impacts which are all assessed as Not Significant, appropriate conservation techniques can be employed to effect sympathetic repairs to historic fabric.

**9.8 Summary**

9.8.1 The process for assessing the impact of excavation-induced ground movements on existing assets has been summarised. The process is simple, and straightforward as well as robust resulting in conservative assessments of potential damage and the need for protective measures. The process is also widely recognised throughout the rest of the world as a suitable approach to excavation-induced potential damage assessment.
Appendix B: Ground movements due to shaft construction
B.1 Categories of ground movement for shaft excavations

B.1.1 The ground movement arising from open excavations such as shafts and boxes, is typically due to the following effects:

- Wall installation;
- Movement of ground behind the wall due to deflection of the support wall and heave in the base;
- Groundwater changes outside the excavation; and
- Movements around the unsupported face for caisson or SCL advanced shafts.

B.2 Factors affecting ground movement

B.2.1 Any excavation in the ground will cause some localised ground movement and shafts are no exception to this. Below are the key factors that affect the magnitude of shaft ground movements:

B.2.2 Depth of excavation – deeper excavations will tend to cause a larger magnitude of settlement adjacent to the shaft as well as a wider settlement trough, affecting a greater area. Deeper shafts result in greater unloading and some heave may occur due to this unloading.

B.2.3 Diameter of shaft – larger diameter shafts may be expected to produce larger ground movements than small diameter shafts. This is a consequence of lower stiffness of wall and ground in the horizontal plane.

B.2.4 Ground properties – the ground properties govern the profile of settlement troughs, the amount of consolidation expected as well as the construction method of the shaft.

B.2.5 Coarse grained materials typically have narrow but deeper settlement troughs. Fine grained materials tend to have wider shallower settlement troughs. The stiffness of ground resists radial movement and heave in the base of the shaft. Ground movement in stiff materials such as rocks is much less than in soils.

B.2.6 Coarse grained materials, when water bearing, will require dewatering of the ground prior to shaft construction. This process can cause ground movements prior to any construction or excavation.

B.2.7 Wall properties – the properties of the walls of the shaft and any supporting structures will affect the settlement trough surrounding the shaft. The stiffer the wall and supports, the smaller the deflections and hence the smaller the ground movements as a result.

B.2.8 Walls and their supports can be increased in stiffness to reduce the risk of settlement as part of the design process.
B.2.9  **Construction methods** – the construction method used for shaft construction or sinking is based on a number of factors including geology, ground and groundwater conditions, depth, diameter, construction site size. They include the following:

- Formation of walls using diaphragm walls;
- Secant piling or contiguous piling;
- Sheet piles lined shafts;
- Wet or dry caisson sinking;
- Underpinning using pre-cast concrete segments; or
- Sprayed concrete lining.

B.2.10 In general diaphragm wall shafts or other types where a stiff structure is put in place before excavation cause less ground movements than those where excavation occurs at the base of the lining. This is provided good workmanship is in place. Top down construction where secondary supports are installed as excavations progress produce stiffer structures and also reduce settlement. Movement due to sinking of wet caissons is typically less than dry caissons as the groundwater effects are minimised. Shaft construction by underpinning produces the greater movements, although movements in SCL shafts can be controlled by careful sequencing of excavation.

B.2.11  **Drainage of groundwater** – during excavation and after completion shafts commonly act as a drain and will cause lowering of the water table. This is likely to result in changes to pore water pressures and consolidation settlement as discussed in section 5.1.15.
### B.3 Assesment of Ground Movements

#### B.3.1

New and Bowers, 1994, proposed the following parabolic equation for predicting ground movements associated with shaft construction, based on monitoring of a 26m deep, 11m diameter, caisson driven shaft in London Clay at Heathrow:

\[ S_d = \frac{(H-d)^2}{H} \]

#### B.3.2

A development of this equations that is used for analysis in the HS2 project which incorporates a dependence on shaft diameter is:

\[ S_d = \frac{\alpha R}{R} \times \frac{(RH-d)^2}{RH} \]

#### B.3.3

Where:

- **S_d** = Settlement at a given distance from the shaft edge (m)
- **d** = distance from the shaft edge (m)
- **H** = depth of the shaft (m)
- **D** = diameter of the shaft (m)
- **\( \alpha \)** = empirical constant dependent on the shaft diameter, ground conditions and construction method, representing the maximum ground movement over the shaft excavation depth (0.0006 for a 26m deep, 11m diameter shaft in London Clay New and Bowers, 1994).
- **R** = the ratio of settlement extent to effective shaft excavation depth

#### B.3.4

Values of \( R \) and \( \alpha \) are dependent on the shaft diameter. The smaller the diameter the stiffer the shaft acts, working hoop stress through the shaft walls. This results in smaller ground movements.
Appendix C: Ground movement due to open cut excavations
C.1 Embedded retaining walls

C.1.1 The use of embedded retaining walls is widely implemented as temporary and/or permanent support of vertical open cut excavations in urban environments. An embedded retaining wall is a wall that penetrates the ground at the excavation base in order to get lateral support from the ground. In addition it can also be supported by structural members such as props, ground anchors and slabs. The most common types of embedded retaining walls used in the UK are presented in table C.1 below.

<table>
<thead>
<tr>
<th>Wall Type</th>
<th>Advantages</th>
<th>Limitations</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sheet pile wall</td>
<td>Economic, predictable surface finish, no spoil – arisings, water retaining, both temporary and permanent use</td>
<td>Maximum length 30m, potential declutching in coarse grained soils, vibration and noise induced by driving</td>
</tr>
<tr>
<td>Combi wall</td>
<td>High capacity</td>
<td>Complex installation</td>
</tr>
<tr>
<td>King post wall</td>
<td>Can be installed around obstructions and at isolated points</td>
<td>Not suitable for long term retaining of water, or excavation below groundwater table in coarse grained soils</td>
</tr>
<tr>
<td>Contiguous pile wall</td>
<td>Cheapest form of concrete pile wall</td>
<td>Not water retaining, not permanent solution unless structural face applied, verticality tolerances to be considered</td>
</tr>
<tr>
<td>Secant pile wall</td>
<td>Permanent water retaining wall (except hard/soft), can achieve high strength, suitable for complex plan shapes</td>
<td>Depth limited by verticality tolerances, more complex construction than contiguous, walers required for temporary props, leakage risk esp. at depth</td>
</tr>
<tr>
<td>Diaphragm/Barrette wall</td>
<td>Permanent water retaining wall, great depths possible, fewer joints than secant pile wall, can be used as permanent wall thus reducing the total width of the structure, simpler temp works (walers avoided)</td>
<td>Large construction site requirements, costly disposal of support fluid, not appropriate for complicated plan outlines</td>
</tr>
</tbody>
</table>

Table C.1 | Common types of embedded walls used in the UK

C.1.2 Depending on the site conditions, the choice of construction of an open cut will be one of the three methods described below:

- **Cantilever wall** (i.e. with no additional support) where full excavation will take place from the ground surface followed by construction of the permanent structure; for shallower excavations, eg. tunnel portal approaches, the cantilever wall may form both the temporary and permanent works;

- **Propped wall and top down sequence** where excavation is done in stages and the permanent internal structure is constructed from the top to the bottom and it is therefore used as propping for the retaining wall; and

- **Propped wall and bottom-up sequence** where excavation is done in stages with a temporary prop (or anchor) put in place after each excavation stage is complete and the permanent works then constructed from the lowest level upwards.
C.2 Sources of ground movement

C.2.1 Ground movement in an open cut occurs mainly due to:

- **Construction of the wall**
  From vibration due to driving of piles or loss of ground support due to boring or excavation for panel construction.

- **Excavation in front of the wall**
  Active pressures will act on the wall forcing the wall to move towards the excavation causing the ground behind the wall to settle and heave will occur at the base of the excavation due to unloading.

- **Changes in pore water pressures**
  A drop or rise of the water table level will cause changes in pore water pressures causing the ground to either settle or heave. The type and magnitude of movements which may develop are sensitive to the specific ground conditions. The types of movements that may occur due to water flow are shown in Figure C.1. It should be noted that a well designed and constructed retaining wall will avoid many of the issues highlighted in Figure C.2, such as water flow through defects in the wall. Monitoring during the works construction can identify local problems and facilitate timely repairs, prior to damaging ground movements developing.

C.2.2 Ground movements will not only occur during construction but will continue to take place in the long term. In open cut excavations in low permeability clays such as the stiff sedimentary clays found in the UK, the swelling at the excavation base is likely to extend over many years following completion of construction. However, as discussed later, long term movements beyond the rear of the embedded retaining wall tend to be small and of little practical significance.

![Figure C.1 | Typical ground movement pattern in open cuts (CIRIA C580)](image-url)
C.3 Factors affecting ground movements

C.3.1 The factors that affect the ground movement and will therefore be considered in choosing the appropriate retaining system are the following:

- **Geometry of the excavation area and the permanent structure**
  The extent and level of observed ground movement beyond the excavation face generally increases with depth of retained soil. The ratio of length to width and to depth of the excavation area (aspect ratio) may also affect the extent of ground movement.

- **Ground conditions**
  In the UK there is extensive experience of constructing open cuts in stiff plastic clays (such as London Clay and Gault Clay). Past experience indicates that the retaining walls will perform well with regards to both short term and long term movements. The anticipated problems when excavating in granular soils will be mainly related to controlling ground movements during installation (due to vibration especially if the granular soils are loose). The most challenging ground conditions tend to be when deep deposits of soft clays and peat are encountered. However, these ground conditions are relatively rare in the UK. Nevertheless deep excavations can be successfully built in soft clays; however specialist techniques are often required to minimise global ground movements.

The performance of open cuts in other ground conditions such as weak rocks (e.g. mercia Mudstone, coal measures, Chalk) and glacial deposits (e.g. lodgement tills) is normally better than in stiff plastic clays and smaller wall and ground movements have generally been observed.
• **Groundwater conditions**
  In areas with high water table drainage and waterproofing will usually be required. The drainage design will aim to minimise changes in the groundwater regime. This is particularly important if the retained soils comprise of soft clays/peat, in order to minimise the risk of long term differential settlement.

• **Surrounding environment**
  Proximity to existing buildings/structures railway lines, underground structures/services or limited construction site space for equipment deposition will increase the surcharge loading on the back of the wall. The allowable movement of these structures will usually be the main design and construction constraint and will largely dictate the design and construction methodology. However this is a routine aspect of modern retaining wall design.

• **Construction sequence**
  If the wall acts as a cantilever during the initial construction stages then the wall deflections will be relatively large. Hence in urban areas it is normal practice to prop the wall as soon as practically possible in order to minimise wall and ground movements.

• **Workmanship**
  Delays in construction or placement of props and over excavation will cause increases in lateral movements. Hence modern construction practice involves the development of detailed method statements of the construction process, together with ground movement monitoring in order to rigorously control these activities.

**C.4 Structure performance**

**c.4.1** In order to estimate ground movement empirical approaches or analytical methods can be used. Empirical correlations are based on case history field measurements which are widely published. Analytical methods are based on numerical models that can be calibrated against comparable experience.

**c.4.2** Various different forms of failure mechanisms that may develop as the ground moves are considered. These are reproduced from CIRIA Report C580 and are shown in Figure C.3. These form the basis for a comprehensive range of checks which are routinely implemented and are documented in current design codes.

![Figure C.3 Examples of failure mechanisms (after CIRIA C580)](image)
Appendix D: Tunnelling projects in the UK post 1980
<table>
<thead>
<tr>
<th>Project name</th>
<th>County</th>
<th>Geology</th>
<th>Construction period</th>
<th>Excavated diameter or maximum dimension (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Crossrail – running tunnels</td>
<td>London</td>
<td>London Clay, Chalk</td>
<td>2012 – present</td>
<td>7.1</td>
</tr>
<tr>
<td>Lee tunnel</td>
<td>London</td>
<td>Chalk</td>
<td>2011 – present</td>
<td>8.8</td>
</tr>
<tr>
<td>London power tunnels</td>
<td>London</td>
<td>London Clay, Lambeth group</td>
<td>2011 – present</td>
<td>4.75</td>
</tr>
<tr>
<td>Airdrie and Coatbridge transfer sewer</td>
<td>Lanarkshire, Scotland</td>
<td>Old Carboniferous series containing Coal Measures</td>
<td>2011-2012</td>
<td>3.05</td>
</tr>
<tr>
<td>A3 Hindhead</td>
<td>Surrey</td>
<td>Sandstone</td>
<td>2007 – 2011</td>
<td>11.6</td>
</tr>
<tr>
<td>Lower Lea Valley cable tunnels</td>
<td>London</td>
<td>London Clay, Lambeth group, Thanet sands</td>
<td>2006 – 2009</td>
<td>4.0</td>
</tr>
<tr>
<td>Dartford cable tunnel</td>
<td>Essex/Kent</td>
<td>Upper Chalk with flints</td>
<td>2003 – 2004</td>
<td>3.5</td>
</tr>
<tr>
<td>High Wycombe</td>
<td>Bucks</td>
<td>Gravels overlying Chalk, Gravels and clays in Thames Flood Plain</td>
<td>2002 – 2004</td>
<td>3.2</td>
</tr>
<tr>
<td>Channel Tunnel rail link</td>
<td>London</td>
<td>London Clay, Woolwich &amp; Reading beds (Lambeth Group), Thanet sands, Harwich formation, Bullhead beds, lower Chalk and Upnor formation</td>
<td>2001 – 2004</td>
<td>8.11</td>
</tr>
<tr>
<td>City of London</td>
<td>London</td>
<td>London Clay</td>
<td>1999 – 2000</td>
<td>2.9</td>
</tr>
<tr>
<td>Channel Tunnel rail link</td>
<td>Kent</td>
<td>Upper and middle Chalk</td>
<td>1998 – 2001</td>
<td>14</td>
</tr>
<tr>
<td>Cardiff – Gabalfa</td>
<td>Cardiff</td>
<td>Alluvium, mercia Mudstone with Sandstone and Siltstone</td>
<td>1998 – 2000</td>
<td>1.75</td>
</tr>
<tr>
<td>Nunhead to Deptford water main</td>
<td>London</td>
<td>London Clay and Woolwich and Reading Beds (Lambeth Group)</td>
<td>1998 – 2000</td>
<td>1.4</td>
</tr>
<tr>
<td>Birmingham – Perry Hill to Gravelly</td>
<td>Birmingham</td>
<td>Mixed clay, sand and gravels</td>
<td>1998 – 2000</td>
<td>3.4</td>
</tr>
<tr>
<td>Cardiff – Rhymney Valley</td>
<td>Cardiff</td>
<td>Sand/gravel to Siltstone/Mudstone</td>
<td>1997 – 2000</td>
<td>2.1</td>
</tr>
<tr>
<td>Folkestone – Interceptor</td>
<td>Kent</td>
<td>Gault Clay overlying Folkestone Beds overlying Sandgate Beds</td>
<td>1997 – 1999</td>
<td>3.2</td>
</tr>
<tr>
<td>Newport – wastewater tunnel</td>
<td>Monmouthshire</td>
<td>Mercia Mudstone, St Maughan Sandstone</td>
<td>1997 – 1999</td>
<td>3.3</td>
</tr>
<tr>
<td>Cardiff – East interceptor</td>
<td>Cardiff</td>
<td>Alluvium overlying Mercia Mudstone, overlying mixed Sandstone, Silurian Siltstone and Mercia Mudstone and overlying Mercia Mudstone</td>
<td>1997 – 1999</td>
<td>2.8</td>
</tr>
<tr>
<td>West Ham to North Greenwich cable tunnel</td>
<td>London</td>
<td>London Clay and Woolwich and Reading Beds (Lambeth Group)</td>
<td>1997 – 1998</td>
<td>2.9</td>
</tr>
<tr>
<td>DLR extension to Lewisham</td>
<td>London</td>
<td>Woolwich and Reading beds (Lambeth group) and Thanet sands</td>
<td>1996 – 1998</td>
<td>5.9</td>
</tr>
<tr>
<td>Brixham – stormwater retention tunnel</td>
<td>Cornwall</td>
<td>N/A</td>
<td>1996 – 1997</td>
<td>3.1</td>
</tr>
<tr>
<td>Falmouth storage tank</td>
<td>Cornwall</td>
<td>Siltstone/Mudstone with clay seams</td>
<td>1996 – 1997</td>
<td>3.1</td>
</tr>
<tr>
<td>Project name</td>
<td>County</td>
<td>Geology</td>
<td>Construction period</td>
<td>Excavated diameter or maximum dimension (m)</td>
</tr>
<tr>
<td>-------------------------------------------------------</td>
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<td>-------------------------------------------------------------------------</td>
<td>---------------------</td>
<td>--------------------------------------------</td>
</tr>
<tr>
<td>St Johns Wood to Pakenham Street cable tunnel</td>
<td>London</td>
<td>London Clay</td>
<td>1996</td>
<td>2.9</td>
</tr>
<tr>
<td>Jersey – St Helier, surface water link and storage cavern</td>
<td>Jersey</td>
<td>Fort regent granite</td>
<td>1994 – 1997</td>
<td>18</td>
</tr>
<tr>
<td>Fylde coastal water improvement</td>
<td>Lancashire</td>
<td>Upper boulder clay, overlying middle sands and lower boulder clay</td>
<td>1994 – 1996</td>
<td>3.4</td>
</tr>
<tr>
<td>Sandwich Bay – Ramsgate and Deal outfalls</td>
<td>Kent</td>
<td>Chalk</td>
<td>1993 – 1995</td>
<td>1.5</td>
</tr>
<tr>
<td>Mansfield – outfall</td>
<td>Nottinghamshire</td>
<td>Middle permian marl and lower magnesium limestone</td>
<td>1993 – 1995</td>
<td>4</td>
</tr>
<tr>
<td>Barking Reach power station</td>
<td>London</td>
<td>Chalk</td>
<td>1993 – 1994</td>
<td>3.9</td>
</tr>
<tr>
<td>Sheffield – Don Valley Stage 5A</td>
<td>Yorkshire</td>
<td>Coal measures</td>
<td>1992 – 1999</td>
<td>3.3</td>
</tr>
<tr>
<td>Bristol – Northern foul water interceptor sewer – Frome Valley</td>
<td>Avon</td>
<td>Keuper marl (mercia Mudstone)</td>
<td>1991 – 1993</td>
<td>2.16</td>
</tr>
<tr>
<td>Project name</td>
<td>County</td>
<td>Geology</td>
<td>Construction period</td>
<td>Excavated diameter or maximum dimension (m)</td>
</tr>
<tr>
<td>--------------</td>
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<td>--------------------------------------------------------------------------</td>
<td>---------------------</td>
<td>--------------------------------------------</td>
</tr>
<tr>
<td>Burgh Heath intercepter</td>
<td>Surrey</td>
<td>Clays, sand and Chalk</td>
<td>1990 – 1991</td>
<td>1.9</td>
</tr>
<tr>
<td>Newton Abbott – Aller Valley</td>
<td>Devon</td>
<td>Silty sands and Gravels with boulders</td>
<td>1990 – 1991</td>
<td>1.5</td>
</tr>
<tr>
<td>London ring main 1B</td>
<td>London</td>
<td>London Clay &amp; Woolwich &amp; Readingbeds (Lambeth group)</td>
<td>1990</td>
<td>3.34</td>
</tr>
<tr>
<td>Sheffield – Don Valley Phase 4</td>
<td>Yorkshire</td>
<td>Coal measures</td>
<td>1989 – 1991</td>
<td>3.47</td>
</tr>
<tr>
<td>London ring main 1A</td>
<td>London</td>
<td>London cay, Woolwich &amp; Reading beds (Lambeth group) and Thanet sands</td>
<td>1988 – 1991</td>
<td>2.82</td>
</tr>
<tr>
<td>Stanstead Airport rail link</td>
<td>Essex</td>
<td>Boulder clay with lenses overlying London Clay with the Kesgrave beds of sands and Gravels between under artesian pressure</td>
<td>1988 – 1990</td>
<td>6.2</td>
</tr>
<tr>
<td>Sizewell B power station</td>
<td>Suffolk</td>
<td>Loose sand and gravel beach deposits</td>
<td>1988 – 1990</td>
<td>5.5</td>
</tr>
<tr>
<td>London ring main 2A</td>
<td>London</td>
<td>London Clay &amp; Woolwich &amp; Reading beds (Lambeth group)</td>
<td>1988 – 1990</td>
<td>2.82</td>
</tr>
<tr>
<td>Brighton mariner development</td>
<td>East Sussex</td>
<td>Lower Chalk</td>
<td>1988 – 1989</td>
<td>1.2</td>
</tr>
<tr>
<td>London ring main</td>
<td>London</td>
<td>London Clay and Woolwich &amp; Reading beds (Lambeth group)</td>
<td>1987 – 1989</td>
<td>2.88</td>
</tr>
<tr>
<td>Conway – immersed tube tunnel</td>
<td>Clywd</td>
<td>N/A</td>
<td>1986 – 1990</td>
<td>N/A</td>
</tr>
<tr>
<td>Sheffield – Don Valley Stage 3</td>
<td>Yorkshire</td>
<td>Coal measures</td>
<td>1986 – 1988</td>
<td>4.7</td>
</tr>
<tr>
<td>Plymouth – Saltash pilot tunnel</td>
<td>Cornwall</td>
<td>Slate</td>
<td>1986</td>
<td>3.6</td>
</tr>
<tr>
<td>Margate – relief sewer</td>
<td>Kent</td>
<td>NACChalk</td>
<td>1985 – 1986</td>
<td>2.5</td>
</tr>
<tr>
<td>Bury – North/South interceptor</td>
<td>Lancashire</td>
<td>Mudstone, shale and Sandstone</td>
<td>1984 – 1985</td>
<td>2</td>
</tr>
<tr>
<td>Sheffield – Don Valley Stage 2</td>
<td>Yorkshire</td>
<td>Coal measures</td>
<td>1983 – 1986</td>
<td>4.65</td>
</tr>
<tr>
<td>Project name</td>
<td>County</td>
<td>Geology</td>
<td>Construction period</td>
<td>Excavated diameter or maximum dimension (m)</td>
</tr>
<tr>
<td>---------------------------------------</td>
<td>-----------------------</td>
<td>-------------------------------------------------------------------------</td>
<td>---------------------</td>
<td>---------------------------------------------</td>
</tr>
<tr>
<td>Blisworth Canal Tunnel Renovation</td>
<td>Northamptonshire</td>
<td>Upper Lias and Northampton Sandstone in crown in middle section</td>
<td>1982 – 1984</td>
<td>7.6</td>
</tr>
<tr>
<td>Weymouth and Portland – Underhill</td>
<td>Dorset</td>
<td>Silty sand</td>
<td>1981 – 1983</td>
<td>2.1</td>
</tr>
<tr>
<td>Falkirk High Railway Tunnel Refurbishment</td>
<td>Falkirk</td>
<td>Coal Measures with Bom length in boulder clay at Edinburgh end</td>
<td>1980</td>
<td>9.62</td>
</tr>
<tr>
<td>Sheffield – Don Valley Stage 1</td>
<td>Yorkshire</td>
<td>Coal measures</td>
<td>1979 – 1983</td>
<td>6.1, 2.7, 2.2, 2.05</td>
</tr>
<tr>
<td>Lewes – Cuilfail Tunnel</td>
<td>East Sussex</td>
<td>Upper and middle Chalk</td>
<td>1978 – 1980</td>
<td>11.9</td>
</tr>
<tr>
<td>Selby Coal Field – Gascoigne Wood Drift</td>
<td>Yorkshire</td>
<td>Glacial deposits, bunter Sandstone, upper and lower magnesian limestone and permian marl, basal sands and coal measures</td>
<td>1977 – 1981</td>
<td>5.5</td>
</tr>
<tr>
<td>Thurrock – Southern Trunk</td>
<td>Essex</td>
<td>Alluvial Thames Flood Plain sands, Gravels and clays</td>
<td>N/A</td>
<td>2.14</td>
</tr>
</tbody>
</table>

Table D.1 | Tunnelling projects in the UK post 1980 (BTS online Database, June 2013 www.britishtunnelling.org.uk)