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Movimientos de Edificios Inducidos por Excavaciones: CRITERIOS DE DAÑO Y GESTIÓN DEL RIESGO

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Criterios de Daño y Gestión del Riesgo

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Eduardo Alonso Pérez de Ágreda
Marcos Arroyo Álvarez de Toledo
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Las obras subterráneas –túneles y excavaciones- son imprescindibles para integrar en la ciudad las infraestructuras de transporte. Uno de los criterios esenciales de su proyecto es la minimización del riesgo a terceros que su ejecución conlleva. Para valorar tal riesgo se debe estimar, en cada caso, la respuesta de las edificaciones circundantes a los movimientos del terreno que la obra subterránea provoca.

Esta es una tarea compleja, en la que se deben conjugar experiencia y conocimientos de ingeniería del terreno y de las estructuras. El uso de criterios de daño basados en modelos simplificados de la respuesta estructural facilita la tarea y permite el estudio sistemático necesario en grandes obras. En ocasiones, sin embargo, tal simplificación lleva aparejado un cierto grado de incertidumbre que se debe suplir con estudios más detallados. En cualquier caso, es imprescindible valorar después los resultados obtenidos por el análisis, estableciendo criterios de gestión del riesgo, en un delicado equilibrio entre las complicaciones –y riesgos propios- de la sobreprotección y el legítimo deseo de seguridad de los terceros.

El objetivo de esta Jornada es presentar una visión actualizada de los criterios de daño estructural de aplicación corriente en la evaluación del impacto sobre la edificación de túneles y excavaciones. Para ello se cuenta con las perspectivas de diferentes expertos en la materia, españoles y extranjeros, algunos de ellos pioneros de estos estudios a nivel mundial. Los distintos artículos aquí reunidos serán de utilidad para proyectistas, constructores, administraciones, técnicos y estudiantes con interés en la construcción de obras subterráneas en medio urbano.

BURLAND presenta los elementos principales de una metodología completa de evaluación de riesgo que, apoyándose en sus trabajos anteriores, se desarrolló, aplicó y verificó sistemáticamente durante la construcción de la extensión del metro de Londres, en los años 90.
El artículo incluye importantes indicaciones para la aplicación de esta metodología a otros casos.

STANDING realiza un repaso sistemático de las técnicas de observación tradicionales (no automatizadas) disponibles para la medida de movimientos en edificios. La aplicación de las técnicas descritas se ilustra con la discusión detallada de tres casos en Londres. Las estructuras observadas tenían cimentaciones de gran rigidez y puede comprobarse como los movimientos observados en el suelo son notablemente distintos a los observados en la estructura.

CORDING presentan numerosas observaciones hechas en edificios históricos con distintas tipologías estructurales en EEUU. Las observaciones realizadas se contrastan con los criterios de daño desarrollados por el autor y sus colaboradores, criterios cuya aplicación está muy extendida en nuestro país.

BOONE presenta una extensión y generalización de los criterios de daño simplificado precedentes. Esta metodología se integra con un tratamiento estadístico de las incertidumbres del problema, para obtener evaluaciones probabilísticas de daño. Esto permite cuantificar los riesgos de forma detallada y dar cabida a la variabilidad –del terreno, de las estructuras– inherente a los estudios sistemáticos de trazado. Se ilustra la aplicación de esta metodología en varios casos reales.

ALEGRE presenta un estudio detallado de la respuesta y sensibilidad estructural de las estructuras de hormigón armado a los asientos del terreno. Las tipologías objeto de estudio son directamente relevantes para muchos casos españoles.

GESTO y GENS desarrollan una extensión novedosa de las metodologías tradicionales de estimación de daño, basadas en la respuesta de una viga de gran canto. Obtienen soluciones analíticas para modos de respuesta que generalizan los asumidos tradicionalmente y, de esta manera, permiten estimar la adecuación de las simplificaciones habituales.

Los organizadores quieren agradecer la disposición de los ponentes a contribuir con sus conocimientos y trabajo a esta Jornada. La dedicación de Mar Obrador ha sido clave para el éxito de esta edición.
ABSTRACT: The construction of tunnels and excavations inevitably causes ground movements. In the urban environment such movements present a risk of damage to buildings and structures. The assessment of the degree of risk of such damage is important both from the point of view of engineering design and for planning and consultation purposes. This paper summarises an approach which draws together in a coherent way a number of related studies including the prediction of ground movement, the description and classification of damage and limiting distortions of buildings. The methodology that is presented has been used world-wide for many urban tunnelling projects.

1 INTRODUCTION

In recent years, assessment of the environmental impact of major construction projects has become a normal and required procedure. The construction of tunnels in urban areas, while having many long-term environmental benefits, can also create significant environmental impacts. During construction, such impacts would include construction traffic, noise, vibration and dust as well as temporary restrictions on access to certain roads and other public areas. Longer term impacts would include land and building acquisition, traffic and ventilation noise and vibration levels, and other impacts such as pollution, ground water changes and effects on ecology.

An environmental impact of tunnelling which is causing increasing public awareness and concern is that of subsidence and its effects on structures and services. Construction of tunnels and deep excavations is inevitably accompanied by ground movements. It is necessary, both for engineering design and for planning and consultation, to develop rational procedures for assessing the risks of damage. Coupled with such assessments is, of course, the requirement for effective protective measures which can be deployed when predicted levels of damage are judged to be unacceptable.

This paper summarises the approach that was originally developed in the 1990s for assessing the risks of subsidence damage for the London Underground Jubilee Line Extension project which involved tunnelling under densely developed areas of central London. The experience gained from this project has been summarised in a two volume book (Burland et al., 2001a and 2001b). The approach to assessing the risks of subsidence damage which is presented here draws on the results of a number of studies including the prediction of ground movements, the
description and classification of damage and limiting distortions of brickwork and masonry walls. These studies are discussed first and then combined to give the overall approach.

2 DEFINITIONS OF GROUND AND FOUNDATION MOVEMENT

A study of the literature reveals a wide variety of confusing symbols and terminology describing foundation movements. Burland & Wroth (1974) proposed a consistent set of definitions based on the displacements (either measured or calculated) of a number of discrete points on the foundations of a building. Care was taken to ensure that the terms do not prejudice any conclusions about the distortions of the superstructure itself since these depend on a large number of additional factors. The definitions appear to have been widely accepted and are illustrated in Figure 1. The following are a few points to note:

(a) Rotation or slope $\theta$ is the change in gradient of a line joining two reference points (e.g. AB in Fig. 1a).

(b) The angular strain $\alpha$ is defined in Figure 1a. It is positive for upward concavity (sagging) and negative for downward concavity (hogging).
(c) **Relative deflection** $\Delta$ is the displacement of a point relative to the line connecting two reference points on either side (see Fig. 1b). The sign convention is as for (b).

(d) **Deflection ratio** (sagging ratio or hogging ratio) is denoted by $\Delta/L$ where $L$ is the distance between the two reference points defining $\Delta$. The sign convention is as for (b) and (c).

(e) **Tilt** $\omega$ describes the rigid body rotation of the structure or a well defined part of it (see Fig. 1c).

(f) **Relative rotation** (angular distortion) $\beta$ is the rotation of the line joining two points, relative to the tilt $\omega$ (see Fig. 1c). It is not always straightforward to identify the tilt and the evaluation of $\beta$ can sometimes be difficult. It is also very important not to confuse relative rotation $\beta$ with angular strain $\alpha$. For these reasons Burland & Wroth (1974) preferred the use of $\Delta/L$ as a measure of building distortion.

(g) **Average horizontal strain** $\varepsilon_h$ is defined as the change of length $\delta L$ over the length $L$. In soil mechanics it is customary to take a reduction of length (compression) as positive.

The above definitions only apply to in-plane deformations and no attempt has been made to define three-dimensional behaviour.

![Figure 2: Settlement trough above an advancing tunnel](image)
3 GROUND MOVEMENT DUE TO TUNNELLING AND EXCAVATION

The construction of tunnels or surface excavations will inevitably be accompanied by movement of the ground around them. At the ground surface these movements manifest in what is called a ‘settlement trough’. Figure 2 shows diagrammatically the surface settlement trough above an advancing tunnel. For ‘greenfield sites’, the shape of this trough transverse to the axis of the tunnel approximates closely to a normal Gaussian distribution curve - an idealisation which has considerable mathematical advantages.

3.1 Settlements caused by tunnel excavation

Figure 3 shows such an idealised transverse settlement trough. Attewell et al (1986) and Rankin (1988) have summarised the current widely used empirical approach to the prediction of immediate surface and near surface ground displacements. The settlement $s$ is given by:

$$s = s_{\text{max}} \exp \left( -\frac{y^2}{2i^2} \right)$$  \hspace{1cm} (1)

where $s_{\text{max}}$ is the maximum settlement and $i$ is the value of $y$ at the point of inflection. It has been found that, for most purposes, $i$ can be related to the depth of the tunnel axis $z_o$ by the linear expression:

$$i = K z_o$$  \hspace{1cm} (2)

The trough width parameter $K$ depends on the soil type. It varies from 0.2 to 0.3 for granular soils through 0.4 to 0.5 for stiff clays to as high as 0.7 for soft silty clays. As a general rule the width of the surface settlement trough is about three times the depth of the tunnel for tunnels in clay strata. It is important to note that, although the value of $K$ for surface settlements is approximately constant for various depths of tunnel in the same ground, Mair et al (1993) have shown that its value increases with depth for subsurface settlements.
The immediate settlements caused by tunnelling are usually characterised by the ‘volume loss’ $V_L$ which is the volume of the surface settlement trough per unit length $V_s$ expressed as a percentage of the notional excavated volume of the tunnel.

Integration of equation (1) gives:

$$V_s = \sqrt{2\pi i s_{\text{max}}}$$

so that:

$$V_L = \frac{3.192i s_{\text{max}}}{D^2}$$

Where $D$ is the diameter of the tunnel. Combining equations (1), (2) and (4) gives the surface settlement $s$ at any distance $y$ from the centre line:

$$s = \left(0.313V_L D^2 \right) \frac{y^2}{K z_o} \exp \left(\frac{-y^2}{2K^2 z_o^2}\right)$$

3.2 Horizontal displacements due to tunnelling

Building damage can also result from horizontal tensile strain, and therefore predictions of horizontal movement are required. Unlike settlements, there are few case histories where horizontal movements have been measured. The data that do exist show reasonable agreement with the assumption of O’Reilly & New (1982) that the resultant vectors of ground movement are directed towards the tunnel axis. It follows that the horizontal displacement $u$ can be related to the settlement $s$ by the expression:

$$u = \frac{s y}{z_o}$$

Equation (6) is easily differentiated to give the horizontal strain $\varepsilon_h$ at any location on the ground surface.

Figure 3 shows the relation between the settlement trough, the horizontal displacements and the horizontal strains occurring at ground level. In the region $i > y > -i$, the horizontal strains are compressive. At the points of inflection the horizontal displacements are a maximum and $\varepsilon_h = 0$. For $i < y < -i$, the horizontal strains are tensile.

3.3 Assessment of surface displacements due to tunnelling

The above empirical equations provide a simple means for estimating the near surface displacements due to tunnelling, assuming ‘green field’ conditions i.e. ignoring the presence of any building or structure.

A key parameter in this assessment is the volume loss $V_L$. This results from a variety of effects which include movement of ground into the face of the tunnel and radial movement towards the
tunnel axis due to reductions in supporting pressures. The magnitude of $V_L$ is critically dependent on the type of ground, the ground water conditions, the tunnelling method, the length of time in providing positive support and the quality of supervision and control. The selection of an appropriate value of $V_L$ for design requires experience and is greatly aided by well documented case histories in similar conditions.

A number of other assumptions are involved in the prediction of ground displacements due to tunnelling. For example, in ground containing layers of clay and granular soils there is uncertainty about the value of the trough width parameter $K$. When two or more tunnels are to be constructed in close proximity, the assumption is usually made that the estimated ground movements for each tunnel acting independently can be superimposed. In some circumstances this assumption may be unconservative and allowance needs to be made for this.

It is clear from the above that, even for ‘green field’ conditions, precise prediction of ground movements due to tunnelling is not realistic. However, it is possible to make reasonable estimates of the likely range of movements provided tunnelling is carried out under the control of suitably qualified and experienced engineers.

### 3.4 Ground movements due to deep excavations

Ground movements around deep excavations are critically dependent both on the ground conditions (e.g. stratigraphy, groundwater conditions, deformation and strength properties) and the method of construction (e.g. sequence of excavation, sequence of propping, rigidity of retaining wall and supports). In general open excavations and those supported by cantilever retaining walls give rise to larger ground movements than strutted excavations and those constructed by ‘top-down’ methods. In the urban situation the latter are clearly to be preferred if building damage is to be minimised.

The calculation of ground movements is not straightforward because of the complexity of the problem, and much experience is required to make any sensible use of complex analyses. It is therefore essential that optimum use is made of previous experience and case histories in similar conditions. Peck (1969) presented a comprehensive survey of vertical movements around deep excavations which was updated by Clough & O'Rourke (1990). Burland et al (1979) summarised the results of over ten years of research into the movements of ground around deep excavations in London clay. The Norwegian Geotechnical Institute have published a number of case histories of excavations in soft clay in the Oslo area (e.g. Karlsrud & Myrvoll (1976)). For well supported excavations in stiff clays Peck’s settlement envelopes are generally very conservative as settlements rarely exceed 0.15% of the depth of excavation. However movements can extend to 3 or 4 times the excavation depth behind the basement wall. Horizontal movements are generally of similar magnitude and distribution to vertical movements, but may be much larger for open and cantilever excavations in stiff clays.

Advanced methods of numerical analysis, based on the finite element method, are widely used for prediction of ground movements around deep excavations. Such analyses can simulate the construction process, modelling the various stages of excavation and support conditions. However, comparison with field observations, shows that successful prediction requires high quality soil samples with the measurement of small-strain stiffness properties using local strain transducers mounted on the sides of the samples (Jardine et al (1984)).
As for tunnelling, it is essential that deep excavation work be carried out under the close supervision of an experienced engineer. Unless positive support is provided rapidly and groundwater is properly controlled, large unexpected ground movements can develop.

4 CLASSIFICATION OF DAMAGE

4.1 Introduction

Assessment of degree of building damage can be a highly subjective, and often emotive, matter. It may be conditioned by a number of factors such as local experience, the attitude of insurers, the cautious approach of a professional engineer or surveyor who might be concerned about litigation, market value and ‘saleability’ of the property etc. In the absence of objective guidelines based on experience, extreme attitudes and unrealistic expectations towards building performance can develop. It is worth stressing that most buildings experience a certain amount of cracking, often unrelated to foundation movement, which can be dealt with during routine maintenance and decoration.

Clearly, if an assessment of risk of damage due to ground movement is to be made, the classification of damage is a key issue. In the U.K. the development of an objective system of classifying damage is proving to be very 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. This classification system will now be described.

4.2 Categories of damage

Three broad categories of building damage can be considered that affect: (i) visual appearance or ‘aesthetics’, (ii) serviceability or function and (iii) stability. As foundation movements increase, damage to a building will progress successively from (i) through to (iii).

It is only a short step from the above three broad categories of damage to the more detailed classification given in Table 1. This defines six categories of damage, numbered 0 to 5 in increasing severity. Normally categories 0, 1 and 2 relate to ‘aesthetic’ damage, 3 and 4 relate to ‘serviceability’ damage and 5 represents damage affecting ‘stability’. It was first put forward by Burland et al (1977) who drew on the work of Jennings & Kerrich (1962), the U.K.National Coal Board (1975) and MacLeod & Littlejohn (1974). Since then it has been adopted with only slight modifications by BRE (1981 and 1990) and the Institution of Structural Engineers, London (1978,1989 and 1994).

The system of classification in Table 1 is based on ‘ease of repair’ of the visible damage. Thus, in order to classify visible damage it is necessary, when carrying out the survey, to assess what type of work would be required to repair the damage both externally and internally. The following important points should be noted:

(a) The classification relates only to the visible damage at a given time and not to its cause or possible progression which are separate issues.

(b) The strong temptation to classify the damage solely on crack width must be resisted. It is the ease of repair which is the key factor in determining the category of damage.
(c) The classification was developed for brickwork or blockwork and stone masonry. It can be adapted for other forms of cladding. It is not intended to apply to reinforced concrete structural elements.

(d) More stringent criteria may be necessary where damage may lead to corrosion, penetration or leakage of harmful liquids and gases or structural failure.

Table 1: Classification of visible damage to walls with particular reference to ease of repair of plaster and brickwork or masonry.

<table>
<thead>
<tr>
<th>Category of damage</th>
<th>Normal degree of severity</th>
<th>Description of typical damage</th>
<th>Note: Crack width is only one factor in assessing category of damage and should not be used on its own as a direct measure of it.</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>Negligible</td>
<td>Hairline cracks less than about 0.1mm</td>
<td></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. Typical crack widths up to approximately 1mm.</td>
<td></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 re-pointing may be required to ensure weathertightness. Doors and windows may stick slightly. Typical crack widths up to approximately 5mm.</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>Moderate</td>
<td>The cracks require some opening up and can be patched by a mason. Re-pointing of external brickwork and possibly a small amount of brickwork to be replaced. Doors and windows sticking. Service pipes may fracture. Weathertightness often impaired. Typical crack widths are approximately 5mm to 15mm or several closely spaced cracks &gt; 3mm.</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>Severe</td>
<td>Extensive repair work involving breaking-out and replacing sections of walls, 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. Typical crack widths are 15 to 25mm but also depends on the number of cracks.</td>
<td></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. Typical crack widths are greater than 25mm but depends on the number of cracks.</td>
<td></td>
</tr>
</tbody>
</table>

1 Note: 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.

Besides defining numerical categories of damage, Table 1 also lists the ‘normal degree of severity’ associated with each category. These descriptions of severity relate to standard
domestic and office buildings and serve as a guide to building owners and occupiers. In special circumstances, such as for a building with valuable or sensitive finishes, this ranking of severity of damage may not be appropriate.

4.3 The division between categories 2 and 3 damage

The dividing line between categories 2 and 3 damage is particularly important. Studies of many case records shows that damage up to category 2 can result from a variety of causes, either from within the structure itself (eg shrinkage or thermal effects) or associated with the ground. Identification of the cause of damage is usually very difficult and frequently it results from a combination of causes. If the damage exceeds category 2 the cause is usually much easier to identify and it is frequently associated with ground movement. Thus the division between categories 2 and 3 damage represents an important threshold which will be referred to later.

5 CONCEPT OF LIMITING TENSILE STRAIN

5.1 Onset of visible cracking

Cracking in masonry walls and finishes usually, but not always, results from tensile strain. Following the work of Polshin & Tokar (1957), Burland & Wroth (1974) investigated the idea that tensile strain might be a fundamental parameter in determining the onset of cracking. A study of the results from numerous large scale tests on masonry panels and walls carried out at the U.K. Building Research Establishment showed that, for a given material, the onset of visible cracking is associated with a reasonably well defined value of average tensile strain which is not sensitive to the mode of deformation. They defined this as the critical tensile strain $\varepsilon_{\text{crit}}$ which is measured over a gauge length of a metre or more.

Burland & Wroth (1974) made the following important observations:

(a) The average values of $\varepsilon_{\text{crit}}$ at which visible cracking occurs are very similar for a variety of types of brickwork and blockwork and are in the range 0.05% to 0.1%.

(b) For reinforced concrete beams the onset of visible cracking occurs at lower values of tensile strain in the range 0.03% to 0.05%.

(c) The above values of $\varepsilon_{\text{crit}}$ are much larger than the local tensile strains corresponding to tensile failure.

(d) The onset of visible cracking does not necessarily represent a limit of serviceability. Provided the cracking is controlled, it may be acceptable to allow deformations well beyond the initiation of visible cracking.

Burland & Wroth (1974) showed how the concept of critical tensile strain could be used in conjunction with simple elastic beams to develop deflection criteria for the onset of visible damage. This work will be discussed in more detail later.
5.2 Limiting tensile strain – a serviceability parameter

Burland et al (1977) replaced the concept of critical tensile strain with that of limiting tensile strain ($\varepsilon_{\text{lim}}$). The importance of this development is that $\varepsilon_{\text{lim}}$ can be used as a serviceability parameter which can be varied to take account of differing materials and serviceability limit states.

Boscardin & Cording (1989) developed this concept of differing levels of tensile strain. Seventeen case records of damage due to excavation induced subsidence were analysed. A variety of building types were involved and they showed that the categories of damage given in Table 1 could be broadly related to ranges of $\varepsilon_{\text{lim}}$. These ranges are tabulated in Table 2. This table is important as it provides the link between estimated building deformations and the possible severity of damage.

Table 2: Relationship between category of damage and limiting tensile strain ($\varepsilon_{\text{lim}}$) (after Boscardin & Cording 1989).

<table>
<thead>
<tr>
<th>Category of damage</th>
<th>Normal degree of severity</th>
<th>Limiting tensile strain ($\varepsilon_{\text{lim}}$)(%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>Negligible</td>
<td>0 - 0.05</td>
</tr>
<tr>
<td>1</td>
<td>Very slight</td>
<td>0.05 - 0.075</td>
</tr>
<tr>
<td>2</td>
<td>Slight</td>
<td>0.075 - 0.15</td>
</tr>
<tr>
<td>3</td>
<td>Moderate*</td>
<td>0.15 - 0.3</td>
</tr>
<tr>
<td>4 to 5</td>
<td>Severe to very severe</td>
<td>&gt; 0.3</td>
</tr>
</tbody>
</table>

*Note: Boscardin & Cording (1989) describe the damage corresponding to $\varepsilon_{\text{lim}}$ 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. There is therefore no evidence to suggest that tensile strains up to 0.3% will result in severe damage.

6 STRAINS IN SIMPLE RECTANGULAR BEAMS

Burland & Wroth (1974) and Burland et al (1977) used the concept of limiting tensile strain to study the onset of cracking in simple weightless elastic beams undergoing sagging and hogging modes of deformation. This simple approach gives considerable insight into the mechanisms controlling cracking. Moreover, it was shown that the criteria for initial cracking of simple beams are in very good agreement with the case records of damaged and undamaged buildings undergoing settlement. Therefore, in many circumstances, it is both reasonable and instructive to represent the facade of a building by means of a simple rectangular beam.

6.1 Sagging and hogging

The approach adopted by Burland & Wroth (1974) is illustrated in Figure 4 where the building is represented by a rectangular beam of length $L$ and height $H$. The problem is to calculate the tensile strains in the beam for a given deflected shape of the building foundations and hence obtain the sagging or hogging ratio $\Delta/L$ at which cracking is initiated. It is immediately obvious
that little can be said about the distribution of strains within the beam unless we know its mode of deformation. Two extreme modes are bending only about a neutral axis at the centre (Fig. 4d) and shearing only (Fig. 4e). In the case of bending only, the maximum tensile strain occurs in the top fibre and that is where cracking will initiate as shown. In the case of shear only, the maximum tensile strains are inclined at 45° giving rise to diagonal cracking. In general both modes of deformation will occur simultaneously and it is necessary to calculate both bending and diagonal tensile strains to ascertain which type is limiting.

The expression for the total mid-span deflection $\Delta$ of a centrally loaded beam having both bending and shear stiffness is given by Timoshenko (1957) as:

$$\Delta = \frac{PL^3}{48EI} \left( 1 + \frac{18EI}{L^2HG} \right)$$

Where $E$ is Young’s modulus, $G$ is the shear modulus, $I$ is the second moment of area and $P$ is the point load. Equ (7) can be re-written in terms of the deflection ratio $\Delta/L$ and the maximum extreme fibre strain $\varepsilon_{b_{max}}$ as follows:

$$\frac{\Delta}{L} = \left( \frac{L}{12t} + \frac{3I}{2yLY G} E \right) \varepsilon_{b_{max}}$$

where $t$ is the distance of the neutral axis from the edge of the beam in tension. Similarly for the maximum diagonal strain $\varepsilon_{d_{max}}$ equ (7) becomes:
\[ \frac{\Delta}{L} = \left(1 + \frac{HL^2}{18LH} \right) \varepsilon_{d,\text{max}} \]  

(9)

Similar expressions are obtained for the case of a uniformly distributed load with the diagonal strains calculated at the quarter points. Burland & Wroth drew the important conclusion that the maximum tensile strains are much more sensitive to the value of \( \Delta/L \) than to the precise distribution of loading.

By setting \( \varepsilon_{\text{max}} = \varepsilon_{\text{lim}} \), equations (8) and (9) define the limiting values of \( \Delta/L \) for the deflection of simple beams. It is evident that, for a given value of \( \varepsilon_{\text{lim}} \), the limiting value of \( \Delta/L \) (whichever is the lowest in equations (8) and (9)) depends on \( L/H \), \( E/G \) and the position of the neutral axis. For example, Burland & Wroth showed that hogging with the neutral axis at the bottom edge is much more damaging than sagging with the neutral axis in the middle - a result that is well borne out in practice. Figure 5 shows the limiting relationship between \( \Delta/L \) normalised by \( \varepsilon_{\text{lim}} \) and \( L/H \) for an isotropic beam \( (E/G = 2.6) \) undergoing hogging with its neutral axis at the bottom edge. For values of \( L/H < 1.5 \) the diagonal strains from equ (9) dominate whereas for \( L/H > 1.5 \) the bending strains dominate.

\[ \varepsilon_{h} = \varepsilon_{b,\text{max}} + \varepsilon_{h} \]  

(10)

Figure 5: Relationship between \( (\Delta/L)/\varepsilon_{\text{lim}} \) and \( L/H \) for rectangular isotropic beams with the neutral axis at the bottom edge using elastic beam theory

6.2 The influence of horizontal strain

It was shown in Section 3 that ground surface movements associated with tunnelling and excavation not only involve sagging and hogging profiles but significant horizontal strains as well (see Fig. 3). Boscardin & Cording (1989) included horizontal extension strain \( \varepsilon_{h} \) in the above analysis using simple superposition i.e. it is assumed that the deflected beam is subjected to uniform extension over its full depth. The resultant extreme fibre strain \( \varepsilon_{hr} \) is given by:
In the shearing region, the resultant diagonal tensile strain $\varepsilon_{dr}$ can be evaluated using the Mohr's circle of strain. The value of $\varepsilon_{dr}$ is then given by:

$$
\varepsilon_{dr} = \varepsilon_h \left(\frac{1 - \nu}{2}\right) + \sqrt{\varepsilon_h^2 \left(\frac{1 + \nu}{2}\right)^2 + \varepsilon_{d,max}^2}
$$

(11)

where $\nu$ is Poisson's ratio. The maximum tensile strain is the greater of $\varepsilon_{br}$ and $\varepsilon_{dr}$. Thus, for a beam of length $L$ and height $H$, it is a straight forward matter to calculate the maximum value of tensile strain $\varepsilon_{max}$ for a given value of $\Delta/L$ and $\varepsilon_h$, in terms of $t$, $E/G$ and $\nu$. This value of $\varepsilon_{max}$ can then be used in conjunction with Table 2 to assess the potential associated damage.

The physical implications of equations (8) to (11) can be illustrated by considering the previous case of the isotropic beam undergoing hogging with its neutral axis at the bottom edge and $\nu = 0.3$. By combining equations (8) and (10) the influence of $\varepsilon_h$ on the limiting values of $\Delta/L$ can be examined for bending strains only, by setting $\varepsilon_{b,\max} = \varepsilon_{\lim}$. Figure 6a shows the normalised relationship between $\Delta/L$ and $\varepsilon_h$. For $\varepsilon_h = 0$ the limiting values of $\Delta/L$ are the same as given in Figure 5 for various values of $L/H$. It can be seen that, as $\varepsilon_h$ increases towards the value of $\varepsilon_{\lim}$, the limiting values of $\Delta/L$ for a given $L/H$ reduce linearly, becoming zero when $\varepsilon_h = \varepsilon_{\lim}$.

Similarly Figure 6b has been derived from equations (9) and (11) for the diagonal strains only. Once again, for $\varepsilon_h = 0$ the limiting values of $\Delta/L$ are recovered from Figure 5. As $\varepsilon_h$ increases, the limiting values of $\Delta/L$ decrease non-linearly at an increasing rate towards zero. It is of interest to note that the values of $\Delta/L$ are not very sensitive to the values of $L/H$ between 0 and 1.5.

Figures 6a and b can be combined to give the resultant normalised limiting relationships between $\Delta/L$ and $\varepsilon_h$ for various values of $L/H$ as shown in Figure 6c. It can be seen that, for $L/H > 1.5$, the bending strains always control. Also for lower values of $L/H$, as $\varepsilon_h$ increases, the controlling strain changes from diagonal to bending. It must be emphasised that Figure 6 relates to the specific case of hogging with the neutral axis at the lower face and with $E/G = 2.6$.

There are similarities between Figure 6c and the well known Boscardin & Cording chart of angular distortion $\beta$ against $\varepsilon_h$. The latter chart has the following limitations:
(a) it only relates to $L/H = 1$

(b) maximum bending strains $\varepsilon_{b_{\text{max}}}$ are ignored

(c) $\beta$ was assumed to be proportional to $\Delta/L$ whereas Burland et al (2004) have shown that the relationship is in fact very sensitive to the load distribution and to the value of $E/G$.

(d) as mentioned in Section 2 the evaluation of $\beta$ is not always straightforward.

By adopting the values of $\varepsilon_{\text{lim}}$ associated with the various categories of damage given in Table 2, Figure 6c can be developed into an interaction diagram showing the relationship between $\Delta/L$ and $\varepsilon_h$ for a particular value of $L/H$. Figure 7 shows such a diagram for $L/H = 1$ and it is directly comparable with the Boscardin & Cording diagram.

6.3 Relevant building dimensions

An important consideration is the definition of the relevant height and length of the building. A typical case of a building affected by a single tunnel settlement trough is shown in Figure 8. The height $H$ is taken as the height from foundation level to the eaves. The roof is usually ignored. It is assumed that a building can be considered separately either side of a point of inflexion, i.e. points of inflexion of the settlement profile (at foundation level) will be used to partition the building. The length of building is not considered beyond the practical limit of the settlement trough, which for a single tunnel can be taken as $2.5i$ (where $s/s_{\text{max}} = 0.044$). In a calculation of building strain, the building span length is required and is defined as the length of building in a hogging or sagging zone (shown as $L_h$ or $L_s$ on Fig. 8) and limited by a point of inflexion or extent of settlement trough.

![Figure 7: Relationship of damage category to deflection ratio and horizontal strain for hogging ($L/H = 1$)](image-url)
The assessment of the risk of damage to buildings due to tunnelling and excavations

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7 EVALUATION OF RISK OF DAMAGE TO BUILDINGS DUE TO SUBSIDENCE

The various concepts discussed in the previous Sections can be combined to develop a rational approach to the assessment of risk of damage to buildings due to tunnelling and excavation. The following broadly describes the approach that was adopted during the planning and enquiry stages of the Jubilee Line Extension underground railway in London and which is now widely used internationally with minor variations.

7.1 Level of risk

The term ‘the level of risk’, or simply ‘the risk’, of damage refers to the possible degree of damage as defined in Table 1. Most buildings are considered to be at ‘low risk’ if the predicted degree of damage falls into the first three categories 0 to 2 (i.e. negligible to slight). At these degrees of damage structural integrity is not at risk and damage can be readily and economically repaired. It will be recalled from Section 4.3 that the threshold between categories 2 and 3 damage is a particularly important one. A major objective of design and construction is to maintain the level of risk below this threshold for all buildings. It should be noted that special consideration has to be given to buildings judged to be of particular sensitivity such as those in poor condition, containing sensitive equipment or of particular historical or architectural significance.
Because of the large number of buildings that are usually involved, the method of assessing risk is a staged process as follows: preliminary assessment; second stage assessment; detailed evaluation. These three stages will now be described briefly.

### 7.2 Preliminary assessment

So as to avoid a large number of complex and unnecessary calculations, a very simple and conservative approach is adopted for the preliminary assessment. It is based on a consideration of both maximum slope and maximum settlement of the ground surface at the location of each building. According to Rankin (1988), a building experiencing a maximum slope $\theta$ of 1/500 and a settlement of less than 10mm has negligible risk of any damage. By drawing contours of ground surface settlement along the route of the proposed tunnel and its associated excavations it is possible to eliminate all buildings having negligible risk. This approach is conservative because it uses ground surface, rather than foundation level, displacements. Also it neglects any interaction between the stiffness of the buildings and the ground. For particularly sensitive buildings it may be necessary to adopt more stringent slope and settlement criteria.

### 7.3 Second stage assessment

The preliminary assessment described above is based on the slope and settlement of the ground surface and provides a conservative initial basis for identifying those buildings along the route requiring further study. The second stage assessment makes use of the work described in the previous sections of this paper. In this approach the façade of a building is represented by a simple beam whose foundations are assumed to follow the displacements of the ground in accordance with the ‘greenfield site’ assumption mentioned in Section 3. The maximum resultant tensile strains are calculated from the pairs of equations (8), (10) and (9), (11). The corresponding potential category of damage, or level of risk, is then obtained from Table 2.

The above approach, though considerably more detailed than the preliminary assessment, is usually still very conservative. Thus the derived categories of damage refer only to possible degrees of damage. In the majority of cases the likely actual damage will be less than the assessed category. The reason for this is that, in calculating the tensile strains, the building is assumed to have no stiffness so that it conforms to the ‘greenfield site’ subsidence trough. In practice, however, 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 the horizontal strains.

Potts & Addenbrooke (1996 and 1997) carried out a parametric study of the influence of building stiffness on ground movements induced by tunnelling using finite element methods incorporating a non-linear elastic-plastic soil model. The building was represented by an equivalent beam having axial and bending stiffness $EA$ and $EI$ (where $E$ is the Young’s modulus, $A$ the cross-sectional area and $I$ the moment of inertia of the beam). The relative axial stiffness $\alpha^*$ and bending stiffness $\rho^*$ are defined as:

$$\alpha^* = \frac{EA}{E_s H} \quad \text{and} \quad \rho^* = \frac{EI}{E_s H^4}$$

Where $H$ is the half-width of the beam ($=B/2$) and $E_s$ is a representative soil stiffness. The eccentricity of the tunnel centre-line is defined as $e$. 


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Figure 9: Influence of relative bending stiffness on settlement profile (Potts & Addenbrooke, 1997)

Figure 10: Bending stiffness modification factors (Potts & Addenbrooke, 1997)
Figure 9 shows the influence of relative bending stiffness on the settlement profile for a 20m deep tunnel excavated beneath a 60m wide building with zero eccentricity. Figure 10 shows the modification factors to the deflection ration $\Delta/L$ that would be obtained from the greenfield site settlement profiles for sagging and hogging modes of deformation at different $e/B$ ratios. It can be seen that, for hogging in particular, the building changes from relatively very flexible to relatively very stiff over quite a small range in relative bending stiffness.

The approach of Potts & Addenbrooke of assessing the influence of global stiffness of a building is a most valuable addition to the existing methodology for assessing the risk of damage. The results can be used to make more realistic assessments of relative deflection and hence average strains within a building. It is only the first step and much has still to be done to include three-dimensional effects of building geometry and in assessing the equivalent flexural and axial stiffness of buildings.

### 7.4 Detailed evaluation

Detailed evaluation is carried out on those buildings that, as a result of the second stage assessment, are classified as being at risk of category 3 damage or greater (see Tab. 1). The approach is a refinement of the second stage assessment in which the particular features of the building and the tunnelling and/or excavation scheme are considered in detail. Because each case is different and has to be treated on its own merits it is not possible to lay down detailed guidelines and procedures. Factors that are taken more closely into account include:

#### 7.4.1 Tunnelling and excavation

The sequence and method of tunnel and excavation construction should be given detailed consideration with a view to reducing volume loss and minimising ground movements as far as is practical.

#### 7.4.2 Structural continuity

Buildings possessing structural continuity such as those of steel and concrete frame construction are less likely to suffer damage than those without structural continuity such as load bearing masonry and brick buildings.

#### 7.4.3 Foundations

Buildings on continuous foundations such as strip footings and rafts are less prone to damaging differential movements (both vertical and horizontal) than those on separate individual foundations or where there is a mixture of foundations (e.g. piles and spread footings).

#### 7.4.4 Orientation of the building

Buildings oriented at a significant skew to the axis of a tunnel may be subjected to warping or twisting effects. These may be accentuated if the tunnel axis passes close to the corner of the building.
7.4.5 Soil/structure interaction

The predicted 'greenfield' displacements will be modified by the stiffness of the building. The detailed analysis of this problem is exceedingly complex and resort is usually made to simplified procedures some of which are described in the report published by the Institution of Structural Engineers (1989). The beneficial effects of building stiffness can be considerable, as demonstrated by some recent measurements on the Mansion House, in the City of London, during tunnelling beneath and nearby - see Figure 11 from a paper by Frischman et al (1994).

![Figure 11: Comparison of observed and 'greenfield site' settlements of the Mansion House, London from driving a 3.05m diameter tunnel at 15m depth (Frischman et al, 1994)](image)

7.4.6 Previous movements

The building may have experienced movements due to a variety of causes such as construction settlement, ground water lowering and nearby previous construction activity. It is important that these effects be assessed as they may reduce the tolerance of the building to future movements. If the building is already cracked an assessment should be made as to whether the tunnelling induced movement will focus on these cracks. Sometimes this can be an advantage as other parts of the building are then at less risk of damage.

As many factors are not amenable to precise calculations, the final assessment of possible degree of damage requires engineering judgement based on informed interpretation of available information and empirical guidelines. Because of the inherently conservative assumptions used in the second stage assessment, the detailed evaluation will usually result in a reduction in the possible degree of damage. Following the detailed evaluation, consideration is given as to whether protective measures need to be adopted. These will usually only be required for buildings remaining in damage categories 3 or higher (see Tab. 1).
8 PROTECTIVE MEASURES

Before considering near surface measures, consideration should be given to measures that can be applied from within the tunnel to reduce the volume loss. There are a variety of such measures such as increasing support at or near the face, reducing the time to provide such support, the use of forepoling, soil nailing in the tunnel face or the use of a pilot tunnel. These approaches tackle the root cause of the problem and may prove much less costly and disruptive than near surface measures.

If, for a particular building, tunnelling protective measures are considered either not technically effective or too expensive, then it will be necessary to consider protective measures applied near the surface or to the building itself. However it must be emphasised that such measures are generally disruptive and can have a significant environmental impact. The main forms of protective measures currently available fall into the following six broad groups:

8.1 Strengthening of the ground

This can be achieved by means of grout injection (cement or chemical) or by ground freezing. It is normally undertaken in granular water-bearing soils. Its primary purpose is to provide a layer of increased stiffness below foundation level or to prevent loss of ground at the tunnel face during excavation.

8.2 Strengthening of the building

Strengthening measures are occasionally undertaken in order that the building may safely sustain the additional stresses or accommodate deformations induced by ground movements. Such measures include the use of tie rods and temporary or permanent propping. Caution is needed in adopting such an approach as the work can be very intrusive and may lead to greater impacts than allowing some cracking to take place which is subsequently repaired.

8.3 Structural jacking

In some special situations (eg sensitive equipment) it may be possible to insert jacks and use them to compensate for tunnelling induced settlement as it takes place.

8.4 Underpinning

There may be circumstances in which the introducing of an alternative foundation system can be used to eliminate or minimizes differential movements caused by tunnelling. However, because the ground movements are usually deep seated, such an approach may not be possible. 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.

8.5 Installation of a physical barrier

Occasionally consideration has been given to the installation of a physical barrier between the building foundation and the tunnel. This might take the form of a slurry trench wall or a row of
secant bored piles. 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.

8.6 Compensation grouting

Compensation grouting consists in the controlled injection of grout between the tunnel and the building foundations in response to observations of ground and building movements during tunnelling. As its name implies, the purpose is to compensate for ground loss. The technique requires detailed instrumentation to monitor the movements of the ground and the building. Experience of compensation grouting is reported by, amongst others, Harris et al (1994), and Harris (2001). The technique was successfully used on the Jubilee Line Extension Project in London for the protection of many historic buildings, including the Big Ben Clock Tower at the Palace of Westminster (Harris et al, 1999). More recently Mair (2008) described the successful application of compensation grouting in granular soils and the innovative application of directional drilling for installation of the grout tubes.

It cannot be emphasised too strongly that all of the above measures are expensive and disruptive and should not be regarded as a substitute for good quality tunnelling and excavation practice aimed at minimising settlement.

9 CONCLUSIONS

This paper summarises briefly a rational and coherent approach to the assessment of risk of damage to buildings due to tunnelling and excavation. The approach is based on the integration of a number of studies relating to prediction of ground movements, categorisation of damage and the factors controlling cracking of masonry and brickwork. Although the approach has an analytical framework it is evident that much reliance is placed on experience and case records.

In developing the approach it became clear that there is a conspicuous shortage of well documented case histories of measured building response to ground movements. In a concerted effort to remedy this, the opportunity offered by the construction of the Jubilee Line Extension was used to carry out a major cooperative research programme into the behaviour of selected buildings along the route. The programme costing over £1M consisted of three key projects:

- **Control ‘greenfield’ sites** were used to make very precise measurements of surface and sub-surface ground movements around the tunnels.

- **Performance of buildings.** Selected buildings were monitored for vertical and horizontal movements and any damage was accurately recorded.

- **Effectiveness of protective measures.** Careful observations and records were kept of buildings where protective measures were implemented.

The outcomes of this major research programme have been published in a two volume book describing the approaches adopted and a large number of case records of building response (Burland et al, 2001a and 2001b). The outputs from this research programme has resulted in
significant progress in our understanding of the tolerance of structures to ground movement due to tunnelling and excavation and in the assessment of the risk of damage to buildings.

REFERENCES


The Institution of Structural Engineers (1994). *Subsidence of low rise buildings*.

ABSTRACT: Ground and structural monitoring forms an essential part of any tunnelling project in the urban environment. Although there are several new developments in monitoring technology, many traditional surveying methods are still relied upon. A summary of conventional monitoring techniques is given, defining terms such as accuracy, precision and resolution. Examples of monitoring results are presented using three case studies from an extensive research project undertaken as part of the construction of the Jubilee Line Extension Project in London. At the same time the response of the ground and buildings to tunnelling are discussed and compared. The results are also discussed in the context of the methods for estimating greenfield and building response to tunnelling and potential building damage (Burland, 1995; Potts & Addenbrooke, 1997). Some of the limitations of these methods are highlighted.

1 INTRODUCTION

In recent years there have been considerable advances in the techniques of bored tunnel construction in soft ground. This has led to more and more tunnelling projects being instigated, frequently in urban areas, in order to deal with increasing congestion problems. As a consequence there is considerable international interest in predicting the ground movements and their effects on buildings. In his seminal paper Peck (1969) defined one of the three most important requirements for successful tunnel design and construction as being that the tunnel should not cause unacceptable damage to surrounding or overlying structures and services – prior to construction, the ground movements should be predicted and their effects on the structures and services assessed.

The ground response from tunnelling involves stress relief from excavation with resulting ground strains. At the ground surface these are usually characterised by a settlement trough and the challenge is to estimate whether the ground movements will cause damage to overlying buildings and structures. There are now well established approaches available for estimating ground movements and potential building damage from tunnelling (Burland, 1995).

Following the completion of the Jubilee Line Extension (JLE) tunnelling project in London a major International Conference on the Response of Buildings to Excavation-Induced Ground Movements was held at Imperial College London in 2001, organized by CIRIA. In addition to the wide variety of papers from many countries, a key feature of this conference was the extensive discussion of the two-volume book ‘Building response to tunnelling – case studies from construction of the Jubilee Line Extension, London’ (Burland et al., 2001). This book, produced by CIRIA, was a result of a major research project sponsored by London Underground Ltd, carried out under the direction of Professor Burland by Imperial College,
together with staff provided from the Jubilee Line Extension project and the Geotechnical Consulting Group. Results from further study using data from this research project were presented and discussed at the Skempton Conference (Burland et al., 2004).

Three other important recent conferences have been organized by the ISSMGE Technical Committee 28 (Underground Construction in Soft Ground): 3rd, 4th and 5th International Symposia on Geotechnical Aspects of Underground Construction in Soft Ground in Toulouse (2002), Amsterdam (2005) and Shanghai (2008). The proceedings from these symposia contain many papers on ground movements due to tunnelling and building response (Kastner, Emeriault, Dias and Guilloux (eds), 2002 and Bakker ; Bezuijen, Broere and Kwast (eds), 2005; the Shanghai proceedings are not yet in print).

This paper will initially give a brief summary of some common methods of monitoring used to assess tunnelling- or excavation-induced ground and structural responses. Monitoring gives a crucial insight into ground and structural behaviour. Professor Burland in his paper submitted to this symposium has given an extensive summary of the development and the latest methods of predicting ground movements from tunnelling and their potential damaging effects on existing infrastructure. This leads the way to comparing observations with predictions and identifying areas where there is still much uncertainty.

Much of the content of this paper is drawn from the extensive JLE research project and the publications arising from it (in particular Burland et al., 2001 and 2004). Data from some of the case studies are presented and discussed, emphasising the role of building stiffness and soil-structure interaction. Figure 1 shows the route of the Jubilee Line Extension and some of the sites referred to in this paper are marked on it.

Figure 1: Route of the Jubilee Line Extension and the research sites.
2 TRADITIONAL SURVEYING AND MONITORING TECHNIQUES

Many advances have been made in recent years with instrumentation for geotechnical and structural monitoring. In particular there have been many developments in remote monitoring where electronic instruments are programmed to record at set intervals and the data are logged and transferred without the need for personnel to visit the site. Although these advances greatly facilitate monitoring operations and enhance our understanding of the ground and structural response to changes in environment (e.g. construction works, changing ground water conditions), it is the author’s experience that usually there is still a necessity for some degree of ‘traditional’ monitoring.

The intention with this section is to discuss some of these traditional methods of monitoring and in particular to give guidance on methods of best practice to maximise accuracy and minimise costs. Much of the experience from which these notes are derived comes from the JLE research project (Standing et al., 2001). Data from some of the case studies are presented later to give examples of the accuracy that can be achieved. Potential pitfalls that can be encountered are also described.

The measurements, which were made at greenfield sites and on structures, were primarily to determine displacements and to monitor the changes in position of points in the ground and on the buildings. These are the primary measurements of interest with this type of project. Four main techniques were used by the research team: precise levelling, precise taping, facade monitoring and measurements with a Demec gauge. Additional notes are provided about electrolevel systems, the measurement of horizontal ground displacements with a micrometer stick (used at the greenfield reference sites) and also subsurface vertical displacement measurements with rod extensometers.

The comprehensive instrumentation that was installed at the two greenfield reference sites of St James’s and Southwark parks allowed deformations in three dimensions at surface and subsurface levels and changes in total and effective stresses to be monitored (further details on the monitoring techniques used and the results obtained are given by Nyren, 1998).

In the following sections there is a brief discussion on accuracy and some comments relating to monitoring in general. The main measuring techniques used by the research team to record building movements and deformations are then described along with their general accuracy and methods of processing and analysing the field data.

2.1 Definitions relating to measurement

Before describing methods, it is worth defining some of the terms used when reporting measurements, assessing instrument performance, and estimating the degrees of uncertainty the methods. The following definitions are quoted from Dunnicliff (1988).

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

- **Accuracy** is the degree of correctness of a measurement, or the nearness of a measurement to the true quantity. The accuracy of a measurement depends on the accuracy of each component of the monitoring system. It is generally evaluated during instrument calibration, where a known value and the measured value are compared. Accuracy is expressed as $\pm x$ units, meaning that the measurement is within $x$ units of the true value.
- **Precision** is the reproducibility and repeatability of measurement, or the nearness of each of a number of measurements to the arithmetic mean. Precision is also expressed as ± x units, meaning that the measurement is within x units of the mean measured value and is often assessed statistically with a degree of confidence associated with the statistical distribution; a larger number of significant digits reflects a higher precision.

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

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

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

- **Gross errors** are caused by carelessness and inexperience. They include – but are not limited to – misreading, booking errors, misnumbering of monitoring points, use of non-standard monitoring techniques, and incorrect use of instrumentation. All measurements are suspect until gross mistakes are identified and eliminated. Measurements should be checked immediately (usually by repeating the measurement) to spot and correct errors during the survey. This is not always possible, however, because of the transient nature of some types of monitoring, e.g. as tunnelling progresses.

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

- **Environmental errors** arise from the influence of factors such as heat, moisture, vibration, atmospheric conditions and lighting. Errors arising from temperature changes, for example, can sometimes be quantified and corrected, but many environmental errors are of unknown magnitude and only acknowledged in a qualitative manner.

- **Observational errors** result from different observers using different monitoring techniques. Well designed automatic monitoring systems have the potential to minimise observational error, as does the use of standard recording forms and marking flags for instrument locations.

The monitoring data presented in this paper were carefully obtained and, at the outset of any analyses with them, they have been further assessed to eliminate as far as possible the errors described above. This has had varying degrees of success, depending on the monitoring system, site conditions and instrument operators. Even after identifying and correcting errors, variation in the readings will remain as a result of random error, comprising the influence of the errors and uncertainties given above; this variation is representative of the precision of measurement.

It is difficult to ascribe specific accuracies to a monitoring system as these vary from site to site. Accuracy is therefore assessed individually for each case study building and discussed in context with the building behaviour. The accuracies for the different measuring systems
given in Table 1 give an indication of the best accuracy that was achieved by the research team.

Table 1: Examples of the best accuracies achieved for the various monitoring techniques.

<table>
<thead>
<tr>
<th>Instrument type (monitoring method)</th>
<th>Building example</th>
<th>Resolution</th>
<th>Precision</th>
<th>Accuracy</th>
</tr>
</thead>
<tbody>
<tr>
<td>Precise level (NA 3003)</td>
<td>Treasury Palace of Westminster</td>
<td>0.01 mm</td>
<td>0.1 mm</td>
<td>±0.2 mm</td>
</tr>
<tr>
<td>Total station (TC 2002)</td>
<td>Ritz: (vertical displacement)</td>
<td>0.1 mm</td>
<td>0.5 mm</td>
<td>±0.5 mm</td>
</tr>
<tr>
<td></td>
<td>Ritz: (horizontal displacement)</td>
<td>0.1 mm</td>
<td>1 mm</td>
<td>±1 mm</td>
</tr>
<tr>
<td></td>
<td>(angular displacement)</td>
<td>0.1 arcsec</td>
<td>2 arcsec</td>
<td>±5 arcsec</td>
</tr>
<tr>
<td>Photogrammetry</td>
<td>Elizabeth House</td>
<td>1 mm</td>
<td>1 mm</td>
<td>±2 mm</td>
</tr>
<tr>
<td>Tape extensometer</td>
<td>Elizabeth House</td>
<td>0.01 mm</td>
<td>0.03 mm</td>
<td>±0.2 mm</td>
</tr>
<tr>
<td>Demec gauge</td>
<td>Palace of Westminster</td>
<td>0.001 mm</td>
<td>0.01 mm</td>
<td>±0.01 mm</td>
</tr>
<tr>
<td>Rod extensometer</td>
<td>Elizabeth House</td>
<td>0.001 mm</td>
<td>0.01 mm</td>
<td>±0.2 mm</td>
</tr>
<tr>
<td>Electrolevel</td>
<td>Elizabeth House</td>
<td>2 arcsec</td>
<td>10 arcsec</td>
<td>±10 arcsec</td>
</tr>
</tbody>
</table>

2.2 General comments on monitoring

There are numerous factors that should be taken into account when planning a monitoring survey, regardless of the measuring techniques employed. Some of those commonly encountered are listed below.

2.2.1 Layout of monitoring points

Careful selection of the layout of the points is fundamental to achieving good quality data and influences the cost and ease of performing a survey. A preliminary site visit to appraise the structure and its features is an essential prerequisite to this activity.

When planning the layout of monitoring points for geodetic surveys, one or two monitoring points should be assigned as reference points, installed well outside the area of influence of the construction works where movements are not expected. If the whole structure is expected to move, these may have to be located on another nearby, uninfluenced structure, preferably of similar construction. These points act as a reference against which the relative movement of the studied structure can be put into context. They might also help with the identification of thermally or seasonally induced movements.

The distribution of monitoring points should be sufficiently close to give information to the required detail. When installing points for levelling, apart from practical constraints of performing the survey itself, consideration should be given to the feasibility of taping between the points, i.e. by avoiding where possible obstructions such as drainpipes.
Attention should also be given to choose positions that take into account features such as expansion or construction joints as these often have a strong influence on the way a building behaves.

It is always a good investment to establish at an early stage the level and position of all monitoring points, in relation to the building itself and to a suitable co-ordinate system. This helps check that coverage is adequate and enables monitoring data to be plotted accurately prior to and during construction activities.

2.2.2 Background monitoring

The importance of a good set of base readings cannot be over-emphasised. It is advisable with all monitoring projects to allocate as long a period as possible to establish background thermal and seasonal changes. This facilitates the interpretation of data and allows these movements to be isolated from other effects such as construction activity. Their magnitude and distribution depend on the way the structure was constructed and the nature of the foundations and the building materials. Often, of course, there simply will not be time to the as comprehensive a set of data as would be wished (say over a year). To some extent, however, seasonal effects can be established after the construction events, although this is not ideal if the works are taking place over a long time period where consolidation might influence the results or when monitoring is required for the implementation of protective measures.

Likely external influences are seasonal or tidal changes in groundwater level, temperature, pressure and the daily changes in the direction of maximum solar radiation. In some special cases even diurnal influences relating to tides, temperature and solar effects should be assessed by continuous monitoring over one or more 24-hour periods to establish the possible extent of daily changes. Subsequent measurements can then be put into context.

2.2.3 Weather and environmental conditions

Weather and environmental conditions have a great influence on the quality of data. These affect both the instruments themselves and their operation, the tripods that support them and also in some cases those taking the measurements! It is good practice to record a note of the weather conditions and if possible the temperature.

Weather conditions that cause particular problems are hot sunny days (even with intermittent sunshine), strong winds and heavy rain and snow. In some instances there is no choice but to survey in such conditions. Some commonly encountered problems and possible solutions are listed below.

- The feet of the tripod sinking into a bituminous surface in hot weather – use pads to spread the load
- The tripod being shaken by strong wind – greater stability is achieved by reducing the height of the legs and increasing the distance between the feet
- Reduced visibility and hence accuracy from rain, snow, mist or heat haze – reduce sight distances if possible
- Instrument malfunctioning because of strong sunlight entering through the rear of the lens – use a shade
- Tape flutter in windy conditions – use a damping system at mid length
— Non-uniform tape temperature because only parts of the tape are exposed to sunlight — if it is not possible to defer the measurement until conditions are favourable, arrange equipment and assistance so that each measurement is made as quickly as possible.

Make sure that the surface on which tripod feet are placed is firm and stable, avoiding loose paving slabs. Try to keep the position of an instrument clear of pedestrians to avoid them walking into the legs of its tripod or otherwise interfering with it.

There are particular problems from vibrations in built-up areas caused by vehicular and pedestrian movement at surface and subsurface levels, e.g. heavy vehicles moving along a narrow road, underground railways. These can cause considerable variations in the accuracy of readings from optical observations with sensitive instruments. When possible, it is advisable to take measurements at quiet periods, e.g. early morning or late evenings. This may also reduce survey time as sights are less likely to be obscured by people or traffic. It also helps avoid poor visibility resulting from traffic fumes.

2.3 Precise levelling

Precise levelling can be carried out in most types of structures and involves the measurement of the height of each point to sub-millimetre accuracy.

Precise levelling at ground level is the most common and primary form of monitoring and for this reason, considerable detail about the technique is included in this chapter. Vertical movements at ground level are usually good indicators of the extent to which other deformations of the building may occur. Precise levelling can be performed anywhere within a building and can help define any relative differences in deformation at various levels.

2.3.1 Planning the layout of monitoring points for precise levelling

As the same points installed for precise levelling can also be used to measure horizontal displacements, their positions are chosen, where possible, to give uninterrupted spans for taping. Ideally a series of points along one side of a building, wall or structure should be established to give a continuous series of spans. Such a layout allows both horizontal and vertical displacements to be measured relative to the same positions. In cases where the potential damage from building subsidence is being predicted, this is particularly relevant as building damage is assessed in terms of settlements, slopes and horizontal strains.

The layout of monitoring points should also be installed around the studied structures at spacings to give adequate coverage of the building, consistent with its structural form, and reflect the expected form of deformations, e.g. such that monitoring points are positioned at closer spacings in areas where large deformations are anticipated.

2.3.2 Precise levelling equipment and its operation

There are five components required for precise levelling.

1 The precise levelling instrument is a key component. The research team used a digital level manufactured by Leica, model type NA 3003. The instrument has the ability to measure, using an internal light-sensitive device, the height of the plane of collimation on a suitable bar-coded staff to a resolution of 0.01 mm and can take multiple measurements of an individual point. Level measurements are only stored on the module to 0.1 mm, perhaps indicating a more realistic resolution based on the manufacturer’s experience.

2 The tripod has to provide a rigid and secure platform on which to mount the level. Tripods are still generally made from wood as they are robust and temperature
insensitive. During surveying it is important to try to follow the same procedure as far as is practicable and this involves setting up the tripod in the same (preferably stable) position.

3 The staff used in most cases by the research team is 2 m long, with a bar-coded invar strip. A shorter 0.8 m long staff was used in conditions with reduced headroom (e.g. the sub-basement of the Treasury). The use of a bar-coded staff, in conjunction with the light-sensing device within the NA3003, eliminates parallax errors introduced by incorrect adjustment of the eye piece.

4 A stable benchmark. It is important that the levelling run starts from a stable datum outside the zone of influence of any activity likely to cause movement. Ideally this should be a deep datum, e.g. a rod extensometer with its anchor installed to a depth well below any zone of influence. If this is not possible (e.g. when the survey is within the basement of a building) the starting point, or the part of the structure in which it is installed, should be checked independently. The research team established a series of benchmarks adjacent to the structures of interest but outside the zone of influence of the construction works. Generally they were located close to or at ground level on structures or pavements that showed no signs of previous deformation and that were not adjacent to trees. Several of the research surveys were within the basements of buildings. In such cases a benchmark was established in the basement as far as possible from the construction works and additional points were established at ground level to extend the survey to unaffected ground. It is also worth noting that the absolute levels of a number of structures studied by the research team were affected by the deep dewatering in the vicinity. These activities resulted in settlement of the ground surface and structures over very wide areas of London. In these areas the settlement affected the benchmark to the same extent as the structures of interest and thus absolute settlements were not measured.

5 Reproducible measuring points. Selection of the type of monitoring point, its fixity and shape, have an important influence on the accuracy of readings. The most repeatable results are obtained when the staff can be placed on the same point in the same way each time. The Building Research Establishment, BRE socket and levelling plug have been designed so that the plug is removable and is screwed into the socket for each survey (see Fig. 2a). Details of the BRE socket and levelling plug are given in BRE Digest 386. This arrangement allows the plug to be repositioned each time to within 0.1 mm in both the vertical and horizontal senses. A smaller version of the BRE socket was produced which makes installation easier and which is also less obtrusive (see Burland & Standing, 1997 and Fig. 2b). Care must be taken to keep the machined surfaces clean in order that the plug can be repositioned in exactly the same manner. The sockets were grouted in place using a sand-cement mortar keeping them as horizontal as possible.

Some alternative, inferior designs of socket and plug available which rely on screwing the plug as tightly as possible into the socket. The sockets used by the JLEP contractors were often different from the BRE type adopted by the research team, being of brass with a diameter and length of about 10 mm and 40 mm respectively. These sockets required the use of a 75 mm extension piece screwed into the levelling plug. A much smaller drilled hole was required for these sockets which were installed using an epoxy-resin with sand filler. The accuracy of measurements using this type of socket is less than that obtained with the BRE sockets with typically a scatter of data at least twice that obtained by the research team.
If an alternative socket design (or surveying technique) is adopted, it is recommended that
the reproducibility with which the plug can be repositioned should be established before the
survey programme commences to be sure that the accuracy required can still be achieved.
This exercise could be done by taking numerous readings from one station to one or more
points, removing and repositioning the plug each time; the variability between different
people screwing in the plug and holding the staff should also be assessed.

Locations that should be avoided as precise levelling points include marked or scribed lines
on ledges, walls or level (approximately) surfaces or nails/studs installed in yielding, soft or
friable surfaces. Installation of points in mortar between bricks or joints between paving
should also be avoided.

2.3.3 Precise levelling procedures

The method of precise levelling followed by the research team uses practices described in
BRE Digest 386. The research team followed, as far as was practicable, a set survey
sequence in which the aim was to set up the tripod at approximately the same location for
each survey and use the same monitoring points for change points. Distances to both
intermediate and (particularly) change points were generally less than 20 m to maintain
accuracy. When the instrument is moved during the levelling run, back- and foresights to
change points should be made at similar distances as far as is practicable to minimise any
collimation errors. Once the furthest measuring point has been sighted, the survey should be
closed either by returning to the initial datum to check on the closure of the run or to a
separate benchmark. Under favourable conditions the closing error should be within 0.3 mm.
A larger error might have to be accepted under adverse surveying conditions.

2.3.4 Measurement accuracy in precise levelling

The precise level instrument used by the research team shows individual measurements to
0.01 mm on its monitor, but the value taken for the level and continuation of levelling is only
stored to 0.1 mm, implying perhaps a more practical degree of accuracy. During levelling an
average of three individual measurements was taken which would generally be within 0.02
mm of each other. However, even if a consistent monitoring procedure is followed and the same equipment is used, the absolute field accuracy is also influenced by a number of factors.

**Monitoring point and staff errors.** When using BRE sockets, the levelling plug can be relocated each time to within 0.1 mm. When taking immediate sights, the accuracy of the measurement does not affect the overall survey. However, for change points, the accuracy of measurement of the fore- and backsight will affect the overall accuracy of the survey. It is good practice to keep the staff on the measuring point throughout the period of both sights. Thus if the plug or socket are loose there is a greater likelihood that the level would be the same for both sights, although this could result in discrepancies with other surveys.

**Weather and environmental conditions** affect the accuracy of precise levelling, as described earlier. Their effect can be quantified to some extent by the overall closing error and the quality of data. Digital levels are particularly susceptible to strong sun and it is recommended that the instrument is shaded in these conditions. In addition hot weather often creates heat hazes and the associated shimmering which interfere with the passage of the infra-red rays between the instrument and the staff.

Traffic vibrations are a common source of error and are best avoided when possible by surveying during quiet periods. Strong or gusting winds are another common source of error. Locations that are particularly susceptible to wind are street corners, the areas around tower blocks and near bodies of water.

In adverse conditions it may be necessary to repeat measurements and relax the permissible differences in readings to a single point in order to complete a survey.

**External influences and instability of the benchmark.** There are a number of external influences which may affect the absolute level of monitoring points. These may not be significant when assessing differential settlements but should be taken into account when assessing absolute measurements. The following list represents examples experienced by the research team:

- Seasonal changes in moisture content of the near-surface soils
- Damage of surface monitoring points by traffic
- Changes in water level in bodies of water, e.g. tidal/seasonal effects
- Construction operations including groundwater control measures
- Temperature changes, e.g. freezing of the ground, softening of bituminous surfaces
- Sinking ground during surveying, e.g. due to rapid tunnelling operations.

These factors should be considered when planning the monitoring surveys and assessing the results of precise levelling.

**Collimation errors.** Precise levelling relies on the instrument having a line of sight in an exactly horizontal plane. Modern digital levels have self-levelling devices. This, however, does not guarantee that the axis of the instrument telescope is in a totally level viewing plane. If the optical axis is not level, errors are introduced which increase with larger sighting distances from the instrument to the staff. During the course of the research monitoring, checks for collimation error were made periodically using the ‘two-peg test’. Additionally, distances to foresights and backsights were kept roughly equal to minimise potential collimation error.
Estimating the overall survey accuracy. The simplest and probably the most representative measure of the overall survey accuracy is the closing error. This gives an indication of the sum effects of the various potential sources of error over the length of the survey. However, it does not necessarily represent the accuracy of individual points. Achieving good accuracy is also related to frequency of measurements and having a good set of base readings to understand background movements. Closing errors in the surveys were generally small, i.e. within ±0.25 mm. Depending on the particular conditions of the survey and environmental conditions, accuracy was sometimes much better, e.g. ±0.1 mm, although on occasions it was worse at ±0.5 mm.

2.4 Precise taping measurements

Precise taping with a tape extensometer was carried out on several of the structures to obtain changes in spans between precise levelling points. Ideally a series of points along one side of a building, wall or structure, with distances between them typically of 5 to 10 m, should be established to give a continuous series of spans (see also earlier comments on planning the layout of monitoring points). This allows horizontal strains to be determined which can then be correlated to changes in level. By careful selection of the location of monitoring points it is possible to obtain a continuous profile of strain over the length of a building. As with precise levelling a sub-millimetre accuracy of measurements was achieved.

The research team used a particular type of extensometer – the Ealey extensometer – but most of the comments below apply to other tape extensometers.

2.4.1 Equipment required for precise taping

The two main components of the tape extensometer are the tape itself and the measuring instrument, which is supported on a frame to which the tape is also attached and spooled. These and the other main components are shown in Figure 3.

The tapes are made of steel and may be of different lengths but are typically 20 m long. Steel is preferred to other materials (e.g. invar) because of its known characteristics, quick response to temperature changes, cheapness and availability. Holes are precisely punched in the tape at 50 mm intervals and the free end of the tape has a hook permanently attached.
During measurements, the hook at the free end of the tape is connected to a demountable eye-bolt that can be screwed into a BRE socket. A second demountable eye-bolt is located at the other end of the span (see Fig. 3). These connectors have two axes of rotation that allow them to act as a universal joint (i.e. the eye-bolt can be positioned at any orientation).

The extensometer instrument itself has a yoke at one end with a spigot which passes through one of the punched holes in the tape at 50 mm spacings. The yoke is attached to sliding shafts which are connected to a strain-gauged device housed within the instrument with an electrical trigger mechanism that illuminates a green display light on the instrument when a tension of 133 N (30 lbf) is applied to it. The tension is applied by means of a winding handle which retracts (or extends) the yoke. This device enables a constant tension to be applied to the tape for each measurement. There is also a trigger connection to a red light that comes on if the tape is over-tensioned. A digital display on the instrument indicates the amount the tape has been wound in or let out.

The tape, being of steel, is very sensitive to temperature and for this reason a good quality thermometer is required. The research team used a platinum resistance digital thermometer, capable of resolving to 0.1°C.

2.4.2 Procedures for taping measurements

As there is little available in the literature about guidance for taping measurements, the following paragraphs are provided to explain the rudiments of the methods used by the research team. Causes of inaccuracy and other site potential site problems are also addressed.

The demountable eye-bolts are screwed into the BRE sockets at each end of the span to be measured. The tape is then hooked to one end of the span and spooled out to reach the other end where the hook on the instrument is attached (Fig. 4). The appropriate punched hole is then located onto the spigot and the tape wound in to take up the slack (it is important that the same punched hole is used each time). The thermometer is usually placed on the ground about halfway along the span being measured. At the start of a series of measurements, the tape should be left set up on the first span for about five minutes before taking readings to allow it to equilibrate with the ambient temperature.

![Figure 4: Sketch of tape extensometer set up ready for measurement of a span.](image)

The tape is tensioned to the correct value as indicated by the green light. For each span a set of three readings should be taken which are within a range of about 0.03 mm. Readings vary if the surrounding temperature is changing (or if the instrument is still equilibrating), they
will generally increase if the temperature is falling and *vice versa*. During the course of the research monitoring with the tape extensometer, it was found that there is a degree of operator influence on the readings (related to the manner in which the tape is tensioned).

It helps if two people take measurements: one to operate the instrument and the other to log the readings and observe the temperature gauge. The second person is needed when people might walk or drive into the tape during measurements. A tensioned tape is easily broken by someone walking into it.

The layout of BRE sockets should provide a continuous series of spans along the building side or facade being monitored. This is not always possible because of irregularities in the walls (recesses, projections, etc.) or because of drainpipes. Spans should be between 5 and 10 m in length. Shorter lengths can lead to less reliable measurements because the instrument length accounts for a greater proportion of the taped length and temperature corrections are not so readily applied. Longer spans are more prone to tape flutter from breezes. Experience indicates that repeatable measurements are more difficult to obtain with longer spans and so are less reliable.

When planning the layout of monitoring points, provision should be made to include one or two control spans, i.e. that are independent of the agency causing the other spans to strain. If the whole structure is expected to strain, these may have to be located on another nearby, uninfluenced structure, preferably of similar construction. These spans act as reference distances on which the extensometer can be checked each time and also provide a measure of accuracy.

### 2.4.3 Factors affecting the accuracy of taping measurements

Establishing the accuracy of a series of taping measurements is not as straightforward as with precise levelling. Factors concerning the use of BRE sockets are common to both types of measurement. The resolution of the tape instrument is 0.01 mm and if the three readings taken for each span fall within a range of 0.03 mm accuracy should be enhanced. However, the overall accuracy is very dependent on environmental conditions, particularly temperature.

The prevalent weather conditions can have serious adverse effects on the accuracy and reliability of readings. Direct sunlight on any part of the tape should be avoided, radiation heat-effects often not being accurately measured by a temperature gauge. The structure being monitored also responds to sunlight further complicating interpretation of the data. If there is a possibility that spans will be exposed to sunlight, readings should be taken early in the morning, i.e. after the overnight period of stabilisation of building temperature and before day time conditions start to have significant effects.

In exposed areas, even small breezes will cause tape flutter. This effect of tugging at the tape causes the green tension indicator light to come on prematurely. Providing the tape span is not too long, (e.g. less than 10 m), this can be largely avoided by the second person holds a piece of folded paper to enclose (rather than to grip tightly) a short length of the tape at about mid-span which damps out the flutter without applying any extra tension to the tape.

As noted above, it was found necessary to follow a rigorous procedure for these measurements in order to avoid the largely operator-dependent gross and observational to which the technique would otherwise be prone. Once implemented, the procedure led to an improved accuracy.
2.4.4 Correction and analysis of taping results

It is changes in span length that are of primary interest. Unless movements are very large requiring a new punched hole position to be used, these are given by the readings shown on the digital display. The overall span is required for determining strains and this can be taken as the sum of the reading given on the digital display, the length of the tape to the punched hole and the instrument length.

The primary correction that has to be implemented to raw readings from site relates to thermally-induced changes in the tape length. Applying temperature corrections can be tackled using (1) a theoretical calculation method or (2) an empirical approach where the tape has been calibrated by measuring a series of spans at various temperatures and producing a chart with correction curves for different span lengths. The former method has been used for the research measurements.

In fact the fabric of the structure is also affected by temperature changes. During the initial processing of taping data, corrections were applied to account for temperature changes to both the steel tape and the building fabric. However, the processed results were not well conditioned and it was evident that the two corrections were essentially cancelling each other out. The coefficient of thermal expansion of concrete or masonry is very similar to that of steel (see BRE Digest 228, 1979). The factor that makes the difference is that the conductivity of the two materials is quite different, the tape equilibrating in a few minutes while the structure might take several weeks. Because of the uncertainty about the time-lag and the temperature distribution within a structure, corrections for any thermally-induced building movements were not applied. It is also useful to appreciate the magnitude of thermally-induced movements of the building, which, after all, are real.

The expression used for correcting the raw data for each span at time, $t$, is given below.

$$s_{At}^* = s_{At}(1 + C_{tape} \Delta T_{At})$$

$s_{At}^*$ is the measured span corrected for the temperature of the tape.

$s_{At}$ is the measured length of span $A$ at time $t$, (i.e. $s_{At}$ is the tape length to the punched hole being used plus the reading taken on digital display, but without taking instrument length into account).

$C_{tape}$ is the coefficient of thermal expansion of tape (0.0000124 / °C for the steel tape used).

$\Delta T_{At}$ is the change in temperature in relation to the average value of that from the base readings.

$$\Delta T_{At} = T_{At} - T_{A0}$$

$T_{At}$ is the measured temperature at time $t$ (over span $A$) and $T_{A0}$ is the average value taken during the base readings at time $t_0$. Positive values relate to an increase in temperature.

The change in span, with changes in tape length due to temperature eliminated, can be given by $\Delta s_{At}^*$, the difference between the current corrected reading and the original span, $s_{A0}$.

$$\Delta s_{At}^* = s_{At}^* - s_{A0}$$

Positive values indicate that the span length is increasing.

It is often useful to convert displacements to strains, especially when assessing potential damage to the fabric of a building, damage classifications often being related to strain ranges. The strain at the midpoint of each span at time, $t$, can be expressed as:
\[ \varepsilon_{tape} = \frac{\Delta s_{st}}{s_{st} + 0.544} \]

where \( L_{inst} \) is the length of the instrument (0.544 m for the Ealey extensometer). The expression is set up to give the sign convention used in structural engineering, i.e. positive values indicate expansion (tension) and negative contraction (compression).

The measurements from a series of spans can also be expressed as cumulative displacements relative to one end of the line by summing individual span changes.

2.5 Facade monitoring with a total station

Facade monitoring involves surveying points on the facade with a high-precision total station (which measures horizontal and vertical angles, and distances to retro-reflective prisms) to establish changes in their positions. Plane facade movements in a three-dimensional Cartesian co-ordinate system can then be determined from these measurements.

2.5.1 Equipment used for facade monitoring

The total station instrument used by the research team for most of the surveying work was a Leica TC 2002 which is fully electronic and all data are stored on a record module (a less accurate Leica TC 1610 was used for a short period for some of the monitoring). The TC2002 instrument can measure angles and distances to a resolution of 0.1 seconds of arc and 0.1 mm respectively. Corrections are entered into the software at the start of the survey to take atmospheric conditions into account (temperature and barometric pressure). The instrument is mounted on a tripod during measurement.

The targets installed on buildings were retro-reflective prisms mounted on a plastic square laminate with cross lines marked on them. This type of target allows distances to be measured with the total station. Sizes generally used are 60 x 60 mm on upper floors or far-off points and 40 x 40 mm on nearer positions. Targets are placed to give good coverage of the building, with a greater density in areas of the building most likely to be affected by construction, and where possible remote from potential vandalism.

2.5.2 Procedures for facade monitoring

Measurements to the targets are made from survey stations at ground level opposite the facade. Reference points affixed to structures outside the zone of influence of JLE construction activities were also established to provide a frame of reference for the angle and distance measurements. These were installed at the same time the targets on the building itself and were assumed to be stationary during all site monitoring.

Survey station positions are carefully selected to maximise the number of targets that can be seen from each. It is important that the targets are seen from at least two stations to supply redundant observations as this considerably increases confidence in the measurements. Also, when the angle of the instrument to the target becomes too oblique it is often not possible to measure distance. It is then essential that angle measurements are made from two stations to such targets.

The survey station positions are marked on the ground surface (e.g. with a nail) and their locations co-ordinated at the start of each survey using the ‘stationary’ reference targets. The stations are also co-ordinated in relation to each other using a tripod-mounted reflector centred over the nail at each location. This reflector is a ‘cube-corner prism’ with cross-hairs for measuring horizontal and vertical angles while the prism returns transmitted waves from the electro-distomat (EDM) back precisely in the same direction as they are received for distance measurement. Coordinating between stations also improves the accuracy of
measurements. Base lines between stations and reference points are then established after a number of initial base readings.

Angle measurements are made on both faces of the instrument many times to eliminate face errors, although in the total station used (i.e. TC2002) they can be eliminated automatically. Taking measurements on both faces does give additional redundant readings, thus leading to an increased accuracy. After surveying all the visible targets, the measurements on the reference targets were repeated. The survey was then repeated from the other survey stations.

Total station measurements were also made to shallow surface settlement points at the St James’s Park greenfield reference site to obtain the three-dimensional movements of the line. Measurements to the points were made using a demountable retro-reflective prismatic target with a thread system of the BRE monitoring plug incorporated in the design. The top of the target rotates freely around the vertical axis via a precision bearing enabling the prism orientation to be adjusted to face the total station squarely. Details of the procedure used at the greenfield reference sites are given by Nyren (1998).

2.5.3 Processing and analysis of total station measurements

Prior to processing the measurements it is necessary to establish a local co-ordinate system and determine the position of all reference points relative to this local system using triangulation and trilateration. The positions were calculated for a number of surveys performed prior to construction activity to establish the initial co-ordinates for each reference point. These were re-calculated for surveys performed later in the construction sequence to confirm that the reference targets were stable relative to each other over longer time periods.

The results are processed with the assistance of a computer program which adjusts all observed values of horizontal and vertical angles and distances by the method of least squares. The principle uses redundant measurements taken during each survey to adjust the measured quantities to satisfy geometric criteria (e.g. triangulation and trilateration) whilst minimising the adjustments made to the actual measurements.

After the analyses are performed, calculated co-ordinates relative to the local system are transformed to the x, y and z co-ordinate system.

2.5.4 Uncertainty and sources of error with total station measurements

The TC2002 resolves to 0.1 second of arc for horizontal and vertical angles, and to 0.1 mm for distances while the TC1610 resolves to 1 arc second for angles and 1 mm for distances. However, the accuracies quoted by the manufacturer for both instruments are about 1 arc second and 1 mm. The accuracy that can be obtained in practice is dependent on numerous factors.

Within the instrument, the relative arrangement of the two axes of rotation and the line-of-sight may result in systematic errors in angle measurements. Ideally the line-of-sight should be exactly perpendicular to the tilting axis, and the tilting axis exactly perpendicular to the standing axis. Deviations from the perpendiculars are manifested as horizontal collimation errors (or line-of-sight errors) and vertical collimation errors (tilting-axis errors) respectively. By repeating measurements on both instrument faces, the two collimation errors can be compensated for by taking the average of the measured angles for each target; this also helps to identify gross errors, and can reduce random errors in the measurements and provides a better statistical average. When the horizontal angles are calculated (as the difference between two horizontal angle readings) the line-of-sight error is minimised. The
errors may also be affected by large ambient temperature fluctuations (e.g. direct sunlight) which can put the instrument out of level.

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

Closing errors (i.e. differences between measurements to reference points at the beginning and end of the survey) were generally less than 5 arc seconds for both vertical and horizontal angles and less than 1mm for distances. These errors are considered to be representative of the general accuracy of measurement for the entire survey, as the source of the closing error and its occurrence within the survey is not easily determined. This angular error equates to approximately 0.5 mm/20 m distance perpendicular to the line of sight. The distance between the instrument and the point being measured and their relative positions therefore also influence accuracy.

Variations in atmospheric temperature and relative humidity affect the infra-red waves used to measure distances. The associated error resulting from changes in ambient temperature for different relative humidities has been estimated to be less than 0.5 mm.

The accuracy of precise surveys with the total station was usually found to be better in the vertical sense than horizontally. This arises principally from the effects of sun or building movements on the reference points. The estimated accuracies from observations made on the Ritz Hotel are given in Table 1.

2.6 Electrolevel monitoring

Electrolevels enable rotations to be measured of either a discrete point or, if mounted on a beam, a span. The rotations can then be integrated from one end of a string of beams to provide a profile of displacements. Electrolevel measuring systems were used extensively on many of the JLE project contracts. The main advantage with these systems is that they are able to provide real-time measurements during tunnelling, compensation grouting or other construction operations. If installed and used under suitable conditions, they can provide an accurate means of determining displacements. In practice it was found that these devices were very temperature sensitive and unless adequately insulated, thermal effects completely masked movements. There were also problems with data management and interpretation. These systems were used in a limited number of cases by the research team, primarily for determining subsurface horizontal displacements which involved installing the devices within boreholes.

2.6.1 Description of electrolevel devices

Electrolevels are small glass vials which contain an electrolytic fluid and three electrodes which are partially immersed in the fluid as shown in Figure 5a. The instruments are energised with a small electric current and the voltage of the arrangement is measured and converted to a digital reading. When the electrolevel is tilted, the length of each electrode immersed in the fluid changes. This causes a change in the resistance of the circuit and a change in the measured voltage for a constant current through the circuit. The voltage change is calibrated against tilt over a 2 to 3° range of rotation in controlled laboratory conditions prior to installation, yielding a near-linear calibration factor for each electrolevel under constant temperature conditions. Continuous strings of lightweight beams with an
electrolevel mounted on each were installed along the walls of numerous buildings along the route of the JLE for real-time monitoring of movements from tunnelling and grouting operations. The devices were also installed in boreholes to measure horizontal ground movements.

2.6.2 Horizontal displacement measurements using electrolevel inclinometers

At the two greenfield reference sites and at Elizabeth House, uniaxial electrolevels which rotate within a single plane were installed by BRE staff at discrete positions within standard inclinometer tubes grouted into boreholes. The key-way slots within inclinometer tubing were aligned with and orthogonal to the tunnel axes. Prior to electrolevel installation, and again after their removal, the tubes were mapped using a standard inclinometer torpedo.

Figure 5: Schematic diagram of an electrolevel showing (a) the basic operating principle and (b) details of the electrolevel carriages used in the instrumentation tubes.

The electrolevel vials were mounted on short carriages as shown in Figure 5b, each with its own independent cable lead and socket running to the surface. On installation the carriages are placed into the tubing grooves and carefully pushed to the desired depth; readings were then taken to verify that the initial positions were aligned near the middle of the response range (i.e. at the null position).

During the main periods of construction, the electrolevels were connected to multiplexer units for automatic computer logging. The multiplexers energised each electrolevel in a set sequence every 15 minutes and the responses were automatically logged using a portable computer. Manual readings were taken during the rest periods between tunnelling events using a Fylde hand-held read-out unit which measured the voltage response after five seconds of energisation.

2.6.3 Uncertainty and sources of error with electrolevel inclinometer measurements

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

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

It has long been known that electrolevels are readily affected by temperature changes, but in the inclinometer boreholes, temperatures are generally constant. However, when using the auto-logging system it was evident that diurnal changes in ambient temperature resulted in variations in response of up to ±4 arc seconds.

Because each electrolevel measures tilt at one discrete point, significant errors may arise in processing the measured data if the displacement gradients change rapidly between adjacent measurement depths. Additionally, only uniaxial electrolevels were used in the inclinometer holes, and it is recognised that components of displacement, other than that which is intended to be assessed, may also affect the measured tilt.

2.6.4 Processing electrolevel inclinometer measurements

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

\[
\theta = (R - R_0) \cdot CF
\]

Horizontal displacements for each inclinometer hole were determined by estimating appropriate gauge lengths for each electrolevel and assuming that the measured rotation is valid over the entire gauge length (i.e. the inclinometer tube is effectively assumed to be ‘hinged’ at the ends of each length). The gauge length limits were assumed to coincide with the mid-point between each adjacent electrolevel.

Over each discrete section of tube the displacements were calculated and then combined to form a cumulative displacement curve over the entire tube length. The interpretation requires movements at either tube end to be known or assumed. In most cases the deep inclinometers below the tunnel axes at the base of the tube was assumed to be stable. For shallow inclinometers (i.e. above the tunnel crown) movements at the top of the tubes were estimated from the surface displacement measurements (e.g. precise levelling). Further details of the analysis of the electrolevel data are given by Nyren (1998).

3 OBSERVED BUILDING RESPONSE TO TUNNELLING

The results from three of the structures monitored at the western end of the Jubilee Line Extension as part of the research project mentioned earlier (Jardine, 2001) are now presented and discussed. The monitoring techniques used to obtain the data and their respective accuracies are also considered along with the processing of the data.

In the following sections, only brief descriptions are given of the buildings and the construction works that affected them. References are provided for each case study where detailed information can be found (in the two-volume book mentioned earlier, Burland et al., 2001). The primary objective of the section is to discuss the buildings’ responses per se but also to compare them with what would be expected from greenfield movements.

The buildings, referred to, in order of their location along the route of the tunnel, heading eastwards, are the Ritz Hotel, Piccadilly, the Treasury, Westminster and Elizabeth House, Waterloo (see Fig. 1). Each building, along with its response and monitoring, is discussed individually. The cumulative lessons learnt are then summarised collectively.
3.1 Ritz Hotel, Piccadilly

The Ritz Hotel is a well-known landmark on Piccadilly in London’s west-end. It was constructed by 1906 and has plan dimensions of about 71m along the front façade and 35m on Arlington Street (see Figs. 6 and 7). The structure is of steel-frame construction, most of which is encased in clinker concrete, which was also used for the floor slabs. The superstructure is supported on concrete pad grillage foundations that vary in size up to 5 m square, founded on London Clay beneath the 7-m deep basement to the building. Above the double-storey height ground floor there are another five storeys in the main part of the structure, over which are two further floors within the sloping roof. The walls to the building are of brickwork, clad to the first coping level with granite and from there to the top of the fifth floor with Portland stone.

The existing Piccadilly line tunnels are shown in plan in Figure 7 together with the brief outlines of the underground works that were made to connect the new Jubilee Line Extension with the Piccadilly line tunnels. The passenger interchange passageway (PIP) was constructed beneath Arlington Street, starting from the shaft shown just north of Park Place. The PIP tunnel was constructed in two stages. First a 2m diameter pilot was excavated, which was opened out later to a final diameter of 4.6 m. The pilot was stopped temporarily just north of Bennett Street and an array of tube-à-manchettes (TAMs) were installed beneath the eastern end of the Ritz as a contingency protective measure prior to proceeding into the area where the pilot tunnel would have an influence on the building. In addition to the PIP tunnel there were a number of localised hand-excavations into and around the existing Piccadilly line tunnels to connect these with the PIP. Further details concerning the structure, its history, the JLEP works and the research monitoring are given by Johnston (2001) and Standing et al. (2001).

The research monitoring of the Ritz was by measurements to two of the façades using a precision total station instrument. Three rows of six retro-reflective targets were installed along the Piccadilly façade at just above the ground floor, second-storey and fourth-storey levels. This façade is roughly transverse to the tunnelling works. Measurements were also made to the Arlington Street façade, but these are not discussed here. The surveys were performed from three stations along Piccadilly on the other side of the street, using two far-off points as references (see Standing et al. (2001) for general details on total station monitoring and Standing et al. (2001) about the surveys specific to the Ritz).

In the following section the data from just one survey, in relation to the base reading surveys is discussed. This survey was made on 17 October 1997, after installation of the TAMs, and construction and enlargement of the pilot tunnel, but prior to any compensation grouting. At this time the settlement of the eastern end of the building was approaching the 14 mm limit at which point compensation grouting was to be implemented.

Measurements with a total station allow three components of movement to be calculated. The data have been analysed to obtain displacements vertically and horizontally parallel and perpendicular to the building façade (referred to as in-plane and out-of-plane movements respectively).

The vertical displacements at each of the three levels of the building façade along its length, as measured on 17 October 1997, are shown in Figure 8a. It can be seen that measured settlements at each position along the length are within 1 mm independent of the level of the targets with very consistent data sets for each level. This is not surprising given the dimensions of the building and the magnitude of movements. The data should be assessed...
considering the accuracy of the total station measurements, which is estimated to be \( \pm 0.5\text{mm} \) and \( \pm 1\text{mm} \) in the vertical and horizontal senses respectively (Standing et al., 2001).

The settlement data were plotted as \( \ln(s/s_{\text{max}}) \) versus the square of the distance from the tunnel centre-line to assess whether the movements followed the form of a Gaussian curve and also deduce a value of the point of inflection along the building façade. The resulting plot indicated that the shape of the settlement trough was essentially Gaussian. A volume loss of just over 3% was calculated from the area beneath the settlement trough, and dividing it by the area of the enlarged pilot tunnel alone. This value is higher than would have been caused by the tunnelling alone. In fact the hand-mining operations around the Piccadilly line tunnels would have also contributed to the building settlements but assigning an excavated area to them is not straightforward: if their equivalent area were taken into account this would reduce the volume loss considerably. Using the values of \( V_l \) and \( i \) appropriate to the measured curves, a Gaussian curve has been generated which is also shown on Figure 8a. It can be seen that the maximum settlement, which is usually assumed to lie above the centre-line of the tunnel, is located at the end of the building. Although the tunnel centre-line is offset to the east by a few metres and lies beneath Arlington Street, the end of the building has been chosen to represent the point where the maximum settlement occurs (and about which the Gaussian curve is symmetrical) to account for the works around the existing Piccadilly line tunnels and also because this is consistent with the three settlement profiles. It is also convenient as the Gaussian curve thus generated can be used to develop profiles of other quantities along the length of the building, namely deflection ratio \( \Delta/L \), horizontal in-plane displacements and horizontal strain.

The same Gaussian curve is shown in Figure 8b, with the position of the point of inflection marked. A profile of deflection ratio along the building, calculated using this curve, is shown in Figure 8c. Following the procedure illustrated in Prof Burland’s paper (Fig. 8), the building has been partitioned into two parts either side of the point of inflection (shown as B), i.e. in the sagging and hogging zones. In the sagging zone, the deflection ratio has been calculated.
by taking chords from point A connecting to the Gaussian curve at distances increasing by 1 m at a time up to the point of inflection at point B. For each of these chords the maximum deflection $\Delta_{\text{max}}$ has been found, and divided by the horizontal distance between A and the point of intersection of the chord with the curve, to give the deflection ratio. The same procedure was then performed but working from the point B back to A to give a complete profile over the sagging zone. It should be noted that the position of the maximum value is usually just under halfway along the chord length and if this second stage were not performed there would only be data for the first half of the sagging zone. The resulting curve of $\Delta/L$ is slightly concave downwards for the first half and concave upwards for the second (see Fig. 8c). This results because of the shape of the Gaussian curve. The maximum deflection ratios were then determined for the hogging zone from point B towards point C, where the settlements become negligible, using the same approach. In fact for this exercise the deflection ratios were calculated by taking chords in the hogging zone beyond C, to the end of the building and from this it can be seen that the deflection ratio reaches a maximum in hogging before starting to diminish. Maximum values of deflection ratio are about 0.006% in sagging and hogging (negative) and are located at parts of the curve where there is maximum curvature.

Figure 7: Position of the Ritz Hotel in relation to the JLEP works and the existing tunnels.

The western end of the Piccadilly façade of the Ritz is about 80 m from the PIP tunnel and so should be well outside the zone of influence of the tunnelling. In viewing the settlement profiles in Figure 8a, it can be seen that there is a small amount of upward movement, evident in the vertical displacement profiles at all three levels, about 60 m from the end of the building, close to the western end. This could partly be due to rigid-body rotation of the building but also could indicate some upward flexure. Flexing of this sort was observed on some of the other buildings (e.g. longitudinally for the Treasury, see Viggiani & Standing,
2001). Any rigid-body rotation would need to be taken into account, in determining values of relative rotation $\beta$. This is one of the disadvantages of using $\beta$ as a damage parameter.

Hypothetical horizontal in-plane displacements along the building have been determined using the same Gaussian curve given in Figure 8b, assuming that the same settlements would have been observed at ground level and that the resultant vectors of movement at ground level are directed to a ‘point-sink’ located at the position of the PIP tunnel axis. It is important to realise that these hypothetical values are based on the assumption that there is no restraint between the ground and the foundation. This unrestrained profile is shown in Figure 8d. As would be expected, displacements are zero above the tunnel centre-line (or axis) because of symmetry and the maximum value coincides with the point of inflection.

Figure 8(a) to (c): Processed data from façade monitoring of the Ritz. (a) Vertical displacements along length of building measured on 17.10.1997 at three levels; (b) Gaussian curve fitted through data; (c) deflection ratios determined from fitted curve.
On the same figure the horizontal displacements from the total station monitoring are shown for the same survey of 17 October 1997. The horizontal movements cannot be determined as accurately as those in the vertical sense due largely to problems with instability of the reference targets. Moreover thermal and seasonal movements of buildings are generally much greater horizontally than vertically (for buildings that are longer than their height). For this reason the data once calculated, have been expressed relative to the movement of the far end of the building, which is outside the zone of influence of the tunnelling works and therefore assumed to be stable (i.e. the end at 72 m). The horizontal displacements at the lower two levels of targets can be seen to be essentially zero, being within about 1 mm of movement. At the upper level horizontal in-plane movements of up to about 2 mm develop towards the east, i.e. towards the PIP tunnel. These might be attributed to a component of the overall building movement caused by the vertical displacements showing that the building is experiencing some bending as well as vertical shearing. The measured in-plane displacements are very small compared with the ground displacements generated from the Gaussian settlement curve, which might be anticipated if the building had no stiffness and offered no restraint at foundation level.

Figure 8(d) and (e). Processing data from façade monitoring of the Ritz. (d) in-plane horizontal displacements and (e) horizontal strains in terms of hypothetical values determined from the fitted Gaussian curve and from field measurements.

A hypothetical in-plane horizontal strain profile has been generated from the unrestrained displacement profile (Fig. 13d) and is shown in Figure 13e compared with the strains determined from the total station measurements. Note that a structural engineering sign
convention has been adopted with positive values relating to extension. The strains are expressed in microstrain; note that the value separating negligible and very slight damage categories given in Table 2 is 500με. The compressive and extension strains in the sagging and hogging zones respectively are clearly defined from the generated curve. The strains in tension, from the generated curve, are all less than 400με, below the 500με value. The strains from the field observations are about an order of magnitude smaller than those from the hypothetical unrestrained profile. It might be noted that all the strains in the sagging zone are very slightly negative, as would be expected, but the values are very small and at the limit of the total station measuring accuracy.

3.2 The Treasury, Westminster

The Treasury is an extensive building just over 200 m long and about 100 m wide with its long southern façade facing Parliament Square and its shorter western side opposite the south-eastern corner of St James’s Park. The building was completed by about 1912 and is constructed from mass masonry with a concrete frame and slabs. It was one of the first concrete frame structures built in London. It was additionally heavily reinforced during the second-world war as it was used to house the Government’s cabinet war rooms. A view of the building and its plan layout are shown in Figures 9 and 10. There are four main floors above street level, and three below, the deepest being the sub-basement where the research monitoring was carried out. The foundations are thought to comprise a concrete slab and localised pad and strip footings.

The building was affected by the two running tunnels which pass beneath its south-western corner (Fig. 10). Tunnel excavation was by open-face shield with a back-hoe and lined with expanded precast concrete segments. After construction of the westbound tunnel, a shaft was sunk outside the Institution of Civil Engineers and a dense array of TAMs installed beneath the building. These were subsequently used for corrective and concurrent compensation grouting during the construction of the eastbound running tunnel. In this paper only the building movements from the westbound tunnel construction are presented, i.e. prior to any compensation grouting or effects from installation of the TAMs. Further details concerning the building and its response to these various other construction activities are given by Viggiani & Standing (2001).

Research monitoring at sub-basement level was by precise levelling and taping along most of the Horse Guards Road side and about half of the Great George Street length, i.e. roughly transverse and parallel to the westbound tunnel alignment. Details of these monitoring techniques and the processing of the data are given by Standing et al. (2001). The main focus here is on the measurements made in the transverse sense to the tunnel.

The progressive development of settlement as the tunnel approached the Horse Guards Road façade is shown in Figure 11a. It can be seen from the dates of surveying that the tunnel was constructed very rapidly beneath the building, at about 47 m/day. It was not possible to take measurements over the first 16m from the corner of the building, which was inaccessible, and so the profiles look very straight in this region. A distinct kink in the data occurs at a distance of about 25m, from survey set 5, with a much steeper settlement profile up to this point. It is possible that the part of the building that was additionally reinforced to protect the cabinet war rooms extended beyond this point.

Comparisons with greenfield measurements made very nearby at the St James’s Park reference site, indicate that the building response was much stiffer with a less curved settlement profile and maximum settlement of just under 15mm compared with the 20mm measured in the park (see Viggiani & Standing, 2001 for further details). Gaussian curves
have been fitted to the final two sets of the levelling data in the same way as described for the Ritz. These profiles are also included on Figure 11a. They highlight the kink in the settlement profile.

Figure 9: The south-west corner of the Treasury with Horse Guards Road façade on the left and Great George Street façade on the right.

Figure 10: General plan of the Treasury building (North upwards).
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The generated curve corresponding to the final survey (set 7) has been used to calculate a profile of deflection ratio along the transverse length of the building in the same manner as described for the Ritz. This is given in Figure 11b. The resulting profile is very similar to that for the Ritz, except magnitudes are slightly greater.

Precise taping measurements were made between the levelling points, using a digital tape extensometer. The horizontal strains along both the Great George Street and Horse Guards Road are plotted against time in Figures 11c and d, and the average temperature during the readings is also plotted against time for comparison in Figure 11e. The westbound tunnel influenced the transverse section (Fig. 11c) from the 12.04.1995, i.e. at a time of 9 days from the origin in these figures. There is no particular trend in the data and very little change in magnitude of strain (some very small strains developed just prior to this). The changes in strain with temperature, which does not change markedly, are more pronounced as can be seen from the data (Figs. 11c to e). It should be noted that the taping measurements have been corrected for temperature changes of the steel tape itself.

The measured horizontal strain along the transverse profile just above foundation level are plotted in Figure 11f (note that the set numbers referred to are different to those for precise levelling). A number of data sets are shown, from both before and after the tunnelling activity. They all lie within a very narrow scatter band of about $\pm 50\mu\varepsilon$ (it should be noted that the accuracy of measurement is estimated to be about $\pm 40\mu\varepsilon$). Profiles of hypothetical horizontal
strain derived from the curves fitted to the final two levelling profiles (Fig. 11a) have been determined in the same way as for the Ritz data and are also shown on Figure 11f. A similar comparison can be made as that for the Ritz. The unrestrained hypothetical horizontal strains are about an order of magnitude greater than the measured values. However, they are of a similar magnitude to those measured at the St James’s Park reference site.

Figure 11(c) to (e): Processed data from precise taping in the sub-basement of the Treasury. Horizontal strains for spans along (c) Great George Street and (d) Horse Guards Road and (e) average temperatures in both parts of the building.

Once again, as for the Ritz, the horizontal strains in the building, this time in the sub-basement close to foundation level, were found to be almost negligible. They are far more significantly controlled by temperature than running tunnel construction, and even then are much smaller than would be estimated considering hypothetical greenfield conditions.
Figure 11(f): Processed data from precise taping in sub-basement of the Treasury. Measured and hypothetical horizontal strains along Horse Guards Rd during construction of westbound running tunnel.

3.3 Elizabeth House, Waterloo

Elizabeth House is situated south of the river Thames and lies just behind Waterloo International Railway Terminal. It is a reinforced concrete frame building, constructed in the 1960s in three separate parts of ten and seven storeys and a tower block at the northern end. It is a long, narrow building with dimensions of about 200 m by 18 m in plan for most of its length. The seven- and ten-storey blocks have two basement levels and are founded on a monolithic 1.4 m thick raft. The two buildings are separated by a full-height expansion joint and there is also an expansion joint mid-way along the ten-storey block that extends from roof level to the first floor level. It was this latter block that was affected by the JLEP tunnelling works and which will be the focus in this paper. A photograph of the seven- and ten-storey blocks and a plan of the area are shown in Figures 12 and 13.

Initially, an access adit was driven to just below the south-western end of the building. Two running tunnels, of 5.6 m diameter, were constructed beneath the building some months later almost simultaneously using the Sprayed Concrete Lining method. They were constructed in an easterly direction and are at a skew to the building. This is significant when considering the profiles on the western side (York Road) compared with those on the eastern side (referred to as the Waterloo side). Just over a year after the running tunnels were complete, a cross-over passage, with maximum diameter of 12 m, was constructed between the two.

A range of monitoring systems was installed on, within and below Elizabeth House. In this paper, the results from precise levelling and taping and subsurface monitoring with in-place electrolevel devices are presented and discussed. The precise levelling and taping were carried out in the lower basement. Six boreholes were made through the basement slab, sets of rod extensometers were installed in five and electrolevel inclinometers were installed in the remaining borehole adjacent to the eastbound running tunnel. The expansion joints and a number of existing cracks in the basement were also monitored using a Demec gauge. Three photogrammetric surveys were made on the western facade of the building. These latter two monitoring activities are briefly referred to here. Further details concerning the building, monitoring and resulting data are given by Standing (2001).

The main focus of this section is on the measurements made and building response during the construction of the running tunnels.
Figure 12: Elizabeth House from York Road looking southwards at seven- and ten-storey buildings.

Figure 13: General plan of Waterloo area, showing Elizabeth House, the surrounding buildings and the JLE tunnels and other subsurface structures.
3.3.1 Analysis and discussion of data from precise levelling at foundation level

The precise levelling results are considered individually along the western (York Road) and eastern (Waterloo) sides of the building in Figures 14a and b respectively. In these figures the settlement profiles from a number of surveys are shown to illustrate the progressively developing troughs. The influence of the access adit is evident from the first data set (23.12.1994) shown from the western side (York Road) in Figure 14a. The westbound tunnel was constructed first and its face was just beyond the rear of the eastern side (Waterloo side) of the building by 06.02.1995. At this time the face of the eastbound tunnel, located further along the length of the building, was just in front of the York Road side (western side). It is evident from the two sets of profiles in Figures 14a and b that the position of the base of the troughs moves to the north (i.e. at increasing distances on the figures) and the troughs deepen and widen. The fact that the bases of the troughs are at different locations at the front and rear of the building indicate that the building is twisting. As the tunnels are within a short distance of one another (roughly three diameters axis-to-axis) the settlement troughs merge.
The settlement troughs exhibit marked curvature along the length of the building (the same was not true across its width) with maximum settlements of just over 17 mm and 13 mm and volume losses calculated as 1.3% and 1.0% at the front (west face) and rear (east face) respectively, by the time the eastbound tunnel face was well clear of the rear of the building.

The expansion joint separating the seven- and ten-storey blocks is located at a distance of about 110 m from the southern end of the building. On the York Road side there is evidence of a small amount of upward movement of the ten-storey block in the vicinity of the joint. This on first sight appears to be rigid-body rotation, but close inspection reveals that the other end of the block also rises by a similar amount. It is very likely that this mechanism results from inward in-plane horizontal displacements towards the central expansion joint in the ten-storey block.

The measured settlement profiles from 08.02.1995 and 06.03.1995 have been used to determine profiles of deflection ratio. These are given in Figures 15a to d. The profiles are different to those of the Ritz and Treasury as they are generated directly from the levelling data rather than from curves fitted through them, consequently they have a more irregular form. However, the general trends in the profiles are similar to those from the smooth hypothetical curves. In terms of deflection ratio $\Delta L / L$ (Figs. 15c and d) for the first, westbound, tunnel the magnitude at the western, York Road, side is almost 0.012%, twice that at the eastern, Waterloo, side of the building, but much smaller than typical cut-off values between negligible and very slight damage categories. They are also of similar magnitude to those
determined for the Ritz and Treasury. After the eastbound tunnel construction, both profiles (front and rear) are of similar shape and maximum magnitude (just under 0.010%) but at a small offset to each other because of the relative position of the tunnels.

Precise levelling data from along the western, York Road, side of the building for the 15-month quiet period after completion of the running tunnels and during the cross-over passage construction are shown in Figures 16a and b. The profiles relating to consolidation settlement (Fig. 16a) are essentially parallel and show that the trough deepens and widens, with maximum settlements increasing from about 17 to 22 mm. Cross-over passage construction took place as a series of complex excavation sequences. This is evident from the shape and manner of movement of the profiles. At some points (e.g. 30.07.1996) parts of the building are rising, towards grid line 1, while others are settling, providing once again evidence of its flexure and the influence of the expansion joint midway along the ten-storey block. By the end of cross-over construction, settlements had increased to a maximum of about 37 mm.

Figure 16. Precise levelling data from the western (York Road) side of Elizabeth House during (a) 15-month period following running tunnel construction and (b) construction cross-over passage.
3.3.2 Analysis and discussion of the data from precise taping at foundation level

Tape extensometer measurements were made in the lower basement in the same manner as for the Treasury building, i.e. between the points used for precise levelling, both longitudinally and across the width of the building. In Figure 17a the strains determined over roughly a two-year period are shown for a number of spans running longitudinally along a large part of the ten-storey block. Strains can be compared with changes in temperature shown in Figure 17b from which it is evident, as was the case at the Treasury, that they are primarily controlled by temperature rather than tunnelling activities (the times when the west- and eastbound running tunnels were constructed are shown). The building can be seen to contract as the temperature reduces, although there is generally a time-lag between ambient temperature change and building response. More detailed plots from the beginning of the period, when many measurements were made are shown in Figures 18a to d, where sets of data from longitudinal and transverse spans are shown for the first 200 days of monitoring. The delay in building response can be better seen in these figures. From mid-October 1994 to January 1995 the drop in temperature of about 8-9°C resulted in a compressive strain of about 50 με. Demec gauge measurements made across the expansion joint between the ten- and seven-storey blocks confirmed this value. Strains of about twice this value would have been expected for a typical coefficient of thermal expansion for concrete. Possibly they did not develop because the underside of the slab is in contact with the ground, which changes temperature even more slowly than the building or because of frictional restraint between the ground and the raft foundation.

Figure 17: Precise taping data versus time for various spans along the eastern (Waterloo) side of the building: (a) changes in span expressed as strains (after correcting for tape temperature changes and tape breakages); (b) changes in temperature over spans.

The horizontal strains at slab level that developed along the length of the building during the construction of the two running tunnels have been isolated by careful selection of a set of base readings, chosen as those on 17/01/1995. By choosing this date as a datum, the
strains caused by thermal changes can be assumed to be almost negligible. The settlements, corresponding strains and changes in temperature are plotted against distance from the southern end of the building for the westbound and eastbound tunnels in parts (a) and (b) of Figure 19 respectively. The data are from the Waterloo side of the building. During construction of the westbound tunnel, settlements developed up to about 4 mm at 40 m from the end of the building (Fig. 14b). This part of the settlement trough corresponds roughly to that of the hogging region and this length of the building correspondingly developed tensile strains to a maximum of about 30 $\mu$e. Changes in temperature during this time can be seen from the lower plot to be within $\pm$ 1 deg.C.

During construction of the eastbound running tunnel, following straight on after the westbound, settlements increased to 14 mm with the base of the settlement trough lying at about 45 m from the end of the building. This resulted in a roughly contemporaneous development of compressive strains with a maximum incremental value of about 50 $\mu$e beneath the base of the settlement trough, diminishing with distance from the tunnel. By the end of the period there were no tensile strains remaining. Again changes in temperature during this time were within $\pm$ 1 deg.C.

![Figure 18: Precise taping data versus time for first 200 days after start of monitoring: (a) changes in span expressed as strain for various spans along the eastern (Waterloo) side of the building; (b) changes in temperature over these spans; (c) and (d) similar plots but for central part of transverse spans.](image)

3.3.3 Analysis and discussion of the subsurface displacement data

Inclinometer tubing was grouted into a borehole located at about one tunnel diameter distance from the eastbound running tunnel and a string of fourteen electrolevel inclinometers installed within it (see Standing, 2001, for further details). Electrolevel devices enable measurements of change in rotation to be made which can then be integrated to obtain horizontal displacements. Profiles of horizontal displacement with depth measured in borehole 1 at different time intervals during construction of the west- and eastbound running tunnels are shown in Figure 20.
Figure 19: Precise levelling and taping data: settlements, horizontal strains and temperature changes in spans along inner Waterloo (eastern) side of Elizabeth house with taping data referenced to 17.01.1995 for construction of (a) westbound and (b) eastbound running tunnels.
The running tunnel construction is estimated to have had most influence on the inclinometer during the period 17.02.1995 to 27.02.1995. The profiles from 09.02 to 15.02.1995 indicate small displacements of up to about 2 mm moving away from the tunnel axis position just prior to its arrival in line with the borehole. Above this level, small displacements directed horizontally towards the tunnel develop uniformly, increasing with elevation, to a value of about 3 mm just beneath slab level. The horizontal displacement profiles from 22.02 to 26.02.1995 are significant. They show a continued increase in horizontal movement with elevation, but over the uppermost 5 m of the ground immediately beneath the slab there is a reversal in the shape of the profiles. This reversal or reduction in the displacements almost certainly results from the interaction between the ground and the underside of the slab. The slab is providing frictional restraint and its axial rigidity impedes lateral ground movement. The maximum horizontal movement at this time is just over 7 mm, but this reduces to less than 4 mm immediately beneath the slab. The lines of horizontal displacement with depth can be extrapolated linearly from the point where the reversal in profiles starts (i.e. 10 m...
depth). This allows an estimate to be made of the horizontal movements at foundation level that would have occurred if interaction with the slab had not inhibited them. The exercise suggests that horizontal movements of up to about 10 mm might have taken place (see broken line in Fig. 20). Similar profiles of horizontal displacement with depth for boreholes close to the running tunnels at the St James’s Park control site are shown in Figure 21. Although tunnelling methods and boreholes offsets are different for the two sites it can be seen that at the greenfield site, where there is no surface restraint, the displacement profiles do not curve back as they do beneath Elizabeth House.

![Graph](image1.png)

**Figure 21:** Profiles of horizontal displacement with depth measured during construction of running tunnels at St James’s Park control site using in-place electrolevel inclinometers adjacent to (a) westbound and (b) eastbound tunnels.

It is instructive to compare the horizontal ground strains with those determined from the St. James's Park greenfield reference site and also those of the building, as measured in the sub-basement using the tape extensometer. It has been estimated (Standing, 2001) that the borehole is about 8.5 m from the centre-line of the eastbound tunnel, i.e. within the compressive zone of the settlement trough. If it is assumed that horizontal displacements above the tunnel centre-line are zero because of symmetry, an estimate of the compressive strains between the position of borehole 1 and the tunnel centre-line can be made. This method is not accurate, especially in view of the skew of the tunnel, but yields an average strain of about 1200 με (determined from a displacement of 10mm over 8.5 m).

The displacement and estimated strain determined for the ground immediately below the Elizabeth House foundation slab compare well with values obtained for the St. James's Park greenfield reference site. Displacements measured at the surface there after the passage of only the westbound running tunnel were about 7 mm from the tunnel centre-line to the point of inflection with a maximum horizontal strain of about 1000 με (see Nyren et al. 2001).
Movements at Elizabeth House might be expected to be larger because the westbound tunnel would have disturbed the ground to some degree prior to the eastbound tunnel being constructed.

In comparing the horizontal strains measured in the sub-basement with those determined from the ground beneath it, the results indicate that only about a tenth of the horizontal strain generated in the ground from tunnelling are transmitted to the building. Two conclusions can be drawn from these measurements.

- The building to some degree inhibits tunnelling-induced horizontal strains in the ground from developing, evidenced from the reversal of the horizontal displacement profile shown in Figure 20.
- The ground strains that are developed are comparable to those determined from the greenfield reference site but the tape extensometer results indicate that they are not transmitted through the slab to the building.

4 SUMMARY AND CONCLUSIONS

A number of traditional monitoring techniques have been described and details given concerning how to achieve good accuracy etc. This is often required when monitoring tunnelling-induced ground and structural movements where small displacements and strains can be critical with respect to potential building damage.

Three case studies from the JLE project in London have been presented to illustrate a range of monitoring data. These three cases represent different structural forms and foundations types and two different tunnelling methods were used. The influence of building stiffness and soil-structure interaction has clearly been shown from these. Also detailed comparisons have been made between the measurements and empirical methods of (greenfield) prediction that are frequently adopted. It is seen that using a Gaussian form gives a good fit to the monitoring data as long as the correct volume loss and trough width parameter are used.

Careful assessments of horizontal strains in the buildings have been made using measurements taken on the façade of the Ritz and within the basements of the Treasury and Elizabeth House. In these cases the strains that developed in the buildings were much smaller than would be estimated using greenfield predictions and a point-sink assumption. The magnitude of strains is just about measurable with the techniques used (total station façade monitoring and precise taping). Thermally-induced horizontal strains were found to be far more significant than those caused by construction activities. In the case of Elizabeth House, it is just possible to observe the compressive and tensile strains generated at foundation level from the sagging and hogging components of the settlement trough. The magnitude of these strains is less than 50με. However, measurements made in the ground beneath the raft foundation of Elizabeth House indicate strains of about 1200 με, similar to those measured in a greenfield condition at St James’s Park. Ground strains are not transmitted to the foundation because they are partly inhibited by frictional restraint and because of the stiffness of the slab.

The absence of significant horizontal strain observed in the buildings is of particular importance, because this has a major effect on current assessment methods that assume buildings deform under Greenfield conditions. If in fact little or no horizontal strain is induced in the building, the tensile strain (and hence degree of damage) may well be very significantly over-estimated. However, it is important to note that the three case studies analysed have
integral foundations (e.g. rafts, or pads and strips connected by a continuous slab). If a structure has discrete isolated footings, it may suffer damage, even if not indicated by predictions (Powderham et al., 2004).

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1 INTRODUCTION

Monitoring of ground movements around tunnels and excavations on the Washington Metro led to development of procedures for assessing ground loss or movements at the boundaries of the excavation or tunnel and the distribution of movements through the soil mass to the ground surface and to adjacent structures (Cording & Hansmire, 1975; O'Rourke & Cording, 1974). As the field investigations progressed in Washington, instrumentation and observations were concentrated on the effect of ground movements on structures (Boscardin & Cording, 1989). More recently, a research program consisting of numerical and model studies correlated with field observations was conducted to assess the relation of building distortion and damage to excavation-induced ground movements (Son, 2003; Laefer, 2001; Ghahreman, 2004).

This paper provides examples of building damage and distortion resulting from excavation or tunneling and evaluates the behavior of the buildings using methods developed in previous studies. In many cases, the investigations were begun after the damage was observed in order to determine mitigation and repair procedures or in order to assess the causes of, and responsibility for, the damage. In most cases, some pre-existing settlement data was available, and, in some cases, pre-condition surveys were available. Most importantly, the structures themselves served as indicators of the type and causes of distortions and damage that were imposed on them. The ability to observe and read the building response is aided by an understanding of the chain of relationships that extends from the excavation or tunnel and adjacent ground loss or ground movement, to the distribution of ground movement and volume change through the soil mass, to the interaction of the ground with the building, and then to the building distortion and damage. To properly assess the building behavior it is necessary to understand not only the ground movement patterns but also the building characteristics.

Cases include buildings where (1) the damage is clearly a result of the excavation or tunneling and there is no other cause of significant building damage, (2) the distortion and damage due to tunneling or excavation is imposed on and affected by pre-existing weaknesses or deterioration in the structure, (3) the distortion and damage due to tunneling or excavation can be separated from pre-existing damage and deterioration in the structure.

The buildings are on shallow foundations in U.S. cities. Most are masonry bearing wall structures built in the 1800s or early 1900s.
2 REVIEW OF GROUND MOVEMENT AND BUILDING DISTORTION

2.1 Sources of ground loss

Sources of ground loss around a shield tunnel are related to (1) the volume loss into the tunnel face, which may be sudden and large if face stability is not controlled, and to soil filling annular voids around the shield and tail of the shield.

2.2 Volume loss, volume change and surface settlement volume

Once the ground loss at the excavation or tunnel is estimated, the volume of the surface settlement trough can be determined. For soft clays, the volume of the surface settlement is approximately equal to or slightly greater than the volume of ground loss as the shield passes. With time, as the clay around the tunnel consolidates due to disturbance, increase in mean stress, or drainage into the tunnel, additional surface settlements will occur, with a wider settlement trough that is influenced by the size of the consolidating zone around the tunnel. As described in Case 2, for the Chicago clay, consolidation developed in a zone extending approximately 4 radii from the center of the tunnel.

For medium to very dense sands, the volume of the surface settlement is less than the volume loss. For loose sands, such as fill around utilities, the volume of the surface settlement may be greater than the volume loss. For twin tunnels, the loosened zone above the first tunnel will be recompressed by the second tunnel so that the total surface settlement volume approaches the volume loss for the two tunnels.

2.3 Distribution of settlements

The magnitude of the settlements can be found once the volume, shape and width of the settlement trough is determined. A Gaussian distribution provides a reasonable fit to observed settlements above a tunnel. The half width, w, of the settlement trough can be estimated by using the angle from the vertical, b, of a line extended from the springline of the tunnel to the ground surface. The angle b is typically in the range of 20 to 30 degrees for tunnels in sand and 35 to 45 degrees for the short term ground movements around tunnels in clay. Maximum settlement, \( \delta_{\text{max}} \), and average settlement slope, \( \delta_{\text{max}}/w \), can then be obtained (Fig. 1).

For a braced excavation, a parabola provides a reasonable distribution for settlements adjacent to the excavation, although the displacements will decrease close to an excavation wall that has good bearing and settles less than the adjacent ground. For excavations in sand, the parabola extends laterally a distance of approximately 2 times the excavation depth and most of the displacement is concentrated in a zone extending laterally 1.5 times the excavation depth.

As the tunnel advances, a moving settlement wave advances ahead of the tunnel, so that structures in the path of the tunnel are impacted by a longitudinal settlement profile that begins at an angle \( \beta \) ahead of the face and has a slope that depends on the distribution of ground losses along the length of the tunnel shield and tail, but typically has a maximum slope approximately equal to the average slope, \( \delta_{\text{max}}/w \), of the settlement cross-section. Thus, although the portion of a building located above the tunnel, in the center of the settlement trough, will be in a zone of lateral compression and sagging (concave) settlement, the advancing longitudinal settlement profile will create a moving zone of lateral extension and hogging
(convex) shaped settlement beneath the structure, that can cause cracking due to shear and extension in walls oriented parallel to the tunnel. In the case of the Evanston tunnel, over half of the ground settlement was caused by long term consolidation of the clay so that the longitudinal strains were not large and cracks did not develop on walls oriented parallel to the tunnel.

Similarly, a moving ground settlement profile may advance away from a braced excavation as it is deepened. Thus, an adjacent building is first impacted by smaller displacements, concentrated nearer the excavation, which may have a sufficient slope to initiate cracking in the portion of the building close to the excavation. At this time, the remainder of the building is likely to be outside the zone of movement so that tilt is small and most of the settlement slope results in angular distortion. With further extension of the settlement profile up to and beyond the full width of the structure, displacements will increase but an increasing proportion of the settlement slope will be comprised of tilt, so that the ratio of angular distortion to settlement slopes drops.

Numerical modeling of distortion and cracking in brick bearing walls has shown the strain-path dependence of the cracking. When a settlement wave was advanced across the base of the structure, the damage in the later stages tended to concentrate on the cracks formed in the early stages in the portion of the bearing wall nearest the excavation so that the pattern of cracking was different than the cases in which the final settlement profile was applied, without a progressive movement across the structure (Son, 2003).

In several projects, large ground movements did not develop until the excavation approached full depth. In these cases, weak layers near or below the base of the excavation caused deep seated movement that developed ground cracks and displacement at distances behind the excavation of 1 to 2.5 times the depth of the excavation. Movements and cracking were sudden, rather than progressive, and a pattern of displacement and cracking near the wall of the excavation did not develop.
2.4 Lateral displacements

For a tunnel, lateral displacements at the ground surface will be largest at the point of inflection of the settlement trough, \( i = 0.4 \, w \). Beyond the point of inflection, lateral strains are in extension, and can be estimated using the relationship shown in Figure 2.

\[
\text{Maximum lateral displacement, } \delta_L \\
@ \text{Inflection point, } i : \\
\delta_L = \frac{1}{3} \delta_w (\tan b / 0.5)
\]

Average tensile strain, \( i \) to 2.5 \( i \) :

\[
\varepsilon_t = \frac{\delta_L}{0.6 \, w} \\
\varepsilon_t = \frac{1}{3} \delta_w (\tan b / 0.5) / 0.6 \, w
\]

\[
\varepsilon_t = 1.1 \left( \frac{\delta_w}{w} \right) \tan b
\]

Figure 2: Estimating lateral displacements due to tunneling.

The lateral strain is proportional to the maximum settlement and increases with increasing \( b \). For an excavation, lateral displacement of the ground surface will be largest for cantilever deflection of the excavation wall (on the order of 1 to 1.5 times the vertical displacement) and the lateral displacement at the ground surface will be of the order of 0.5 to 1.0 times the vertical for the bulging displacements occurring below strut and tieback levels (Milligan, 1974).

2.5 Long term displacements

Long term volume change (consolidation) occurs in clay surrounding the tunnel due to (1) disturbance of the clay around the tunnel perimeter, (2) increase in the mean stress in the ground due to the pressure applied during shoving of the shield and expansion or grouting of the lining, and (3) drainage into the tunnel reducing porewater pressures below the original ambient porewater pressures. The volume change results in surface settlements that typically are wider and flatter than the initial settlement trough created by ground loss into the tunnel. Case 2, the Evanston tunnels at a depth of 18 m, had consolidation resulting from both increases in mean stress and drainage into the tunnel. Over one half the surface settlement was caused by the long term settlement.

2.6 Damage criterion for assessing building distortion and damage.

The damage criterion of Figure 3 relates damage levels to the angular distortion and the lateral strain within distorting portions of the building. The equation for the state of strain at a point has been used to obtain the principal extension strain in the structure from the combination of angular distortion and lateral strain (Cording, et al, 2001). The principal extension strain boundaries shown in the plot of angular distortion vs lateral strain in Figure 3 have been...
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adjusted slightly from that developed by Boscardin & Cording, 1989, in order to fit them to the relationship for the state of strain at a point. (Boscardin & Cording, 1989, described the relationship in terms of the distortion of a deep beam for the case of a length/height ratio, L/H, equal to one.)

![Diagram](https://example.com/diagram.png)

**Figure 3:** Damage criterion based on lateral strain and angular distortion (Boscardin & Cording, 1989).

In fact, with slight modification, the relationship has been expressed in terms of the state of strain at a point, as shown in Figure 3, and has broad application to the distortion of building walls over a wide range of L/H ratios. Each of the boundaries between damage levels (negligible, very slight, slight, moderate, and severe) represents a constant value of the principal extension strain. The description of the damage levels was developed by Burland et al (1977). The impact of a given distortion level and cracking will differ for different buildings, and depending on the details of their sensitivity and significance, and they should be evaluated on a case by case basis.

The state of strain within a point can be used to describe the average strain within a structural element or bay. The structural elements are strained by the ground movements acting along the base. The angular distortion, or shear strain, is equal to the average settlement slope across the structural element minus the tilt (Fig. 4). The lateral strain at the base is equal to the extension of the base divided by the base length. These two values represent a single point on the plot of lateral strain vs. angular distortion. A separate measurement can be made of the lateral strain at the top of the element.

The lateral strain at the top may increase due to a convex (hogging) soil profile. Bending effects can occur in the central portion of a structure with low height/width ratios impacted by a wide
settlement profile. In this case, the distortion level includes the lateral strain at the top due to bending, as well as any shear distortions. However, for most of the observed settlement profiles, the portions of a structure impacted by ground movement have a relatively high height/width ratio and a low effective shear stiffness that they act as a deep beam. In such cases, significant bending and extension cannot develop at the top, near the end of the building, unless there are weaknesses or cracks extending up the wall, such as are formed by the junction between façade and bearing wall or by cracks that originally formed and extended because of the angular distortion of the wall (Fig. 5). Joists and roof structures or cross walls perpendicular to the displacing wall may reduce the opening of cracks in the upper floor levels.

\[
\beta = \frac{\delta V}{L} = \text{Slope} - \text{Tilt}
\]

\[
\varepsilon_L = \frac{\delta L}{L}
\]

**Figure 4:** Distortion and strains imposed on a structural element by ground movement.

1. Cracks develop due to angular distortion and lateral strain
2. Cracks extend upward
3. Plane of weakness is formed that allows bending and opening of cracks at top

**Figure 5:** Progression of angular distortion and lateral strain.

### 2.7 Effect of building stiffness on reducing distortions and strains

In cases where the building is relatively stiff, the green-field ground movements will be modified, and the distortions of the structure will be less than those estimated assuming the structure conforms to the shape of the green-field settlement profile. Boscardin & Cording (1989) show a relationship between the axial stiffness of grade beams within a structure and the reduction in
lateral building strain ($\varepsilon_h$) from the green-field lateral ground strain ($\varepsilon_{hg}$), where $E_gA$ is the stiffness and area of the grade beam foundation, $E_s$ is the soil stiffness, $H$ is the height of excavation or the length of the section of the foundation being strained, and $S$ is the spacing between grade beams (Fig. 6). Large reductions in lateral strain result if the foundation has grade beams, reinforced wall footings, or structural slabs.

Several cases are presented in this paper in which there are no grade beams or structural slabs in the basement floor, but the main floor and upper floors are concrete and tied into the bearing walls. In these cases, lateral extension and cracking may be present at the basement or first floor level, but wall cracks narrow and close as they extend to the level of the concrete floors. Vertical cracks extending higher may be limited to construction joints or weak zones within the structure, such as stair wells or boundaries between buildings.

Figure 7 shows the relationship between the shear stiffness of a masonry bearing wall building and the angular distortion, $\beta$, with respect to the change in ground slope, $\Delta GS$, between adjacent structural units (Son, 2003). The relationship was developed from a series of parametric analyses using the distinct element code, UDEC, in which the stiffness of the masonry blocks and the shear and normal stiffness of the mortar joints is modeled, and the joints can separate and slide when the stresses reach the strength of the mortar.

![Figure 6: Effect of axial stiffness of foundation on lateral strain in structure (Boscardin & Cording, 1989).](image)

The effective shear stiffness, $G$, to be entered in the calculation of the relative shear stiffness in the abscissa of the plot, is a reduced value that takes into account the presence of window penetrations in the wall. The ratio of angular distortion to the change in ground slope, $B/\Delta GS$, increases approximately linearly with the logarithm of relative soil/wall stiffness. However, as strains increase, with respect to the tensile cracking strain of the mortar (represented by $DGS/\varepsilon_t$), and cracks between the masonry blocks extend and enlarge, the ratio of angular distortion to the change in ground slope, $B/\Delta GS$, increases dramatically and approaches one. The downdrag of the façade (FD) on the brick wall causes the ratio to increase from the case of no façade downdrag (NFD).
3 TYPES OF STRUCTURES

Most of the cases described in this paper are masonry bearing wall structures on shallow wall foundations in urban areas of the United States. Most were built in the 1800s and early 1900s.

Typically, in the early to mid-1800s, the multiple wyths of the brick walls, including the façade walls, were tied together with a combination of headers and stretchers. In the late 1800’s and early 1900’s, almost all bearing wall structures had alley, foundation, and interior walls of common bricks consisting of two or more wyths of stretchers tied together every 6 rows with a row of headers, whereas the building facades were faced with a single wyth of a running bond (stretchers, no headers) with common brick behind. Over the long term, the façade walls tend to be more susceptible to cracking or displacement due to deterioration and environmental effects than the common brick walls.

Brick-bearing wall townhouses typically have bearing walls perpendicular to the street with timber joists extending between bearing walls, with spans of approximately 6 m between adjacent brick bearing walls. The joists are seated in pockets, usually one wyth wide. The end of the joist may have a tapered (fire) cut (shorter joist length at the top) that allows it to fall out of the wall more easily if it burns in a fire and not cause the wall to collapse. The joists set on the seat, they are not tied to the wall.

Houses with wider spans between exterior brick bearing walls had intermediate timber bearing walls. Floors may have sagged inward toward the timber bearing wall because of shrinkage of the timber bearing wall after it was installed. This is the case for several of the historic houses in Washington DC, built in the early 1800’s. Door frames can be observed to have undergone shear distortion which is down toward the center of the building.

Commercial brick bearing wall structures have timber beams and posts supporting timber floor joists between brick bearing walls.
Monumental structures, such as the Masonic Temple built in 1870 in Philadelphia, had masonry bearing walls and floors consisting of I beams with jacked arches of brick between the beams.

In the early 1900s, many of the larger brick bearing wall structures, had cast reinforced concrete floors formed by pouring a T beam -- a concrete slab and beam formed by placing clay tile in the form to fill the space between concrete beams. The concrete floors, unlike the timber joists, were tied to the masonry walls.

4 EXAMPLES OF BUILDING RESPONSE: BRICK BEARING WALLS PARALLEL TO EXCAVATION

For bearing walls parallel to an excavation or tunnel, the wall distortions largely occur on the cross walls or side walls, and the relative stiffness relationships apply to those walls. Between the brick side walls, the stiffness of the cross-section will be controlled by the floor stiffnesses and their connections to the bearing walls, and to the infills between floors.

The other issue for brick bearing walls constructed in the 1800’s is displacement of the floor joists out of their seats, reducing the bearing area to the point that the joist falls and the floor collapses.

In the early 1900’s, concrete floors were used in many brick bearing wall structures, providing a connection to the masonry bearing wall. These frames are subject to sidesway.

4.1 Distortion of two brick bearing wall buildings on G-1 tunnel alignment

After the initial program of measurement of greenfield movements around shield tunnels, the Washington, D.C. Metro investigations continued with observations of the impact of ground movements on the distortion and damage to buildings located near tunnels and excavations. Most of the buildings were old, and sometimes historic, and were constructed with masonry bearing walls on shallow foundations. The structures were not reinforced and were sensitive to both lateral and vertical ground displacements. During this period, a pair of unoccupied buildings along the Washington Metro G-1 tunnel alignment, south of the Anacostia River, was instrumented with tape extensometers extending horizontally and diagonally at each floor in the structure, as well as tiltmeters on the bearing wall of the building (Fig. 8). Building 1 was located in a zone with relatively small lateral strain. Building 1 is separated from Building 2 so that most of its movement was tilt. Extension of the diagonals showed that a very small shear displacement took place in Building 1. Building 2 was located in the outer portion of the settlement profile. Small lateral strains developed at foundation level, and increased in upper floors because of formation of an open vertical crack between the bearing wall nearest Building 1 and the facade. Small displacement of the joists in their seats would have occurred as a result of the opening of this crack.

The building was modeled numerically using a distinct element analysis. The same settlement and lateral displacement profile was applied as was observed in the field, showed close correlation with the building distortions (Son, 2003). Opening of a vertical crack at the bearing wall of building 2 was achieved in the discrete element analysis when the floor loads in building 2 were reduced to the weight of the flooring and joists alone, with no live loads. This was the
condition existing in the building, because the rooms were empty. When the analysis was first run, live loads of 10 psf were applied to the floors of building No. 2, and there was no opening of a crack between the bearing wall and the facade in the upper part of the building.

![Diagram of tilt and angular distortion](image)

Figure 8: Distortion of two 2-story buildings, bearing walls parallel to tunnel, G-1 tunnel, Washington Metro.

4.2 Distortion and damage to brick bearing wall structure with concrete floors, Evanston, Illinois.

Tunneling was carried out beneath an EW street in Evanston, Illinois, adjacent to a brick bearing wall structure with concrete floors, built in the early 1900’s. The bearing walls of the building are parallel to the tunnel (Fig. 9). The tunnel was 3.6 m in diameter, was advanced with a shield with a wheel excavator, and was supported with 100 mm steel ribs spaced 1.2 m on center and timber lagging, with a final cast concrete lining. The tunnel was located at a depth of 17 meters in a soft to medium strength clay, which was deposited during late glacial stages in Lake
Chicago (an extension of Lake Michigan that covered most of Chicago and Evanston) and is often referred to as the Chicago Clay, the medium in which the Chicago Subway was built in 1938-1941 (Terzaghi, 1942a, 1942b).

Ground movement patterns and settlements typical of this reach of the tunnel alignment were obtained at an instrumentation test section (TS-3) located approximately 300 m to the west of the building, which was installed to monitor movements and pore-pressures in the soft to medium strength clays prior to passing under a railroad line (Srisirirujakanakorn, 2005). The consistent tunneling method, similarity of the soil conditions, and similarity of movement patterns in this reach of the tunnel justifies using TS-3 settlement data for estimating the green-field ground movements at the building. The immediate and long term settlements at test section 3 are shown in Figures 10 and 11, respectively.

At TS-3, immediate ground settlement at tunnel centerline was 30 mm, and the settlement zone extended from tunnel springline to the surface at an angle, b, of 38°, giving a half width, w, of the settlement trough of 19 m from tunnel centerline. (The half width, w, is determined as the location where a triangular settlement distribution of the same volume as the surface settlement volume, would intersect the surface, and w is 2.5 i, where i is the inflection point.)
Additional long term ground settlement of 34 mm was measured at TS-3 over a period of 445 days. The two primary causes of the consolidation resulted from consolidation of the Chicago clay around the tunnel. The two primary causes of the consolidation were (1) drainage into the tunnel, through the relatively permeable initial lining of steel ribs and timber lagging, and (2) a permanent increase in the mean stress in the clay surrounding the tunnel caused by the pressures applied to the ground as the shield was shoved forward and as the lining was expanded against the ground behind the tail of the shield. The increase in mean stress was initially evidenced by the development of excess pore pressures in the clay surrounding the tunnel, in a zone
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Concentrated within approximately 4 radii of tunnel centerline. Dissipation of the excess pore pressures and further reduction of pressure due to drainage over the following year resulted in the consolidation. The half width of the settlement trough was 23 m for the long term settlements, which was wider than the settlement trough that developed during tunneling because the zone of consolidation around the tunnel is wider than the zone of ground loss into the tunnel. (The immediate settlements as the shield passes and long term settlements due to consolidation should be evaluated separately in order to properly assess their zones of influence).

The South Wing of the building is located in the tension zone of the settlement trough. The green-field ground surface settlement at tunnel centerline is estimated from TS-3 data to be 75 mm at tunnel centerline and 27 mm at the location of the South bearing wall, which is close to the measured settlement of 30 mm for the South bearing wall at the east end of the building. Lateral ground displacement at the edge of the building was estimated from test section measurements to be 29 mm (Fig. 12).

Figure 12: Profile at Section A, East façade wall.

Figures 12 and 13 show the pattern of displacements and the crack damage at Section A on the east facade wall resulting from tunneling. Diagonal shear cracks developed at ground level and above the windows closest to the South wall. Damage to plaster ceilings also took place in the 3rd floor room near the South wall. Further from the South wall, cracking due to lateral extension could be observed in the basement floor and walls. A vertical tension crack was located outside...
the zone of lateral greenfield displacements because the stiffness of the floors in the South Wing caused the tension crack to extend to the boundary between the South and Central wings. Cracks opened a total of approximately 17 mm. The lateral extension did not extend above the basement level because of the stiffness of the concrete floors.

### Figure 13: Location of cracks on East Façade, Section A, Evanston, Illinois.

Approximately 5 to 20 mm of outward displacement of the brick was present above many of the lintels on the façades (Fig. 13). Further from the tunnel, in the courtyard, the façades had not been video-taped in the pre-construction survey and it was being alleged that the displacements were caused by tunneling. Reports were discovered indicating that recommendations for repair of the cracks had been made several years prior to tunneling. Furthermore, the outward displacement above the lintels was not compatible with the pattern and location of ground movements due to tunneling.

Figure 14 shows the profile of the building at Section B. In this section, the building is narrower and therefore most of the ground movement resulted in tilt and translation of the building with little distortion. Damage was limited to one or two hairline cracks.

### 4.3 Case 3: Concentrated settlement due to trench excavation adjacent to gymnasium with brick bearing walls and concrete floor

Settlement of bearing walls parallel to an excavation can induce shear cracks in attached cross walls and end walls, as illustrated in the case of a brick bearing wall building with gymnasiums on two levels (Fig. 15). Settlement of the bearing wall was only 22 to 31 mm due to a small
adjacent trench that was mistakenly excavated along the length of the wall in stiff clays below the footing level. The settlements caused large angular distortion and damage that was concentrated over a short distance of approximately 3 m along the cross walls and end wall.

Figure 14: Tilt and Translation at Section B, Evanston.

Because the floor between the two gymnasiums was concrete and tied to the bearing wall it did not separate from the wall. Outward displacement of the bearing wall at footing level caused the wall to rotate about the concrete floor level between the two gymnasiums and produced a minor horizontal tension crack on the inside of the bearing wall in the upper gymnasium, approximately 2 m above the level of the concrete floor.

4.4 Case 4: Distortion and damage to a 3 story apartment building with timber floor joists. Bearing wall parallel to excavation, loss of bearing of floor joists

A major concern for bearing walls parallel to an excavation is illustrated in Figures 16 and 17 where settlement and lateral displacement in the range of 40 to 75 mm occurred along the bearing wall. The lateral displacement at the building foundation wall caused loss of 58 mm of joist bearing so that the floor joists on all floors and at the roof had to be temporarily supported with posts and beams. The building was subsequently demolished. The average lateral strain between adjacent bearing walls exceeded 1/100, in the severe to very severe damage range.
(a) Case 3: Settlement of bearing wall of Gymnasium.

Figure 15: (b) Case 3: Shear cracks in cross wall due to settlement of bearing wall.
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Figure 16: Case 4: Distortion and damage to apartment building.

Figure 17: Case 4: Plan and profile of distortion and damage.
57 mm reduction in joist bearing

57 mm reduction in joist bearing

Figure 18: Case 4: Outward displacement of wall by 57 mm reduces joist bearing.

5 LATERAL SEPARATION OF WALLS AT UPPER FLOOR LEVELS

Tunneling beneath 7th St with an open face shield on the first phase of the Washington Metro project in the early 1970’s resulted in ground loss into the face and surface settlements on the order of 50 mm of the façade wall of a brick bearing wall building. The settlement resulted in diagonal shear cracks between windows located on the bearing wall adjacent to the façade, as well as distortion of the window frame (Figs. 19 and 20). The shear cracks extended almost to the roof of the four story building and separated the façade wall from the bearing wall, causing an outward lateral displacement of 28 mm near the top of the façade wall. Other buildings along the street were similarly affected and facades were temporarily braced and tied back to prevent their collapse.

Independently, in the last few years, a series of distinct element analyses using UDEC were performed in which four-story buildings on elastic foundations were subjected to settlement and lateral displacement patterns. Individual bricks were modeled, assigning stiffness and strength values to the mortar between bricks. With shear distortion and lateral strain, the mortar was capable of cracking and the individual bricks could displace. As the settlements increased, the shear cracks widened and propagated upward between window levels until a continuous series of shear cracks between windows caused separation of the façade from the bearing wall and allowed outward lateral displacement of the façade wall. Settlements of 25 mm produced shear cracks and separation over the full height of the wall, with lateral displacements of 25 mm at the top of the wall. Once a critical settlement level was reached, the shear cracks would extend and coalesce and the lateral displacements at the top of the wall would amplify. Subsequently, the writer has observed cases in the field in which settlements of less than 20 mm have caused shear cracks and outward lateral displacements at the top of the wall on the order of 3 to 4 times the settlement magnitude.
Figure 19: Washington Metro, 7th Street building, bearing walls perpendicular to tunnel line: Settlement due to ground loss into tunnel causes outward lateral displacement of 28 mm near connection of façade wall with bearing wall.

Figure 20: Progression of shear cracks, 7th Street building.
Separation of either façade walls or bearing walls can occur due to settlements caused by excavation or tunneling. Lateral displacements can occur even without the propagation of shear fractures as was described in the previous paragraphs. Pre-existing weaknesses on or between the walls or at the connection between the wall and floors can cause separation and lateral displacement at the top of the wall.

When undergoing ground settlement, brick walls may displace outward when subjected to:

- Low floor loads on joists seated on the bearing wall
- Intermediate walls, such as cross walls or non bearing walls that pick up the load of the floor joists as the bearing wall settles causing the joists to become unloaded on the bearing wall.
- Water damage and rotting of joists in their seats.
- Uncontrolled drainage from roof, water damage, deterioration, lack of maintenance or repointing of mortar joints in the side wall.
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 Poor or deteriorated connection of façade wall to bearing wall
 Angular distortion and shear cracks extending between window openings to upper portion of wall.

6 FRAME STRUCTURES

Figure 22 illustrates the results obtained with a 2-D finite element analysis which modeled both the excavation wall, the soil mass and the adjacent building frame using a hypo-plastic constitutive model for sand (Ghahreman, 2004). Side sway of the frame reduced the distortion of the bay nearest the excavation, and resulted in an almost equal and opposite distortion of the third bay. Angular distortions at each floor level above the lower floor, in a given bay, were similar. From the angular distortion, the change in bending moment in the beam due to the distortion can be estimated. The first floor columns were not laterally tied with grade beams and were sensitive to the lateral displacements at the foundation level, a condition similar to that encountered in the Heathrow Car Park (Powderham & Burland (2004) where the first floor columns were damaged. Direct analysis of the bending in the columns can be made by applying the displacement to the base and subtracting the sidesway at the top of the column.

Side sway causes the angular distortion to be spread across more bays and results in lower angular distortions than occur in a brick bearing wall in which the shear cracks and displacements are concentrated in the first section of the wall and the portions of the wall further from the first section may tilt but do not undergo significant distortion. For this case damage often concentrates near the wall, in the first building unit and sidesway is minimal, and angular distortion is approximately four times the deflection ratio. For frames, the sidesway can reduce the angular distortion to twice the deflection ratio.

Wood frame structures with brick veneer have been observed where settlement has caused the frame to side sway and has resulted in small lateral displacement of the wood frame on the far wall as evidenced by a small separation of the window frame from the brick veneer.

Excavation and installation of underpinning beneath a 12-story hotel with brick bearing walls resulted in a small amount of side sway which caused the far right wall in Figure 23 to displace and along with it, the adjacent bearing wall of the historic structure built in the early 1800's. Lateral displacements were concentrated in the stairwell and hall on the left side of the historic structure, which forms a natural structural weakness. The side sway of the left wall of the house resulted in small cracks in the cross walls and crown molding in the hall (Fig. 24), and sticking of a door at a cross-wall into the hall (Fig. 25). The lateral strain was superposed on previous distortions that the building had undergone in which floors had sagged over time toward the center of the building, most likely a result of shrinkage of the central timber bearing wall. The door frame into the hallway had been racked by this previous displacement. Recognition of the pattern of distortion of the structures confirmed that the excavation had caused the side sway, but also showed that the needed repair to the historic structure was primarily patching of the plaster and repair of finishes in the hallway of the historic building. It was important to demonstrate that the damage was not caused by any instability or settlement of the foundation of the structure and there was no need for replacement or repairs to the building’s bearing wall.
Abaqus FEM with Hypo-plastic Soil, Concrete Frame Excavation

Depth: 25 m

Displacement magnification factor: 100

Figure 22: Side sway of elastic frame on a soil mass adjacent to a braced excavation
Figure 23: Excavation to the left of the hotel causes small settlement of hotel foundation, resulting in sidesway of hotel and the left wall of the historic structure.

Figure 24: Cracking of plaster in hallway on left side of historic structure.
7 CONCLUSIONS

In the cases illustrated, the structures themselves served as indicators of the type and causes of distortions and damage that were imposed on them. The ability to observe and read the building response is aided by an understanding of the chain of relationships that extends from the excavation or tunnel and adjacent ground loss or ground movement, to the distribution of ground movement and volume change through the soil mass, to the interaction of the ground with the building, and then to the building distortion and damage. To properly assess the building behavior – both distortion and damage – it is necessary to understand not only the ground movement patterns but also the building’s structural characteristics and finishes: how the building was built, maintained, and repaired. Most of the structures that are affected by excavation and tunneling are on shallow foundations and are older structures, which means that they have a long history of repair and, in some cases, neglect. Some are historic structures or within historic districts. Often the effects of excavation and tunneling are superposed on pre-existing distortions and deterioration. In many cases, pre-existing conditions are separate from and unrelated to the excavation- or tunnel-induced damage. It is most common and natural for residents and owners of buildings to observe and attribute pre-existing conditions in their building to the effects of the adjacent excavation and tunneling. Pre-construction photographs and video are important in resolving such issues. In addition, a logical evaluation and description of the limits and magnitude of ground movements and building distortions due to excavation or
tunneling and early recognition and acceptance of responsibility for excavation or tunneling induced-damage can be most helpful in avoiding disputes.

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Terzaghi, K. (1942a) "Liner plate tunnels on the Chicago (Ill.) Subway, ASCE Proceedings, V. 68, No. 6

ABSTRACT: Estimating potential construction-induced damage to neighbouring buildings is critical for urban underground construction projects. A number of methods have been proposed for this task over the past 50 years. The diversity of urban structure types, their condition, and various modes of displacement that contribute to damage challenge practitioners and researchers alike. Choosing applicable underground construction methods necessitates an examination of risks to surface facilities. Assumptions regarding complex variables, including ground conditions, construction technologies, and construction workmanship can influence decisions of significant technical and economical consequence. This paper illustrates that by combining construction-induced ground deformation patterns, well-known damage category criteria, strain superposition and critical strain concepts, and stochastic modeling techniques, the potential effects of ground deformations on buildings and their root causes can be better understood to allow for more informed decision-making.

1 INTRODUCTION

Settlement and horizontal displacements of structures caused by underground construction can result in costly damage. “Practice shows that clients of large civil engineering contracts are usually exposed to economic risks in excess of what is considered reasonable to third parties who, in many cases, are not the direct beneficiaries of the project” (Eskesen et al. 2004). When things go wrong, it is the “deep pockets” of public agencies, contractor consortia, or their insurers that become targets of litigation. “In 2002 The [British Tunnelling Society] received representations from the insurance industry who...were so concerned that they advised BTS that they were actually considering not offering insurance on tunnel projects unless something could be done to improve the situation.”(BTS 2006). Assessing third party property risks is one focus of the resulting A Code of Practice for Risk Management of Tunnel Works (ITIG 2006).

Usually, the most damage-sensitive buildings are low-rise buildings with masonry load bearing walls or frame structures with masonry in-fill walls. Simplified geometric ground displacement parameters including “angular distortion” and “deflection ratio”, sometimes coupled with horizontal strain, have historically been considered as criteria by which to judge the potential for damage to structures. These criteria have been useful tools but they do not reflect the diversity of building structures, inherent variability in ground conditions, workmanship, and other variables. Such simplifications can over- or under-estimate the real potential for damage. Problems with common assessment methods include: over-
generalization of damage criteria; multiple definitions for similar criteria; building dimension variability is not well accounted for; poor correlation to physically observable damages; and they are not readily adaptable to probabilistic approaches to risk analysis. Therefore, a closed-form analytical approach based on strain superposition principles was developed to evaluate damage that ground displacements may induce in susceptible buildings (Boone 1996, Boone et al. 1999, Boone 2001). Applying these various approaches to understanding, evaluating, mitigating, and managing risks for a single structure can be problematic. Applying these methods for large urban transit projects, that potentially could affect hundreds of structures, becomes even more troublesome. This paper summarizes a number of previous studies related to damage analysis methods, briefly describes several case histories, and outlines a rational approach to evaluating building damage risks.

Figure 1: Relationship between angular distortion, horizontal strain and damage category (after Boscardin & Cording, 1989).

1: Relationship between deflection ratio, horizontal strain and damage category for hogging (after Burland, 1997).

2 BUILDING DAMAGE ANALYSIS METHODS

2.1 Background

Skempton & MacDonald (1956) suggested that “...the settlement characteristic causing cracking is probably the radius of curvature. But a characteristic which is more readily
evaluated, and which is only slightly less logical, is the angular distortion; this conveniently expressed by the ratio of the differential settlement $\delta$ and the distance $L$ between two points."

No consideration was given to the dimensions of the affected parts of the structures. Categorization of damage was limited to the distinction between "cracking" and "structural damage." Burland & Wroth (1975) compared the behaviour of load bearing masonry walls undergoing settlement to the bending of a deep beam subjected to a central point load. They chose the ratio of the central deflection, $\Delta$, and the equivalent beam length, $L$, to characterize deformation (deflection ratio), this being directly related to the curvature of the structure. They combined equations for bending of a deep beam and critical strain values to relate the onset of cracking to deformation and beam dimensions. Two different displacement modes were identified: hogging displacement in which the settlement profile is concave downward, and sagging in which the profile is concave upward. One important aspect of their work included a categorization of damage severity based on observed crack width (Tab. 1). This damage categorization is intuitive, practical, and related to measurable evidence. Yet, deformation and critical strain criteria were not directly linked to damage categories and thus the relative degree of damage beyond the initial cracking of masonry was not readily identifiable. Boscardin & Cording (1989) illustrated the importance of horizontal extension in initiating damage, particularly associated with underground construction. Angular distortion, $\beta$, was defined in this case as the maximum change in slope along the "beam" or wall. Damage categories were related to the angular distortion and horizontal strain criteria (Fig. 1) and Table 1. The resulting damage categorization method was limited to structures with a length to height ratio ($L/H$) of about 1. Later modification of the critical strain approach by Burland (1997) included lateral strain and adapted different values of critical strain to reflect different damage categories, as illustrated in Figure 2. This approach was also generally limited to the case of $L/H = 1$ unless successive graphical constructions are carried out. Boone (2001) and Netzel (2006) completed comparisons of these methods and found that many cases, using angular distortion or deflection ratio can lead to over- or under-estimation of damage.

<table>
<thead>
<tr>
<th>Damage Category</th>
<th>Description of Typical Damage</th>
<th>Crack Width</th>
</tr>
</thead>
<tbody>
<tr>
<td>Negligible (0)</td>
<td>Hairline cracks</td>
<td>&lt; 0.1 mm</td>
</tr>
<tr>
<td>Very Slight (1)</td>
<td>Very slight damage includes fine cracks which can be easily treated during normal decoration, perhaps an isolated slight fracture in building, and cracks in external brickwork visible on close inspection.</td>
<td>1 mm</td>
</tr>
<tr>
<td>Slight (2)</td>
<td>Slight damage includes cracks which can be easily filled and redecoration would probably be required, several slight fractures may appear showing the inside of the building, cracks which are visible externally and some re-pointing may be required, and doors and windows may stick.</td>
<td>&lt; 5 mm</td>
</tr>
<tr>
<td>Damage Level</td>
<td>Description</td>
<td>Thresholds</td>
</tr>
<tr>
<td>--------------</td>
<td>-----------------------------------------------------------------------------</td>
<td>-----------------------------</td>
</tr>
<tr>
<td>Moderate (3)</td>
<td>Moderate damage includes cracks that require some opening up and can be patched by a mason, recurrent cracks that can be masked by suitable linings, re-pointing of external brickwork and possibly a small amount of brickwork replacement may be required, doors and windows stick, service pipes may fracture, and weather-tightness is often impaired.</td>
<td>5 mm to 15 mm or a number of cracks &gt; 3 mm</td>
</tr>
<tr>
<td>Severe (4)</td>
<td>Severe damage includes large cracks requiring extensive repair work involving breaking-out and replacing sections of walls (especially over doors and windows), distorted windows and door frames, noticeably sloping floors, leaning or bulging walls, some loss of bearing in beams, and disrupted service pipes.</td>
<td>15 mm to 25 mm but also depends on the number of cracks</td>
</tr>
<tr>
<td>Very Severe (5)</td>
<td>Very severe damage often requires a major repair job involving partial or complete rebuilding, beams lose bearing, walls lean and require shoring, windows are broken with distortion, and there is danger of structural instability.</td>
<td>&gt; 25 mm</td>
</tr>
</tbody>
</table>

### 2.2 Strain Superposition Method

The strain superposition approach is based on fundamental considerations of ground movement, the deformations that these might induce on an overlying structure, and observed crack formation and enlargement in real building walls (Boone 1996, Boone et al. 1999, Boone 2001). The strain superposition method considers that deformation of bearing walls is analogous to deformation of uniformly loaded deep beams and can be reduced to three basic modes: bending, shear, and direct extension. In this case, the bending and shear components of angular distortion are separated based on an equation for deformation of uniformly-loaded deep beam. Using the well-known equation for beam deformation, assuming a ratio of the elastic and shear modulus of common masonry materials, and assuming a neutral axis position, the proportions of the total maximum central deformation due to bending and shear can be readily defined. The shear strain and bending tensile strain can then be separately determined. By adding the direct lateral extension strain to the bending strain, and subsequently applying plane-strain mechanics, the maximum principal tensile strain can also be determined. When calculating a finite deformation, a length to which the strain is applied must be derived or assumed. For bending and lateral extension, the change in length of the extreme fibre of the idealized beam is used (e.g., the wall top for hogging displacement). For simplicity, a “diagonal” average length is developed using the minimum length of either half the wall length or the wall height and the principal strain angle to approximate an average principal strain trajectory (Boone 1996) and cracking in the major principal strain direction. Though this concept calculates the strains and deformations in one location relative to the actual principal strain trajectories, shear and tensile strains occur throughout the wall and the associated cracking can develop in many areas (particularly near opening corners). Many of the same fundamental principles also apply to cracking damage of frame buildings. In general, shear strains dominate distortions of in-fill walls as a result of differential displacement and the confining effect of columns and beams. Concrete frames produce different in-fill wall deformation patterns than steel frames as concrete frame connections provide nearly “fixed-end” beam conditions, while it is difficult to achieve fixed-ends with welded or bolted connections. Figure 3 illustrates basic steps for evaluating the potential damage category for load-bearing walls and in-fill walls within frames.
Evaluating risks of construction-induced building damage for large underground construction projects

S. Boone

Figure 3: (a) Damage evaluation for lead-bearing walls. (b) Damage evaluation for in-fill walls (left) and beams (right).

The threshold for the on-set of cracking (critical strain), for poor mortar and brick construction can be as little as 0.001%. The degree to which masonry can withstand strain without cracking depends on age, composition, and construction quality. Small cracks in masonry are also common as a result of construction defects, temperature-induced shrinkage and expansion, and other factors. Therefore, low values of tolerable strain are generally assumed.
(between 0.01% and 0.03%) for relatively new or undamaged masonry materials. For very old buildings or those that have suffered self-weight settlement or other damage, it may be more appropriate to use a critical strain equal to or very near 0%. Strain, however, is not necessarily a direct indicator of damage and is difficult to measure. Consider a wall with a length of 10 m that is subjected to a 0.3% strain. If this deformation is manifested in only one crack, then the damage could be considered “severe” (crack width of 30 mm). Alternatively, if 10 cracks of no more than 3 mm each develop, the damage might be categorized as “moderate.” The strain superposition method further considers that beyond the critical strain, cracks will widen in general preference to new cracks forming. Based on case history data, for buildings with wall heights ranging between one and four stories and wall lengths of 3 to 20 m, the maximum crack width is about two thirds of the cumulative crack width (Fig. 4).

Figure 4: Probability crack widths for brick and block walls, \( C_i = \text{individual crack width} \).

Figure 5: Comparison of case histories and strain superposition method of damage estimation. Categories of damage Negligible, Very Slight and Very Severe not shown.
Having defined the deformation geometry, separated the deformation modes, applied plane-strain mechanics principles, and applied these to reasonable estimations of building dimensions, finite cumulative crack widths for total and principal tensile strains can be calculated. These cumulative crack widths then can be compared to the relative distribution of crack widths and Table 1 to determine damage category (Figs. 4 and 5). While neither principal nor total tensile crack widths may exceed a particular crack width threshold individually, their combined effects may produce sufficient cracking to exceed the threshold damage category as shown in Fig. 5.

### 2.3 Numerical and Physical Modelling

Potts & Addenbrooke (1997) completed a parametric set of numerical analyses examining the influence of structures on the ground displacement patterns over machine-bored tunnels. The geometry of their analyses is illustrated in Figure 6 in which: the excavated tunnel diameter was fixed, two tunnel depths of 20 m and 34 m were considered, and the overlying building was represented by an idealized beam of width, B, with its centre at an offset distance, e, from the tunnel centreline. The principal parameters studied were the axial and bending stiffness of the structure relative to the stiffness of the underlying ground. Two parameters were used to describe the relative bending stiffness, $\rho^*$, and relative axial stiffness, $\alpha^*$, of the structure:

$$\rho^* = \frac{EI}{E_s H^4}; \quad \alpha^* = \frac{EA}{E_s H}$$

where $E$ is the Young's modulus, $A$ the cross-sectional area, and $I$ the internal moment of inertia, $H$ the half width (i.e., $B/2$) of the beam and $E_s$ is the secant modulus of the soil taken at 0.01% strain. The results (Fig. 6) indicated that buildings of high relative bending stiffness will attenuate the curvature of settlement profiles and horizontal strains will be reduced for those with high axial stiffness. Taylor & Grant (1998) conducted a series of centrifugal model tests and the results were generally consistent with the trends of Figure 6.

![Figure 6](image)

Figure 6: Design curves for modification factor for deflection ratio (left) and horizontal strain (right) (after Potts & Addenbrooke, 1987; data points from Simic, 2006).

Burd et al. (1998) and Houlsby et al. (1999) completed three-dimensional finite element modelling of a combined tunnelling and building damage problem in which a tunnel passed beneath a building at a skewed angle. The model included a ground mass of undrained clay, progressive tunnel construction, and a masonry building represented by interconnected 2-dimensional plane stress element façades. A non-linear soil constitutive model was used and
masonry was modelled as an elastic material unable to resist tensile strains. This work also indicated that the presence of a building can have an effect on the displacement patterns. They further concluded that façades subjected to sagging are more resistant to crack damage than those undergoing hogging displacements. Simic (2006) completed back analyses of 26 buildings affected by the construction of the Madrid metro and compared “green field” displacement patterns with those of adjacent buildings. This data plotted in Figure 6 illustrates wide scatter between the measured points and the design curves derived from numerical modelling. Simic (2006) concluded that buildings in the sagging mode tend to behave less stiff and thus are more subject to damage and influence the displacement pattern less than parametric numerical models suggest, but that buildings exhibiting hogging displacements more closely followed the parametric numerical model trends. Son & Cording (2005) conducted a series of scale physical model tests to evaluate displacement and damage patterns of 2 to 4 storey brick walls undergoing hogging displacements adjacent to a supported excavation. The geometry and material properties were scaled to 1/4 to 1/10 of field dimensions. They also completed a series of numerical model tests that were first calibrated to the physical model tests and then used to model other situations. This work was completed using the UDEC distinct element code (Itasca 2000). They concluded that initial ground displacements near the building front primarily induce shear distortion and lateral strain at the building base. The load of the settling façade increased distortion near the front end of the bearing wall and caused cracks to concentrate near the façade wall or the first column of windows. As the excavation deepened and displacements influenced a larger proportion of the structure, the model buildings experienced bending deformation and crack development at the wall top. Cracks initially formed around openings and their formation decreased building stiffness such that the model structures conformed more closely with the “green field” displacement patterns. They further concluded that “cracking strongly controls building response to ground movement, and elastic analysis alone can produce misleading results about building response.” This cracking-induced transition in behaviour is evident through the ratio of $\beta$ and $\Delta/L$. They found that the ratios ranged between about 3, for elastic and minor cracking conditions, and 4, for moderate to severe cracking conditions. A ratio of 3 indicated that the building distortion was less than the change in ground slope in “green field” conditions, whereas a ratio of 4 was consistent with the “green field” conditions. As part of this work, the damage criteria developed by Boscardin & Cording (1989) was updated, with little change, in terms of a general state of strain at a point and, thus was considered to be independent of E/G, L/H or the neutral axis position. Reasonably good agreement was shown between the numerical and physical models and the state of strain concept and damage criteria (Fig. 1). Son & Cording (2007) completed another companion set of numerical modelling exercises to better understand the effect of building openings on the ratio of bending to shear stiffness for brick load bearing walls without the influence of floor or roof structures. The effect of openings on such walls significantly reduced the shear stiffness and had a limited effect on the bending stiffness. For most cases, the shear stiffness decreased by about 45% to 60% when the opening area increased from 0 to 30% while the bending stiffness decreased by 20 to 26%. The shear stiffness of joint mortar was also shown to have an effect on the overall shear stiffness of the wall. Given that the shear stiffness was more significantly affected by these variables, the ratio between bending and shear stiffness was shown to be relatively high, between 12 and 23. 5, prior to the onset of cracking. Thus, for low L/H ratios as well as for undamaged walls, shear deformation controlled the threshold of displacements at which cracking occurred.
The numerical analyses described above relate to parametric studies with the intent of evaluating simplified approaches to the soil-structure interaction problems of building responses to underground construction. Bryson (2002), however, completed detailed structural finite element analyses of a concrete frame structure with masonry infill walls adjacent to a deep excavation in Chicago (see example 3.2 below). The software utilized for this purpose was ETABS and SAP2000-7.4 (CSI 2000) for general three dimensional structural analyses and for detailed analyses of selected structural components, respectively. Both self-weight settlement and the displacements induced by the excavation were considered within the modelling steps. The superposed strains from both of these stages were then used to compare to limiting strain criteria for the concrete and masonry materials. This work indicated that the building sections deformed primarily in shear and a comparison of the induced strains and observed cracking indicated a critical strains for pure shear of about 0.0035%, consistent with the information summarized by Burland & Wroth (1974) and Boone (1996). Bryson (2002) demonstrated that a highly detailed structural finite element analysis of a structure could reasonably indicate the building’s cracking response to induced displacements.

3 DAMAGED BUILDING EXAMPLES

3.1 Toronto

During a subway expansion project in the mid-1990s, a deep cut and cover excavation was made beneath an urban street. The buildings lining the street were mostly 50 to 100 year old low-rise residential and commercial structures built of masonry load-bearing walls (brick or block). Four of these structures were measured in detail and described by Boone et al. (1999). Prior to construction, condition surveys were carried out in general accordance with the guidelines prepared by the Building Research Establishment (BRE 1989a, 1989b). Sketches and photographs of visible cracks on interior and exterior walls were compiled along with summaries of the building construction and condition. Crack widths were generally noted as “hairline” (being visible on close inspection but a fraction of 1 mm wide), or estimated to 1 mm to 2 mm intervals of width. Following construction, each building was visited once again, and changes to conditions or new damage were noted. Where damage was observed to be Slight or greater in the post-construction survey reports, detailed width measurements were made for each observable crack. One building that exhibited Moderate damage at the conclusion of construction is illustrated in Figure 7. This masonry-block bearing-wall structure was situated 2 m from the edge of an 18.5 m deep cut through glacial till and dense sand deposits that was supported by soldier-piles and lagging and preloaded cross-braces (Boone & Westland, 2004). Ground and structure monitoring data suggested that this building had little influence on the final displacement profile; i.e., the building deformed to match the “green-field” displacement profile of the cross-street immediately adjacent to the building.
Figure 7: Damaged building in Toronto. Inclinometer positioned immediately adjacent to front face of structure.

Based on the criteria of Burland (1997), the building would be expected to suffer damage near the threshold of Very Slight to Slight damage, given the final ground and building
displacement patterns. Using the approach of Boscardin & Cording (1989), it would again be expected that the damage would fall near the threshold between Very Slight to Slight damage. Using the strain superposition approach, however, the building would be anticipated to suffer damage near the threshold of the Slight to Moderate categories. The most damaging mode of displacement, in this particular case, is the horizontal extension strain. Clearly, the pre-existing damage was compounded by the excavation work and the overall end-of-construction condition of Moderate damage was the accumulated total of these conditions and changes.

Figure 8: Damaged building in Chicago, north-south wall section. Inclinometer located about 1 m north and 5 m east of west end of wall (after Bryson, 2002; and Finno et al., 2005).

3.2 Chicago

Bryson (2002) and Finno et al. (2005) presented a detailed case history of a building damaged by an adjacent cut and cover subway station excavation that wrapped around and abutted the north and west walls of the structure. Two sections of the building are illustrated in Figures 8 and 9. The 12.2 m deep excavation through soft to medium clay was supported by a secant pile wall, one level of cross-lot struts, and two levels of soil anchors.
Inclinometers and survey points were used to monitor ground and structure displacements. The exterior wall illustrated in Figure 8 was subjected to displacements arising parallel to the main excavation as well as from the stair and escalator area shaft along the north wall. This wall, situated approximately 2 m from and parallel to the main excavation face, is constructed of limestone-clad masonry walls for the north and south sections and the center section, set back from the plane of the north and south sections by about 2.5 m, contains many windows and the main entranceway.

Figure 9: Damaged building in Chicago, east-west frame section. Inclinometer located about 2 m west of west end of wall (after Bryson, 2002 and Finno et al., 2005).
The interior frame section shown in Figure 9 was oriented perpendicular to the main excavation where vertical displacements were at a maximum. Displacements related to two specific construction and damage stages are illustrated in Figures 8 and 9, being when cracking was first observed and the final measurements after construction. Maximum measured lateral displacements of the soil mass at foundation level were about 5 mm. However, survey measurements indicated that lateral strain transfer from the underlying soft clay to the structure was negligible.

Treating the entire north-south exterior wall as a single wall in a sagging mode of displacement, $\Delta L$ values of 0.012% and 0.027%, for both damage stages respectively, suggest that damage would be Negligible using the criteria of Burland (1997). Using the criteria of Boscardin & Cording (1989), it could be expected that this structure would suffer Negligible damage when cracking first appeared and might experience damage near the threshold between Negligible and Very Slight for this particular wall at the final stage. Applying strain superposition principles to this same idealised wall, damage for the day cracking was first observed would be near the lower threshold but within the Slight damage category. Moderate damage would be anticipated for the end-of-construction condition. However, the wall in this case may not behave as a single entity on account of the set-back centre section with many openings. If the north and south sections are evaluated independently but still considered within the overall sagging settlement profile, whereby because of their small L/H ratio they are primarily subjected to shear displacement (analogous to a single panel in a frame building), the strain superposition method indicates that these sections would experience Slight damage at the onset of cracking and post-construction stages, respectively, though with differing magnitudes of cumulative diagonal crack width.

For the east-west frame and infill wall section within a hogging-shape displacement profile, $\Delta L$ values of 0.013% and 0.048% suggest a Negligible damage category for the both the day cracking was first observed and the end of construction condition using the Burland (1997) approach. The method suggested by Boscardin & Cording (1989) would indicate damage categories of Negligible and near the threshold between Negligible and Very Slight categories for these two respective damage stages. Use of the strain superposition method to evaluate the infill sections of this wall, for the first floor section between Points A and B on Figure 9, indicates that the differential settlement would cause Very Slight and Slight damage at the onset of cracking and end of construction, respectively, more closely resembling the actual performance. Similar damage would be expected to occur in each of the building levels in which there existed a masonry infill wall within the concrete frame.

Finno et al. (2005) recognised the difficulty in using the idealized deep beam analogy common to the methods of Boscardin & Cording (1989), Burland (1997) and the load-bearing wall aspect of the strain superposition method (Boone 1996). The deep beam model was applied to this case using a variety of E/G values and neutral axis positions using an extension of Burland & Wroth’s (1975) method developed by Voss (2003). An E/G value of 0.5 was used for the north and south wall segments of the building’s west wall that were assumed to be very stiff because of the lack of openings. The more flexible centre section of this same wall was assigned an E/G value of 2.6, and the frame sections were considered the most flexible and assigned an E/G value of 12.5. Thus, the E/G ratio varied by a factor of 25. This approach did not indicate that cracking would occur. It is clear in this case, that the north-south wall does not readily lend itself to the idealized deep beam analogy, with its very open and set back centre section. The deep beam analogy is also clearly not appropriate for the east-west frame with infill walls. Furthermore, this building experienced displacements in both hogging and sagging modes, and the methods of Burland (1997) and Boscardin &
Cording (1989) are not well adapted to directly evaluating the sagging displacement mode. Given that the intense structural finite element analysis completed for this building by Bryson (2002) would not necessarily be practical for a preliminary-stage damage potential assessment of similar frame structures, Finno et al. (2005) developed a simplified approach to evaluating the strains in this structure by considering the building analogous to a laminate beam in which the concrete floor diaphragms provided significant tensile restraint. This laminate beam model was shown to reasonably indicate when cracking would first occur in different sections of the structure. Back-calculation of the ratio between the overall bending stiffness and shear stiffness of these same sections, being the ratio of $EI/GA_v$, where $A_v$ is the cross-sectional area in shear, suggested that when considering the overall wall section, the variability of this ratio was far less than used for the deep beam analogy and was closer to about 4.5, highlighting the difficulty in choosing appropriate $E/G$ values. Further, the laminate beam model indicated that the position of the neutral axis of these various wall and frame sections ranged between about 0.35 to 0.49 times the wall height with the smallest value for the sagging wall section. Although the simplified laminate beam approach reasonably identified the onset of cracking, it did not lend further insight into the degree of damage that might be anticipated for greater displacement magnitudes after the onset of cracking.

Figure 10: Damaged Foochow Methodist Church in Singapore (after Shirlaw et al., 2006).

3.3 Singapore

Structures on mixed foundation types can be a particularly difficult problem for urban underground construction projects (Shirlaw et al. 2006). During construction for the North East Line subway, the Foochow Methodist Church was damaged. Initial observations of the facility early in subway planning suggested that it was all one building of similar construction. During construction for a neighbouring development, shortly before subway construction started, the Sunday school settled about 15 mm along the north wall, rotated away from the remainder of the building, and a crack opened at the junction of the two building sections, opening gradually from ground level to a maximum of about 12 mm at the top. This structure behaviour indicated that the building was on mixed foundations and a detailed structural assessment was carried out in which the foundation design and structures' history were better defined. The church consisted of a number of sections including the original three-storey sanctuary supported on shallow spread foundations and an addition to the sanctuary, surrounding the original, founded on steel H-piles driven through the underlying marine clay.
The nearby Sunday school was built on wood piles driven into the marine clay, and the relatively new kindergarten addition between these was supported on steel H-piles driven fully through the clay (Fig. 10). Because of the sanctuary’s sensitivity to damage, this was underpinned prior to subway construction so that all sections would be supported on piles. Later construction of the nearby subway tunnel produced consolidation settlement of the marine clay beneath the church. The Sunday school settled 14 mm at the junction of the structures, 100 mm on the north side, and tilted (1:130) relative to the remaining building sections. Cracks up to 50 mm wide opened at the junction of the structures. Other cracks also appeared in the Sunday school, kindergarten and sanctuary areas, but the Sunday school suffered the worst damage. In this particular case the differing foundation systems led to different structure responses to the soft clay consolidation resulting in Severe damage.

Figure 11: Damaged 148 Race Course Road in Singapore (after Shirlaw et al., 2006).

The case of 146 and 148 Race Course Road illustrates two adjacent buildings supported by different foundation types (Fig. 11). The two-storey 148 Race Course Road building was constructed of masonry block in the 1950s and supported on wood piles driven into the underlying marine clay. Next door, a similar structure was demolished and a four-storey concrete building was constructed in the 1990s and supported on steel pile foundations taken fully through the marine clay to a more competent bearing stratum. The exterior walls of the two buildings abutted one another. During construction of the 1990s structure, half of the former masonry party wall was removed and a thin membrane was placed between the new and old walls. However, because of the roughness of the remaining party-wall there was significant vertical interlocking of the two structures. During construction of the nearby subway tunnel, consolidation settlement of the underlying soft marine clay occurred and one side of the building settled by 107 mm. This racking distortion caused Very Severe damage, but the building was repaired and returned to service.

Clearly, in both these cases, the modes of displacement and damage severity are not readily suited to analysis using any of the aforementioned methods. None of the buildings can be treated as an idealized beam, nor are the damages related to a clear case of differential settlement distortion of a frame structure. If the façade of 148 Race Course Road were considered analogous to a single panel and the strain superposition method equations for an infill wall in a steel frame were utilized to assess the racking damage, this would indicate a Very Severe damage category, consistent with the final outcome. However, if this case were to be evaluated prior to construction, the results would be predicated on developing a clear
estimate of the ground displacement and a judgement that settlement would be partially attenuated on one side by the abutting structure.

4 APPLICATION OF DAMAGE ANALYSES METHODS

For underground urban transit projects it has become relatively common to follow a step-wise process for evaluating the effect that construction may have on nearby buildings. First, a number of discrete settlement analyses are completed for several general cases of construction methods and ground conditions along the transit tunnel alignment. Typically, upper, average, and lower bound displacement expectations are developed for representative ground conditions. Where estimated settlements are less than some small value, perhaps 10 mm (Rankin, 1988), buildings outside this boundary are eliminated from further evaluation. A second step is undertaken for all buildings along the route within the settlement threshold boundary. These are subject to a preliminary assessment in which relatively conservative and simple methods are applied to evaluate the potential degree of damage that the structures might suffer during construction from the anticipated ground displacement cases (considering both settlement and horizontal strain). Third, those buildings that the second step suggests might suffer damage beyond some acceptability threshold are evaluated individually in detail, iteratively using sophisticated methods depending on the nature and sensitivity of the building. The general multiple stage assessment approach has been described by a number of authors and will not be discussed in greater detail herein (e.g., Rankin 1988, Boone et al. 1998, Burland 2001, Harris & Franzius 2006). This paper focuses on the second step of this process so that it can be robust, rational, and provide insight for developing suitable risk management strategies.

Table 2: Example variables related to tunnelling-induced building damage.

<table>
<thead>
<tr>
<th>GROUND RESPONSES</th>
<th>BUILDING RESPONSES TO GROUND DISPLACEMENT</th>
</tr>
</thead>
<tbody>
<tr>
<td>Face pressure</td>
<td>Age and condition of structure (critical strain)</td>
</tr>
<tr>
<td>Annular gap grouting</td>
<td>Proximity to tunnel (displacement mode, magnitude)</td>
</tr>
<tr>
<td>TBM attitude control</td>
<td>Height and length of building relative to displacement patterns</td>
</tr>
<tr>
<td>Soil properties</td>
<td>Superstructure (bearing wall, frame, floors, connections, etc.)</td>
</tr>
<tr>
<td>Groundwater conditions</td>
<td>Openings in building (bending stiffness, shear stiffness)</td>
</tr>
<tr>
<td>Consolidation responses</td>
<td>Foundation type(s)</td>
</tr>
<tr>
<td>Depth and diameter of tunnel</td>
<td>Material types &amp; finishes (brick, block, plaster, stucco, etc.)</td>
</tr>
</tbody>
</table>

The process described above, while useful in assessing ground displacements and the potential building damage for individual structures, is largely based on discrete assumptions and does not quantitatively evaluate risk, being the probability that a damaging event will occur and the magnitude of the event consequences. For example, consider that for a machine-bored tunnel built through soft ground through a dense urban area, many factors will influence the final responses of buildings to the construction; some of these are identified in Table 2. It would be a rare case in which all of these factors are precisely known before construction. In addition, all methods used in evaluating both ground responses to underground construction and building damage assessment are abstract mathematical...
representations of a real world problem and, therefore, “As far as the laws of mathematics refer to reality, they are not certain; and as far as they are certain, they do not refer to reality” (Einstein, 1921). Each source of uncertainty is a source of risk. Some account must be taken of the uncertainties in each one of these variables.

The keys to managing risks are identifying the uncertainties and developing sound approaches to controlling these uncertainties at their source (e.g. Shirlaw et al. 2006). With problems such as assessing building damage from tunnel construction with multiple variables, stochastic modeling likely represents the most technically sound approach to project decision-making.

4.1 Example Project

During design for a new subway project, a review board suggested that potential cost savings could be achieved by raising the vertical alignment of twin bored tunnels to between 10 and 18 m below the ground surface to limit the depth of station excavations and structures. Because of limited horizontal space between the piled foundations of buildings on either side of the alignment, the tunnels would be built at different elevations (upper and lower) with the lower tunnel driven first (Fig. 12). A target maximum surface settlement trough unit volume, $V_s$, equal to 0.5% was selected by the project designer to limit the potential for damaging nearly 2,000 historical buildings along the subway route. The 17th century buildings typically measured 18 m perpendicular to the street (with little variation), 10 m to 16 m high, and 7 m wide parallel to the tunnel with their brick masonry walls supported on driven wood piles (Netzel & Kaalberg, 2001). The designer concluded that the tunnels must be driven at depths of between about 21 and 35 m in the soft to firm marine deposits to limit building damage to acceptable levels. Critical to the assessment of feasibility of both designs (shallow vs. deep) were assumptions regarding construction performance. An independent performance and building damage assessment was therefore completed to assist with decision making.

![Figure 12: Example tunnel arrangement, shallow tunnel option, 15 m wide corridor, lower tunnel position varied in elevation and distance from piles and upper tunnel.](image-url)
For machine-bored tunnels in soft ground, surface displacements arise from factors such as excess excavation of soil at the tunnel face, excess face pressure (inducing heave), poor grouting of the annular space between the cut and lining outside diameters, long-term consolidation of soft soils induced by changes in pore-water pressures (e.g., excessive face pressures, grouting pressures, seepage), machine attitude control, and uncontrolled ingress of groundwater and soil particularly where flowing soils overlie more competent soils. Each of these factors represents elements that induce risk to adjacent structures.

Estimating ground displacements caused by tunnelling is often completed assuming values for “volume loss”, $V_s$, defined as the unit surface volume of the settlement trough as a percent of the unit tunnel face volume (e.g., Peck, 1969, New & O’Reilly, 1991). Volume loss assumptions are typically based on case histories where “envelopes” of displacement are drawn around field data to capture the maximum ground displacements experienced above the tunnels. The most common surface settlement trough shape is that of an inverted normal probability distribution curve. Ground displacements can be calculated using the mathematical properties of this curve, the area under which is assumed equivalent to the unit volume loss, and an assessment of the trough width. However, volume loss estimates lump all of the identifiable root causes together, obscuring reasons for differences in performance. Further, envelopes of settlement, while capturing the most severe conditions of maximum displacement, do not represent a realistic picture of overall performance (Fig. 13).

![Figure 13: Probability distribution of surface settlement through unit volume from machine tunnelling.](image_url)

Figure 13 illustrates values of $V_s$ associated with tunnels driven through a variety of soil conditions in Singapore (Shirlaw et al. 2003) and through dense granular and stiff to hard cohesive glacial soils in Toronto. These projects illustrate that measured ground displacements were less than the target maximum ($V_s = 2\%$ in both cases) close to 90% of the time, and less than half the target maximum more than 60% of the time. The probability of excessive settlements in both these cases ($V_s>3\%$) ranged between 5% and 10%. Figure 14 illustrates several examples of EPB pressure control performance in several soil types (average EPB pressure per liner ring). Bakker et al. (2000) illustrated a direct relationship between injected tail void grout volumes and settlement for the first of two parallel tunnels in the Netherlands. The probability of achieving target grout volumes in that particular case was about 40%. Figure 15 illustrates the variability of grouting performance for a number of published cases.
Ground displacements for this independent project evaluation were estimated using, in parallel, numerical modeling and a combination of the methods proposed by Lee et al. (1992), Loganathan & Poulos (1998), modified for considerations of depth and stability identified by Atkinson & Potts (1977) and Mair (1992). Additional detail on the application of the combined displacement analysis methods is provided in Boone & Carvalho (2002). The combined methods allowed characterization of the direct effects of face pressure control, tail void grouting, and TBM alignment control and their influence on displacements since the surface expression of “volume loss” was a result of these calculations rather than an initial assumption. It was assumed that face pressures could typically be maintained at about 80% of the target value, ranging between a low of about 50% and a high of 120%. It was further assumed that the tail void could typically be grouted to approximately 80% of the theoretical volume in peat, very soft clay, and loose sand below groundwater, with a range of between 50% and 100%. These analyses were carried out to examine the ground displacements both perpendicular and parallel to the tunnel to understand the effects that these may have on both the facades and bearing walls of the nearby structures.

Using deterministic values of tail void grouting success and face pressure control and average soil properties, the displacements along the building foundations were calculated. The strain superposition method of building damage potential estimation was then used to characterize the response of the buildings to the induced displacement, producing the results in Table 3. This analysis assumed that control of tunnelling variables (e.g. face pressures and grouting success) remained constant and identical for both upper and lower tunnels, an unlikely field result based on experience. The results suggested that for the shallow tunnels option to maintain structure damage to less than the “slight” category, control of tail grouting and face pressure would both have to be within 5% of the theoretical values at all times to result in maximum $V_s$ value of less than 0.5%. Assuming typical performance values for
ratios of grout injection and face pressure as noted above, the deterministic analysis suggested that all buildings near the tunnels would suffer moderate to severe damage. In this case, if the same performance indicators were to be applied to the deeper original design, building damage would be expected to be “slight” for all buildings. This also assumed, however, that the ground building damage prediction methods were accurate and appropriate. As illustrated by Boone (2001) in a comparison of damage prediction methods for over 100 case histories, all methods can either over- or under-predict the damage category by differing degrees.

Figure 15: Examples of tail void grouting performance where \( V_i \) = volume injected, \( V_g \) = theoretical gap volume to be filled, and \( V_{\text{average}} \) = average volume of ground injected on a per metre basis.

Whether by using “volume loss” or workmanship assumptions, a deterministic assessment of ground displacements and consequent building damage prompts a number of important questions including:

- What soil properties are appropriate to evaluate the potential ground displacements: worst case, average, or best case and, what are the chances that each or any of the selected cases will prevail/occur?
- What are the chances that the volume loss or workmanship assumptions will not be correct or conservative? and
- Clearly, planning for the worst case everywhere may be prohibitively expensive and unrealistic, but what funds may be necessary to set aside to address damage that does occur?

A comparison of the tunnel options was completed to assist in answering these questions, applying the same analytical methods as described above, through a “Monte Carlo” (stochastic) simulation of the tunnels passing by 100 buildings, to better understand the risk implications of the potential tunnel options. For this simulation, ground displacement and
building damage calculations were performed by randomly selecting values for each variable based on its statistical distribution. This process was repeated several thousand times to develop probability density and cumulative probability distributions for the results. Table 3 summarizes analysis input data where it was considered that a normal probability distribution represented variation of soil properties based on statistical analyses of field and laboratory testing results (Noord/Zuidlijn 2000). Face pressure and grout injection control were assumed to be consistent with other tunnelling projects in soft marine clay soils, typified by Figures 13 and 15. Although the modeling process for the combined uniform and normal random variables is relatively simple, the output illustrates several significant issues. The analyses indicated that mean volume losses at the ground surface are about 28% better for the deep tunnel option (Fig. 16), and, as shown in Figure 17, the probability for limiting damage improves by about 45% such that the probability of buildings sustaining “slight” or less damage is about 85%. Notably, although the deterministic approach indicates that all buildings would suffer only slight damage, explicitly considering the variability in soil conditions and tunnelling workmanship (root causes), and uncertainty in prediction accuracy suggests that there remains a 15% probability that buildings could experience moderate to severe damage. Based on similar analyses for each potential tunnel arrangement option it was concluded that the shallow tunnel options left little room for error and unforeseen conditions, and demanded performance control that was unlikely to be achieved in practice.

Table 3: Results of deterministic performance predictions.

<table>
<thead>
<tr>
<th>$V_s (%)$</th>
<th>Grout injected/Target volume</th>
<th>Face pressure/Target pressure</th>
<th>Potential damage category</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shallow tunnel option (upper 11 m, Lower 18 m, to tunnel axis)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.4U</td>
<td>0.95</td>
<td>0.95</td>
<td>Slight</td>
</tr>
<tr>
<td>0.4L</td>
<td>0.95</td>
<td>0.95</td>
<td>Slight</td>
</tr>
<tr>
<td>0.5U</td>
<td>0.95</td>
<td>0.85</td>
<td>Moderate</td>
</tr>
<tr>
<td>0.4L</td>
<td>0.95</td>
<td>0.85</td>
<td>Slight</td>
</tr>
<tr>
<td>1.0U</td>
<td>0.90</td>
<td>0.75</td>
<td>Moderate</td>
</tr>
<tr>
<td>0.9L</td>
<td>0.90</td>
<td>0.75</td>
<td>Slight</td>
</tr>
<tr>
<td>1.7U</td>
<td>0.80</td>
<td>0.80</td>
<td>Severe</td>
</tr>
<tr>
<td>1.4L</td>
<td>0.80</td>
<td>0.80</td>
<td>Moderate</td>
</tr>
<tr>
<td>2.0U</td>
<td>0.78</td>
<td>0.70</td>
<td>Severe</td>
</tr>
<tr>
<td>1.7L</td>
<td>0.78</td>
<td>0.70</td>
<td>Moderate</td>
</tr>
<tr>
<td>Deep design tunnel option (upper 21 m, Lower 28 m to tunnel axis)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.3U</td>
<td>0.80</td>
<td>0.80</td>
<td>Slight</td>
</tr>
<tr>
<td>1.1L</td>
<td>0.80</td>
<td>0.80</td>
<td>Slight</td>
</tr>
</tbody>
</table>

Notes: 1 Volume loss for tunnels separate – cumulative volume loss not shown. 2 Damage category shown for cumulative effect of both tunnels.
In this case, stochastic analyses allowed use of realistic tunnelling performance estimates, based on measured data, to determine the probability of structures suffering different categories of damage. This approach also avoided either over-emphasizing potential damage if lower-bound (or conservative) soil parameters were used or the possibility that, if average or upper-bound soil parameters were used, significant risks of damage might be overlooked. Although the example stochastic analyses considered the major influences of ground conditions, tunnel depth, face pressure control, grouting success, building dimensions, relative tunnel and building positions, and accuracy of the damage assessment method, other influences could readily be considered for future projects if they could be adequately statistically characterized.

Table 4: Input to stochastic analyses.

<table>
<thead>
<tr>
<th>Building height</th>
<th>Uniform</th>
<th>Min. 10 m; Max. 16 m</th>
</tr>
</thead>
<tbody>
<tr>
<td>Building length</td>
<td>Constant</td>
<td>18 m</td>
</tr>
<tr>
<td>Face pressure/Target pressure</td>
<td>Normal</td>
<td>Mean 0.8; Std. Dev. 0.2</td>
</tr>
<tr>
<td>Grout injected/Target volume</td>
<td>Normal</td>
<td>Mean 0.9; Std. Dev. 0.08</td>
</tr>
<tr>
<td>Predicted category-Actual damage category</td>
<td>Normal</td>
<td>Mean 0.45; Std. Dev. 0.8</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Layer</th>
<th>Soil properties – Mean (Std. Dev.)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$\gamma_{\text{nat}}$ (kN/m$^3$)</td>
</tr>
<tr>
<td>Fill</td>
<td>15.0 (3.1)</td>
</tr>
<tr>
<td>Peat</td>
<td>10.5 (1.1)</td>
</tr>
<tr>
<td>Marine clay</td>
<td>16.5 (1.7)</td>
</tr>
<tr>
<td>Marine clayey sand</td>
<td>17.9 (1.0)</td>
</tr>
<tr>
<td>Marine clay</td>
<td>15.2 (1.5)</td>
</tr>
<tr>
<td>Peat</td>
<td>11.7 (1.2)</td>
</tr>
<tr>
<td>Sand</td>
<td>19.8 (0.9)</td>
</tr>
<tr>
<td>Silt &amp; sand</td>
<td>18.5 (1.5)</td>
</tr>
<tr>
<td>Sand</td>
<td>19.0 (0.9)</td>
</tr>
<tr>
<td>Marine Clay</td>
<td>18.0 (0.8)</td>
</tr>
<tr>
<td>Marine Clay</td>
<td>17.9 (0.4)</td>
</tr>
</tbody>
</table>

$\gamma_{\text{nat}}$ = natural unit weight, $c'$ = effective cohesion intercept, $\phi'$ = effective friction angle, $S_u$ = undrained shear strength, $E'_{50}$ = secant elastic modulus at 50% failure stress, * = assumed value for stochastic analysis, all other statistical data based on Noord/Zuidlijn (2000).

Having rationally identified the probability of differing degrees of building damage, the financial consequences can then also be rationally addressed through use of probability-based cost analyses methods (McGrath et al. 2006, Roberds & McGrath 2006). Cumulative
probability distributions of financial cost can then be used by decision-makers to make better strategic financial decisions based upon the owner’s tolerance of risk and the cost-benefit evaluation of risk management or reduction options.

Figure 16: Estimation of surface settlement through volume using stochastic analyses for shallow and deep tunnel options.

Figure 17: Probability distributions for building damage category using stochastic analyses for shallow and deep tunnel options.

5 DISCUSSION

Over the past 50 years, a number of simplified methods have been proposed for evaluating the potential for settlement and horizontal ground strain to cause damage to buildings. A number of these approaches use deep beam analogies to permit use of structural and material engineering mechanics to assess the effect of changes in building geometry that may be forced by ground displacement patterns. Many of the methods focus on the important criteria of when cracking first occurs in a building since this represents the first physical
changes observed by concerned owners and occupants. All of the methods, however, suffer from their inherent simplification of a complex problem.

A comparison of numerical modelling work, physical test data (see Boone 1996 for summary), and case histories suggests that while using discrete critical strain values as a basis for determining the onset of cracking in relatively intact or new masonry materials, values of critical strains for entire walls of older masonry structures may be very nearly zero. Opening of pre-existing cracks, initially formed by self-weight settlement or temperature fluctuation and subsequently masked by finishes, may be a more common form of damage initiation and propagation than onset of cracking in relatively intact materials. Caution must be exercised in choosing any values of critical strain for use in building damage analyses.

Comparing the work of Son & Cording (2005) and Boone (1996, 2001) indicates that the strain superposition approach overestimates the maximum crack width by about 30% for the scaled physical models and numerical models. Yet, case histories suggest that the combined angular distortion and horizontal strain criteria first proposed by Boscardin & Cording (1989) may underestimate potential damage. The relationship Son & Cording (2005) identified between the slope of the deflection curve and deflection ratio for damaged structures that nearly conform to the green field settlement profile is identical to that indicated by Boone (1996), where $\frac{\beta}{(\Delta L)} = 4 \left(\frac{\nu_{\text{max}}}{L^2}\right)$, Fig. 3a. For undamaged structures, however, this ratio is closer to 2. Simic (2006) provides examples where sagging building cases are not necessarily less susceptible to damage than hogging building profiles and the field cases depart from trends indicated by numerical modelling. The work of Finno et al. (2005) suggests that the neutral axis of walls (for use in the deep beam analogy) may not be at the base for hogging displacement modes and that floor and roof structures may provide significant tensile restraint depending on the building’s design and construction, particularly if these methods are applied to combined bearing wall and frame structures. All these investigators illustrate the difficulty related to choosing appropriate values of bending and shear stiffness for use in simplified approaches to building damage assessment. All of the research and practice related to this field of study clearly indicates that their remains uncertainty with all simplified methods of building damage analysis and that there is no single “correct” method for analysis.

Bryson (2002) and Son & Cording (2005, 2007) demonstrate successful application of structural numerical modelling techniques to specific cases and building details. However, the effort required for individual structures can be intensive. The intensive effort, while necessary for some structures, may not be appropriate or practical for evaluating many individual buildings along an urban underground transit project. It is clear in all of the research and published case histories reviewed for this and precursor papers, that the peculiarities of local construction materials and practices may have a significant influence on the response of buildings and whether one or the other of the simplified methods represents a better choice for a local area. What, then, is the solution?

The most practical and reliable course of action for evaluating the potential for and consequences of damaging nearby buildings, as a result of underground construction for large urban projects, is summarized below.

- Complete detailed parametric numerical analysis of a few selected building types, much in the form completed by Bryson (2002) and Son & Cording (2005, 2007) to understand better how local materials and building details influence:
  - the neutral axis of walls that could be considered analogous to deep beams;
the ratio of bending to shear stiffness of entire walls and the influence floor and roof systems (tensile restraint) may have on these;

appropriate critical strain values considering pre-existing damage from thermal, self-weight, construction practices, or other aging issues;

the relationship between building stiffness, ground stiffness, and how the building may influence the ground displacement patterns.

- Or, complete back-analysis of a number of representative buildings (of similar age and design) within the vicinity, to better understand the issues listed above.

- Use and modify the strain superposition method, as appropriate for the above factors, to develop a set of closed form equations that may be applied widely to many structures for second-stage building assessments. The strain superposition method, as first published (Boone, 1996), provides a transparent approach for normalizing the equation for deflection of a uniformly loaded deep beam, and, if appropriate values for \( \frac{E}{G} \) or \( \frac{EI}{GA} \), can be determined, the appropriate proportion of bending and shear for any given length of wall can be derived; thus, the constants in Step 5, Figure 3a, would be calibrated to local conditions. If the position of the neutral axis can be better assigned to a position other than the mid-height of the wall, the equation of Step 8, Figure 3a, can also be calibrated to local conditions. Further, if an appropriate critical strain value for entire walls can be discerned that is consistent with local building materials, ages, and condition, then Steps 12 and 13 of Figure 3a, and 5 and 6 of Figure 3b can be applied having been locally calibrated.

- Use a calibrated strain superposition approach within a stochastic model, which can be completed in spreadsheet software, in which the variables associated with tunnel construction control and the building damage assessment process are identified and characterised according to their uncertainty (mean, standard deviation, etc.).

The resulting distributions related to probability of damage can subsequently be used for understanding the cost-benefit of implementing different risk minimization or management strategies.

### 6 CONCLUSIONS

Stochastic analyses offer a powerful tool for examining the risks and potential performance related to complex construction projects, particularly when underground construction has the potential to significantly influence nearby or overlying facilities owned by third parties. It is clearly becoming a requirement of major projects to apply quantitative risk analyses during planning and design. For any quantitative risk analysis and subsequent risk management, the root causes of the risk must be understood. It is suggested that it is no longer acceptable to utilize technical analysis tools that are overly simplified such that they can not be adapted to consider real variability in physical systems. The strain superposition method, as described in this paper, can, by virtue of its direct and transparent analytical procedure to arrive at damage category predictions, be readily incorporated into stochastic risk analyses in which the assumptions about building responses can be locally calibrated and even the uncertainty related to the method accuracy can be considered. By combining probabilistic damage assessments with distributions of likely consequential costs, a more realistic understanding of potential project performance can be developed in support of more informed decisions.
REFERENCES


Evaluating risks of construction-induced building damage for large underground construction projects

S. Boone


ABSTRACT: Los túneles urbanos producen asientos superficiales que suponen movimientos no uniformes en edificios próximos. La principal dificultad para su estudio asociada al terreno, entre varias, es que los movimientos son tridimensionales, están sujetos a variaciones de niveles freáticos y heterogeneidad geotécnica y además varían en el tiempo. La dificultad asociada a los edificios es que cada uno es un prototipo, no hay dos iguales, la respuesta depende de muchos factores (rigidez, geometrías, posición en la cubeta…) y es siempre diferente. Por ello es loable el esfuerzo de los investigadores para ir acotando incertidumbres, horquillando casuísticas y escogiendo variables que nos acerquen al conocimiento del problema (criterios de deformaciones críticas, criterios de daños, modelos de deformación del terreno, interacción suelo – estructura...). En este documento, se plantea la respuesta de las estructuras de hormigón, una vez se han recorrido los pasos de los procedimientos que llevan a considerar tras un filtro y estudio de los edificios de la traza al algún edificio con daños ligeros, moderados o severos. Se realiza entonces un estudio de sensibilidad de la estructura a los asientos previstos, como una herramienta más de apoyo asociada a la instrumentación y al seguimiento de daños.

1 INTRODUCCIÓN

Las obras subterráneas urbanas y las actuaciones que conllevan (túneles, estaciones, pantallas de protección, tratamientos previos…) pueden suponer movimientos no uniformes en los edificios próximos.

La principal dificultad para su estudio, asociada al terreno, es que los movimientos reales son tridimensionales, están sujetos a variaciones del nivel freático, a heterogeneidades geotécnicas y además varían en el tiempo, y dependen de la profundidad y el diámetro del túnel y el espesor del recubrimiento de terreno. También afecta el proceso constructivo (método de excavación, velocidad de avance, desfase en los frentes de túneles paralelos, etc.).

La dificultad asociada a los edificios estriba en que cada uno es un prototipo, no hay dos iguales y su respuesta es siempre diferente.

Por ello es loable el esfuerzo de Burland & Wroth (1974), Boscardin & Cording (1989) y Potts & Addenbroke (1993) para ir acotando incertidumbres, horquillando casuísticas, y escogiendo las variables y conceptos que acerquen al conocimiento del problema (criterios de deformaciones críticas, criterios de daños,).

A partir de esos esfuerzos teóricos los proyectos realizan cálculos de asientos y tratan de clasificar todos los edificios de la traza afectados por la cubeta, dentro de un nivel de daños.
Como la información a nivel de proyecto puede ser escasa, se realizan catálogos de los daños en los edificios afectados (dentro de la cubeta de asientos) tratando de identificar riesgos que complementen los detectados en proyecto.

Antes de comenzar la obra y durante la ejecución de la misma, se plantean las instrumentaciones necesarias para controlar los principales parámetros que definen los riesgos de asientos (piezómetros, inclinómetros, nivelaciones del terreno, instalación de puntos fijos, teodolitos monitorizados con prismas de convergencia en puntos singulares de los edificios, secciones de control en túnel y en el exterior, mediciones de temperatura y humedad, etc.).

Esta instrumentación es muy importante porque valida los modelos de proyecto, o los corrige en función de la respuesta real del terreno. También puede ocurrir que el tipo de movimiento obligue a modificar la instrumentación para, por ejemplo, discernir entre movimientos de sólido rígido o de distorsión.

Conocido el nivel de daños en el edificio, y el asiento previsto con mayor aproximación gracias al aprendizaje de los tramos anteriores, se está en condiciones de realizar el estudio de sensibilidad de la estructura de hormigón a dichos movimientos del terreno.

No obstante, la identificación del riesgo no se puede ceñir solo a la estructura, ya que aunque es importante, es también muy importante que los subsistemas ligados a la misma que van a “avisar” probablemente antes que ella, estén localizados identificados y en su caso instrumentados. Cuando además los edificios tienen daños o éstos pueden producirse, deberán adoptarse las medidas preventivas que se consideren.

2 EFECTOS DE LOS MOVIMIENTOS DEL TERRENO EN ESTRUCTURAS DE HORMIGÓN

De forma resumida los factores que alejan el problema de los esquemas sencillos que nos permiten una aproximación son en lo que se refiere a las estructuras de hormigón:

- Cada estructura es un prototipo, no hay dos iguales.
- Ante acciones exteriores (movimientos del terreno en este caso) funcionan con el principio de la mínima energía.
- En zonas urbanas, muchas estructuras comparten medianera, a veces con cimentaciones diferentes que pueden arrastrar a las vecinas.
- No es fácil modelizar estructuras existentes antiguas, aunque sean de hormigón, porque los materiales son heterogéneos, las uniones pueden conllevar vicios ocultos, y muchas veces simplemente no son accesibles si no se realizan catas de información para su definición. Por otra parte es necesario conocer la idiosincrasia constructiva de las distintas épocas que en el caso de Barcelona es de varios siglos. En cualquier caso no se debe olvidar que los modelos proporcionan resultados que deben entenderse como una aproximación a una realidad más compleja.
- La posición del edificio y su estructura (en este caso de hormigón armado) respecto a la cubeta puede ser un factor determinante.
- El tipo de edificio, que envuelve una gran variedad de estructuras y cimentaciones:
• Cimentaciones superficiales a base de cimientos corridos, zapatas aisladas o arriostradas y losas
• Cimentaciones profundas a base de pozos, pilotes o pantallas.
• Estructuras a base de pórticos de hormigón y forjados unidireccionales; pilares y losas aligeradas o no, estructuras prefabricadas…
• Infraestructuras continuas (túneles, conducciones...)

A pesar de lo anterior, al igual que se ha ido avanzando en la modelización del terreno, también se ha avanzado en la modelización de las estructuras, tratando de acotar los riesgos.

Véase un análisis de sensibilidad general para estructuras aporticadas y ejemplos de estructura cimentada sobre losa o de afectaciones en estructuras continuas como los túneles.

2.1 Efectos sobre estructuras aporticadas

Los movimientos verticales relativos producen variaciones de esfuerzos en jácenas, pilares y forjados (momentos flectores y esfuerzos cortantes en las jácenas y forjados y también esfuerzos axiles en pilares) que pueden aumentar el estado tensional del hormigón y las armaduras en el mismo sentido que las cargas exteriores, pero que también pueden producir una inversión del signo de los esfuerzos. Como consecuencia de ello, cabe esperar la aparición de fisuras o el aumento de ellas, especialmente de los elementos horizontales sometidos a flexión, así como la posible plastificación de las armaduras o el agotamiento del hormigón en compresión, si el descenso relativo es muy grande.

La fisuración en sí misma no reduce la seguridad de la estructura, pues la resistencia depende de la cuantía de armadura y de la capacidad resistente a compresión del hormigón, aunque sí puede afectar a su durabilidad y aspecto estético. Por otra parte, cuando la pieza fisura, reduce sensiblemente su rigidez, por lo que los esfuerzos y las tensiones provocados por los movimientos impuestos del terreno, que son proporcionales a la rigidez, se reducen mucho y con ello el ancho de fisura. Es por ello por lo que se considera que en estructuras de hormigón armado, los movimientos relativos del terreno no son tan preocupantes como en el caso de estructuras a base de muros de carga de mampostería sin armar.

A continuación se realiza un estudio teórico cuyo objetivo es conocer el valor del movimiento relativo permisible en función de los daños que puedan producirse en la estructura, los cuales se asocian a incrementos de ancho de fisura y de tensiones en las armaduras o en el hormigón. Este estudio se realiza tanto para jácenas planas como para jácenas de canto.

Para ello consideraremos el caso de una jácena de un pórtico, de sección rectangular, situada entre dos pilares, entre los cuales se produce un movimiento vertical relativo. Las dimensiones de la sección son b y h, el canto útil es d, la armadura principal de tracción es As situada a una distancia del paramento d’=h-d. Se supondrá que la jácena, de luz de cálculo l, está armada por igual frente a momentos positivos y negativos (es decir que los momentos en centro de vano y en apoyos son iguales, lo cual es frecuente teniendo en cuenta que no existe empotramiento perfecto en los extremos de la jácena) y que su rigidez elástica es EI, siendo I la inercia de la sección I=bh^3/12, y E el módulo elástico del hormigón.

Frente a un descenso relativo de un apoyo de valor δ, el momento flector y el esfuerzo cortante generados son, respectivamente:

En realidad, si la pieza fisura, como será habitual en jácenas de hormigón armado en condiciones de servicio, la rigidez es menor que la elástica, siendo su valor, para el caso de sección rectangular:

$$K_{fs} = E_s A_s (d - x)\left(d - \frac{x}{3}\right)$$  \hspace{1cm} \text{(2)}$$

siendo $x$ la profundidad de la fibra neutra en servicio, que es función de la cuantía geométrica de armaduras, $\rho = A_s / b d$. Para jácenas de canto, las cuantías usuales son reducidas ($\rho \approx 0.005$ a 0.01), mientras que para jácenas planas, con menor canto pero mayor ancho, la cuantía geométrica es algo mayor (del orden de 0.01 a 0.015) pero suele haber armadura de compresión, con lo que la profundidad de la fibra neutra en ambos casos es similar y oscila alrededor de $x/d = 0.30$ que es un valor conservador. Con este valor, la rigidez fisurada valdrá:

$$K_{fs} = E_s A_s (d - 0.3d)\left(d - \frac{0.3d}{3}\right) = 0.63 E_s A_s d^2$$  \hspace{1cm} \text{(3)}$$

El momento flector generado valdrá, por tanto:
Estudios de sensibilidad frente a cubetas de asiento de estructuras de hormigón
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\[ M = \frac{3.78E_s A_s d^2 \Delta \sigma_s}{l^2} = A_s \Delta \sigma_s z \]  (4)

donde

\[ \Delta \sigma_s \] es el incremento de tensión de la armadura de tracción

\[ z = d - x/3 \] es el brazo mecánico, que siendo \( x = 0.30d \), vale \( z = 0.9d \)

Por tanto, despejando \( \Delta \sigma_s \) de la ecuación [4], teniendo en cuenta que \( E_s = 200000 \) Mpa, se obtiene el siguiente incremento de tensión en la armadura de tracción:

\[ \Delta \sigma_s = \frac{3.78E_s A_s d^2 \Delta \sigma_s}{A_s z l^2} = \frac{756000d^2 \Delta \sigma_s}{0.9d l^2} = 840000 \frac{d}{l} \frac{\Delta \sigma_s}{l} \]  (5)

La ecuación anterior relaciona el incremento de tensión en la armadura pasiva con el movimiento relativo de apoyo y con la esbeltez de la pieza. Para jácenas de canto, la relación \( d/l \) puede aproximarse por 1/12.5, mientras que para jácenas planas esta relación está más próxima a 1/22 por ejemplo, en pórticos con luces de 5 m las jácenas de canto pueden ser de 0,35x0,45m, mientras que las jácenas planas pueden ser de 0,70x0,25m. Con ello, la ecuación anterior, que relaciona incremento de tensión en la armadura con asiento diferencial vale:

Para jácenas de canto

\[ \Delta \sigma_s = 70000 \frac{\delta}{l} \]  (6)

Para jácenas planas

\[ \Delta \sigma_s = 40000 \frac{\delta}{l} \]  (7)

La distorsión o asiento diferencial relativo \( \delta/l \) máxima permisible sin que se produzcan daños irreversibles en la estructura vendrá limitada por que se alcance la plastificación de la armadura, por la apertura excesiva de las fisuras o por que se alcance la tensión máxima de compresión en el hormigón.

Para obtener la distorsión correspondiente a estas tres situaciones es preciso relacionar la tensión de la armadura de tracción con la deformación del acero traccionado, el incremento del ancho de fisura y el incremento de deformación del hormigón. Igualmente es necesario evaluar aproximadamente la tensión de trabajo del acero en condiciones de servicio.

- Que no se alcance la plastificación de la armadura \( (\varepsilon_s = \varepsilon_y) \)

La deformación de la armadura y su tensión están relacionadas por el módulo elástico del acero, esto es \( (\varepsilon_s = \sigma_s/E_s) \). La tensión de trabajo en condiciones de servicio, para la situación permanente o transitoria, en la combinación cuasi permanente de cargas, puede estimarse en 180 Mpa. Con esta tensión del acero, el incremento de tensión hasta alcanzar la plastificación de la armadura es:

\[ \Delta \sigma_s = f_{yd} - \sigma_s = 365 - 180 = 185 \text{ MPa} \]  (8)

con lo cual el valor de la distorsión admisible es:

Para jácenas de canto

\[ \frac{\delta}{l} = \frac{185}{70000} = 0.00264 = \frac{1}{375} \]  (9)
Para jácenas planas  \( \frac{\delta}{l} = \frac{185}{40000} = 0.00462 = \frac{1}{215} \)  

- Que no se supere un ancho excesivo de fisura \((w = 0.4 \text{ mm})\)

El ancho de fisura considerado máximo que no debe superarse, por su visibilidad y por el efecto que produce en los habitantes del edificio, es de 0,4 mm. Estos valores mayor del usualmente aceptado en proyecto (para el que adoptaremos 0,20 mm, admisible por razones de durabilidad en ambientes interiores). Teniendo en cuenta ademas, que bajo la acción de deformaciones impuestas el ancho de fisura característico está relacionado con el ancho medio por un factor 1,3 en lugar del 1,7 utilizado para cargas directas, y considerando la fisuración estabilizada (esto es, no se forman más fisuras, sino que se abren las ya existentes y, por lo tanto, la separación media entre fisuras es constante), el incremento de tensión de la armadura de tracción para superar este valor es, para uno y otro tipo de jácena:

\[ \text{Jácenas de canto y planas} \quad \Delta \sigma_{sm} = \frac{\Delta w_k \cdot 1.7 \cdot \sigma_{sm}}{1.3 \cdot w_k} = \frac{0.20 \cdot 1.7 \cdot 180}{1.3 \cdot 0.20} = 235 \text{ Mpa} \]

con lo cual el valor de la distorsión admisible es:

Para jácenas de canto  \( \frac{\delta}{l} = \frac{235}{70000} = 0.00336 = \frac{1}{300} \)  

Para jácenas planas  \( \frac{\delta}{l} = \frac{112}{40000} = 0.00588 = \frac{1}{175} \)  

- Que no se alcance una tensión de compresión superior la resistencia del hormigón

El incremento de tensión en el hormigón y el de la armadura pasiva están relacionados por la siguiente ecuación de compatibilidad:

\[ \Delta \sigma_c = \frac{E_c}{E_s} \cdot \Delta \sigma_s \cdot \frac{x}{d - x} = 0.043 \cdot \Delta \sigma_s = f_{ck} - \sigma_{c0} \]

Siendo \( \sigma_{c0} \) la tensión del hormigón bajo las cargas de servicio, cuyo valor, adoptando un coeficiente de equivalencia entre hormigón y acero de \( n = E_s/E_c = 7.7 \text{es} \):

\[ \sigma_{c0} = \frac{E_s}{E_c} \cdot \sigma_{cs0} \cdot \frac{x}{d - x} = \frac{180}{7.7} \cdot \frac{0.3d}{0.70d} = 10 \text{ Mpa} \]

Con ello, suponiendo que la resistencia del hormigón en el momento actual es de 20 MPa (frente a los 17.5 MPa iniciales en el momento de la construcción), el incremento de tensión en el hormigón es \( \Delta \sigma_c = 20. - 10 = 10 \text{ Mpa} \) y el incremento de tensión en la armadura debe ser:

\[ \Delta \sigma_s = n \Delta \sigma_c \cdot \frac{d - x}{x} = 7.7 \cdot (20 - 10) \cdot \frac{0.7}{0.3} = 180 \text{ MPa} \]

y la distorsión asociada, será:

\[ \text{Para jácenas de canto} \quad \frac{\delta}{l} = \frac{180}{70000} = 0.00257 \approx \frac{1}{390} \]
Para jácenas planas \( \frac{\delta}{l} = \frac{143}{40000} = 0.0035 = \frac{1}{225} \) \( (18) \)

Todo lo anterior se puede resumir en la siguiente tabla:

<table>
<thead>
<tr>
<th>Criterio limitativo</th>
<th>Jácena de canto</th>
<th>Jácena plana</th>
</tr>
</thead>
<tbody>
<tr>
<td>Plastificación armadura</td>
<td>1/375</td>
<td>1/215</td>
</tr>
<tr>
<td>Ancho de fisura w=0,4mm</td>
<td>1/300</td>
<td>1/175</td>
</tr>
<tr>
<td>Tensión hormigón =fck</td>
<td>1/390</td>
<td>1/225</td>
</tr>
</tbody>
</table>

Los anteriores estudios están hechos suponiendo una pieza biempotrada, lo cual es excesivamente conservador. Si en lugar de considerar empotramientos perfectos, se considera la rigidez al giro real del nudo, el empotramiento puede sustituirse por un muelle de rigidez \( K \) y el momento generado por el descenso de apoyo, en lugar de ser el de la expresión \([1]\) es el de la expresión.

\[
M = \frac{6EI\delta}{l^2(1+6\alpha)} ; \quad V = \frac{3EI\delta}{l^3(1+6\alpha)}
\]

\( \alpha = \frac{EI}{Kl} \). En este caso, como \( \alpha > 0 \), el momento se relaja y las condiciones anteriores son menos restrictivas. Considerando la rigidez de los pilares del orden de tres veces la de las vigas, el factor \( \alpha = 1/24 \), con lo que el momento pasa a ser un 20\% menor:

\[
M = \frac{6EI\delta}{l^2(1+6\alpha)} = 0.8 \times \frac{6EI\delta}{l^2} ; \quad V = 0.8 \times \frac{3EI\delta}{l^3}
\]

\( (20) \)

Por tanto la distorsión admisible sería un 25\% mayor. No obstante, habría que considerar el efecto de los movimientos horizontales relativos, que generan tracciones y pueden ser desfavorables.

Los efectos de los movimientos diferenciales horizontales son mucho más difíciles de evaluar, pues dependen de la rigidez de los pilares, de la existencia de vigas de atado en la cimentación y del efecto de los edificios colindantes. No obstante para tenerlos en cuenta, parece razonable limitar los asientos anteriormente calculados a valores algo más estrictos. Se considera, por tanto, que el efecto de estos movimientos compensa, aproximadamente la falta de rigidez de los nudos, quedando los siguientes valores para las distorsiones aceptables:

\[
\text{Para jácenas de canto } \frac{\delta}{l} = \frac{1}{375} \quad \text{(21)}
\]

\[
\text{Para jácenas planas } \frac{\delta}{l} = \frac{1}{225} \quad \text{(22)}
\]

Los valores anteriores han sido verificados mediante un cálculo estructural en régimen no lineal que tiene en cuenta la fisuración y otros fenómenos causantes del comportamiento no lineal del hormigón y de las armaduras. Los resultados obtenidos confirman los valores antes expuestos.
De los mismos se deduce que las estructuras de hormigón no son las primeras en “avisar” del daño pese a que el hormigón para trabajar a tracción se ha microfisurado. Son en general los subsistemas ligados a ellos, (tabiques, cerramientos, solados, …) los que con su mayor fragilidad ante distorsiones tienen limitaciones más estrictas (l/1000 a l/300) y se fisuran antes.

Es el caso de este ejemplo, en el que un pórtico de hormigón de 10 alturas, responde correctamente a unos asientos diferenciales de una cubeta de asientos, pero no así los tabiques atracados a la estructura, que se fisuran como consecuencia de la distorsión.

Figura 2: Sección del pórtico A-A’
2.2 Efectos en losas de cimentación

Se trata de un edificio de dos plantas con estructura a base losas aligeradas (reticular) sobre pilares que se apoyan en una losa de cimentación de hormigón armado. El edificio está adosado a una nave de tipología diferente existiendo una junta entre los dos.
Al realizarse las pantallas laterales de la Estación próximas al edificio y los jet grouting para crear la losa de fondo, se producen asientos en el edificio de varios centímetros, que se detectan a partir de la instrumentación.

El monolitismo de la estructura provoca un movimiento de sólido rígido, mientras los asientos son de poca entidad. A partir de un incremento de asiento, se pueden llegar a producir tracciones en la losa que generan daños en la estructura.

No obstante, los subsistemas ligados a la estructura, y la instrumentación realizada no detectan movimientos significativos más allá de los de sólido rígido.

La modelización y la instrumentación primero y el seguimiento de daños después permiten indicar a partir de que asiento diferencial tendremos excesivas tracciones en la losa, y un posible comportamiento no lineal de la estructura al redistribuirse los esfuerzos como consecuencia de la fisuración, y la respuesta del terreno.

Figura 4: Asentamientos diferenciales en estructura apoyada en losa de cimentación.
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Figura 5: Análisis estructural

Características del terreno de cimentación:
- Limo arcilloso
- \( \gamma_{aparent} \) (hasta 1.10 m) = 1.95 ton/m\(^3\)
- \( \gamma_{sumergida} \) (hasta 1.10 m) = 0.95 ton/m\(^3\)
- \( e_0 = 0.7 \)
- \( \sigma_{adm} = 0.4 \text{ kg/cm}^2 \)

Figura 6: Control de movimientos.

Análisis 2

Caso mas desfavorable: Desplazamiento relativo combinando los puntos de medición en superficie y los puntos de medición sobre el edificio
2.3 Efectos sobre túneles

En los túneles las subsidencias pueden afectar a las vías, ya que las tolerancias para la circulación de trenes pueden ser estrictas. Las afectaciones en la solera se suman a las fisuras de fatiga ya existentes, teniendo que discernir entre daños de asiento y daños de fatiga, y controlando en puntos significativos si las fisuras de fatiga aumentan.
Nuevamente tenemos tres herramientas, cuya definición previa es importante para tener criterio durante su evolución: el estado actual del túnel, (sus soleras, sus juntas la afectación en su caso del balasto,...); la instrumentación planteada y su seguimiento para crear alarmas cuando se sobrepase alguna limitación geométrica; y los estudios de sensibilidad a asientos que definen a priori los riesgos de los movimientos y los limitan.

En el caso presentado, las subsidencias dieron lugar a un aumento del nivel de daños en solera, en especial en la zona de mayor diámetro (Estación) movimientos de las juntas y microfisuras en algún tramo de túnel, pero la respuesta de la infraestructura fue correcta y solo requirió una inyección de las nuevas fisuras procesadas por la subsidencia en la solera.
Figura 11. Daños en la solera.

Figura 12. Detalle de fisuras por fatiga.
3 IDENTIFICACIÓN DE RIESGOS EN ESTRUCTURAS DE HORMIGÓN. EJEMPLOS.

Cuando se realiza un estudio de los daños de un edificio, siguiendo los procedimientos típicos de una patología, se debe recopilar el máximo de información, hacer una toma de datos exhaustiva de los daños, realizar catas y ensayos encaminados a determinar causas de daños existentes, y posteriormente identificar los riesgos que a partir de una correcta definición del estado actual de la estructura y los subsistemas a ella ligados te permitan clasificar situaciones singulares y adoptar las medidas preventivas idóneas.

3.1 Riesgos asociados a operaciones previas

Unos riesgos identificables que son los primeros en aparecer, son los que provienen como consecuencia de operaciones previas:

- Tablestacados o hincado de pantallas que producen vibraciones en las estructuras y sensación de malestar en los usuarios cuando son impactos fuertes o continuados en función del terreno.
  
  Ambas situaciones tienen solución arbitrando medios que varíen las frecuencias naturales de vibración que pueden entrar en resonancia con por ejemplo los forjados de los pisos.

- Inyección de compensación, o “jet grouting” que desplazan el terreno y pueden provocar asientos significativos previos a sumar, a las posteriores consecuencias del
paso de la tuneladora o al vaciado de tierras de la pantalla, y que se tienen que tener en cuenta en la suma final de distorsiones.

- Servicios afectados, o instalaciones no previstas pueden al descalzarse producir daños por escapes de agua, gas, etc.

3.2 Riesgos asociados a los materiales

- Se debería de empezar por definir los límites del hormigón, por ejemplo en las cimentaciones de hormigón ciclópeo u hormigón y mampostería, u hormigón disgregado y lavado, o simplemente inexistencia de cimientos corridos en edificios antiguos.

También los muchos hormigones con resistencias entre 50 y 125 kg/cm² en pilares y otros muchos elementos, pueden dar lugar a respuestas más frágiles ante subsidencias.

3.3 Riesgos asociados a vicios ocultos

- Los vicios ocultos, consecuencia de uniones mal ejecutadas, que nunca estuvieron unidas pero que mantuvieron su “proximidad” mientras no se han producido efectos horizontales significativos o distorsiones, son unos riesgos difíciles de identificar.

Al estar ocultos no permiten actuaciones preventivas en el edificio, salvo que se detecten haciendo catas, etc. y dejen de ser “ocultos”.

- En algunos terrenos como los yesíferos en Aragón, uniones deficientes pueden “desplazar” los pilotes de una cimentación, y provocar redistribuciones en la estructura de soporte.

- Otro ejemplo de vicio oculto son los zunchos perimetrales de atado que no son tales, que se ejecutaban aprovechando los cascotes de los revoltones, y tenían zonas completamente disgregadas que han estado durante más de 50 años mostrando solo fisuras en su perímetro pero respondiendo a las cargas verticales más como un encachado de grava que como un zuncho. Una vez descubierto y entendida la respuesta, aunque sea difícil encajarla en un modelo (pared sobre un medio elástico), se deben realizar mejoras en el elemento (medidas preventivas), por ejemplo inyectando una lechada de cemento, que al menos de consistencia, y mejore la situación previa aunque no la lleve a las actuales exigencias normativas.

3.4 Riesgos asociados a daños

- En estructuras aporticadas, los daños más comunes en la estructura consecuencia de asientos diferenciales del terreno son las fisuras sensiblemente horizontales en pilares de zapata o cimiento profundo centrado, y fisuras en las losas aligeradas a la salida de los capiteles, o de redistribución de esfuerzos en dicha losa. Si se van a producir nuevos asientos en una estructura que se asentó con anterioridad, hay que considerarlos en el modelo y estudiar si es necesario de entrada una medida de refuerzo preventiva.

No hay que olvidar que por lo general los riesgos en estructuras de hormigón vienen precedidos por “avisos” generados con los subsistemas ligados a la estructura que tienen menos ductilidad (remates de fachadas, aplacados que se desadheren con el correspondiente riesgo de caída, tabiques, cerramientos, pavimentos y elementos singulares), y que es necesario instrumentar para su seguimiento.
A efectos del estudio de sensibilidad hay que tratar de incorporar en el modelo las redistribuciones anteriores para sumarlas a los futuros efectos, teniendo en cuenta de esta forma “la historia de cargas” y las etapas sucesivas por las que pasa la estructura.


Figura 15. Cascotes hormigonados en tres capas en un tramo del zuncho.

3.5 Riesgos asociados a usos

En ocasiones las soleras de infraestructuras continuas, o de naves de almacén, no admiten prácticamente desviaciones verticales de las vías, o carriles con tolerancias muy estrictas. Ello obliga a extremar cuidados para evitar asientos diferenciales que obliguen a cortar el servicio o parar la producción.
La presencia de juntas estancas, por un lado ayuda a interpretar movimientos de sólido rígido, pero en ocasiones el movimiento de la junta hace perder la estanquidad por estar el nivel freático a ese nivel en una superficie importante que puede ocasionar ponerfuera de servicio la infraestructura, al menos momentáneamente.

4 CONCLUSIONES

- Los modelos de comportamiento del terreno y los edificios, aun siendo necesarios para acotar riesgos, no dejan de ser una aproximación a una realidad que es general mucho más compleja, y que dificulta la identificación de riesgos.

- Las distorsiones en estructuras aporticadas de hormigón llevan a valores de l/375 a l/225 para hablar de daños, esto significa que tardan en “avisar” en general más que los subsistemas que la rodean en los edificios (tabiques, cerramientos y fachadas, pavimentos, etc).

- Las estructuras de hormigón son en general seguras y con capacidad de redistribución de esfuerzos para los niveles de subsidencias y de distorsiones que generan los diámetros grandes utilizados en Barcelona. No obstante las cubetas de asiento producidas por esos diámetros son incompatibles en general con los subsistemas más frágiles ligados a la estructura como pueden ser los tabiques que con distorsiones de l/500 se señalan, y obligan a los actuales protocolos de actuación preventivos de instrumentación y de seguimiento.

- Hay que transmitir el rigor al técnico, hay que tecnificar la respuesta, para dar tranquilidad los usuarios (que en la actualidad son muy sensibles a los daños como consecuencia de los fallos habidos en las últimas infraestructuras).

- Las estructuras de hormigón hablan a través de sus grietas, y hay que escucharlas, con una correcta instrumentación que permita descifrar su lenguaje.

BIBLIOGRAFIA


ABSTRACT: The assessment of building damage plays an increasingly important role in the design of urban underground construction projects. In this context, the equivalent beam approach pioneered by Burland & Worth (1974) has become widely used. According to this method, each building is represented by a thick beam and the damage is related to the maximum tensile strain that appears in the beam for a given displacement distribution. To estimate the maximum tensile strain, different simplifying hypotheses are usually made about the modes of deformation of the beam. Here we show that the problem of obtaining the strain field in a thick beam for a specific distribution of vertical and horizontal displacements at foundation level can be solved analytically by applying Timoshenko's beam theory, without introducing additional hypotheses. We present several application examples of the analytical procedure and the hypotheses assumed in conventional methods are thereby examined.

1 INTRODUCTION

The performance of an underground excavation in an urban environment usually requires the assessment of the potential damage that such an excavation may produce on adjacent buildings and structures. To estimate the likely damage, the settlement trough caused by the excavation must first be estimated. Afterwards, the magnitude of the damage that this settlement profile can induce on nearby buildings must be evaluated. Usually, the problem is divided into simpler parts that are analyzed separately. Recently, the development of advanced computational tools allows the adoption of more global approaches.

In the London Jubilee Line Extension works, a reference in urban tunnelling, a standard three-stage damage assessment approach was established. With the objective of avoiding complex and unnecessary calculations, the preliminary phase uses a fast evaluation of damage level based on the contours of expected ground surface movements. In the second stage, the maximum tensile strain of each potentially affected building is estimated through the assumption that it may be represented by an equivalent beam. This establishes a certain category of damage for the building. The third stage of detailed analysis is only performed for buildings that, after the previous phases, are ascribed a high risk of damage. This paper focuses on the second stage of this approach.

A major development in the evaluation of damage to buildings due to ground movements was the equivalent beam approach introduced by Burland & Wroth (1974). In this model, the building is represented by an elastic rectangular deep beam (Fig. 1), the foundation of which is assumed to follow a given settlement profile. Burland & Wroth considered two possible modes of deformation: bending and shear. In the first case, the cracks are due to the tensile strains $\varepsilon_b$ close to the beam edge in tension, whereas in the second case the cracks are related to the tensile strain $\varepsilon_d$ caused by shear deformations. The authors use results by...
Timoshenko (1955) and obtain expressions relating the deflection ratio \( DR = \Delta / L \) of a mid-point loaded beam with the maximum bending strain \( \varepsilon_{b,\text{max}} \) and the maximum shear strain \( \varepsilon_{d,\text{max}} \),

\[
\frac{\Delta}{L} = \left( \frac{L}{12h} + \frac{3I}{2hLH G} \right) \varepsilon_{b,\text{max}} , \quad \frac{\Delta}{L} = \left( 1 + \frac{HL^2 G}{18I E} \right) \varepsilon_{d,\text{max}} \tag{1}
\]

where \( h \) is the distance to the neutral axis from the edge of the beam in tension, which usually is chosen as \( h = H \) for a building under hogging (neutral axis at the bottom edge) and \( h = \frac{H}{2} \) for a building under sagging (neutral axis in the middle). For a given deflection ratio, an estimation of the maximum tensile strain in the building can be obtained as the maximum of the values \( \varepsilon_{b,\text{max}}, \varepsilon_{d,\text{max}} \) provided by Equations (1). The concepts underlying this approach constitute to this day the main framework for the evaluation of building damage in geotechnical engineering.

Figure 1: The equivalent beam approach (Burland & Wroth, 1974).

Later, Boscardin & Cording (1989) incorporated in their analysis the effect of the horizontal strains \( \varepsilon_h \) that the excavation can induce in the foundations of the building. They directly superposed this effect to Burland & Wroth’s (1974) bending and shear strains. For the shear strain case, the Mohr’s circle for strains was used under the assumption of plane stress. The incorporation of horizontal strains leads to the expressions:

\[
\varepsilon_{br} = \varepsilon_{b,\text{max}} + \varepsilon_h , \quad \varepsilon_{dr} = \frac{1-\nu}{2} \varepsilon_h + \sqrt{\frac{\varepsilon_h^2}{2} \left( 1 + \frac{\nu}{2} \right)^2 + \varepsilon_{d,\text{max}}^2} \tag{2}
\]

and now the maximum tensile strain in the building is given by the maximum of \( \varepsilon_{br}, \varepsilon_{dr} \).

Geddes (1991) observed, however, that the horizontal strains in the soil surface do not necessarily correspond to the horizontal strains in the foundation, but it would be necessary
to consider the behavior of the soil-structure interface. Thus, in general, Boscardin & Cording’s (1989) procedure tends to overestimate the horizontal strain.

The value for the maximum tensile strain obtained from Equations (1) or (2) places the building into a specific category of damage (Fig. 2). Categories 0, 1 and 2 relate to aesthetic damage, categories 3 and 4 relate to serviceability damage and category 5 corresponds to damage affecting stability.

For the application of this approach, the parts of the building under hogging and under sagging are often treated as separated beams, and each part has its own deflection ratio, extreme value of horizontal strain and geometric and constitutive properties (Fig. 3, left). For each given set of values \( h, \frac{L}{H}, \frac{E}{G} \), Equations (1) and (2) can be used to produce diagrams relating category of damage with horizontal strain and deflection ratio. For instance, the diagram shown in Figure 3 (right) corresponds to the case \( \frac{L}{H} = 1, \frac{E}{G} = 2.6 \) (isotropic beam with \( \nu = 0.3 \)) and \( h = H \) (hogging).

<table>
<thead>
<tr>
<th>Category of damage</th>
<th>Normal degree of severity</th>
<th>Limiting Tensile strain [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>Negligible</td>
<td>0 - 0.05</td>
</tr>
<tr>
<td>1</td>
<td>Very slight</td>
<td>0.05 - 0.075</td>
</tr>
<tr>
<td>2</td>
<td>Slight</td>
<td>0.075 - 0.15</td>
</tr>
<tr>
<td>3</td>
<td>Moderate(^a)</td>
<td>0.15 - 0.3</td>
</tr>
<tr>
<td>4 to 5</td>
<td>Severe to Very Severe</td>
<td>&gt;0.3</td>
</tr>
</tbody>
</table>

Figure 2. Category of damage and tensile strain (after Boscardin& Cording, 1989).

Boscardin & Cording (1989) produced similar diagrams relating category of damage to horizontal strain and relative rotation, \( \beta \) (instead of deflection ratio). Relative rotation is the rotation of the line joining two reference points of the settlement profile, relative to the tilt of the building, which describes the rigid body rotation of the whole superstructure or a well defined part of it. Sometimes the tilt is difficult to identify accurately, so that the precise evaluation of \( \beta \) can be uncertain on occasions (Burland et al., 2004).

Potts & Addenbrooke (1997) carried out a numerical study to evaluate the influence of the stiffness of buildings on the strains that the excavation of a tunnel can produce on them. Those authors proposed the use of four modification factors in the equivalent beam approach (two for the deflection coefficient under hogging or sagging and two for the horizontal strain under hogging or sagging). Those factors depend on the stiffness of the soil and of the building, and are applied to the deflection ratio and the horizontal strain before the application of Equations (1) and (2). Franzius (2003) extended the parametric analyses carried out by Potts & Addenbrook (1997). He considered the effects of building weight, of the presence of a soil-structure interface and of the three-dimensional nature of the tunnel excavation process. In particular, Franzius’ (2003) results confirm Geddes (1991) claim that the soil-structure interface can reduce considerably the horizontal strain in the building.

In recent years several soil-structure interaction problems have been analyzed by means of complex three-dimensional numerical models in which buildings are represented more accurately than in the equivalent beam approach. The effort that such studies require makes them still impractical for routine damage evaluations; they can however be useful during the third stage of the damage assessment process.
2 AN ANALYTICAL APPROACH

The application of Equations (1) and (2) in the equivalent beam approach is based on a series of hypotheses. It is assumed that the maximum tensile strain is insensitive to the form of loading, so that the expressions for the deflection ratio of a centrally loaded beam can be used in general. Moreover, it is assumed that the parts of the building under hogging and sagging can be treated as independent beams. In addition, it is necessary to make an independent decision about the position of the neutral axis in each part of the building, and, finally, values of strain that correspond to different points of the beam are combined in the same Mohr’s circle. Boscardin & Cording’s diagrams are based on relative rotations, which can be defined at each point of any particular settlement profile. However a location for the neutral axis is still assumed and the evaluation of the tilt is, often, far from straightforward. The equivalent beam approach has become widely used, but, sometimes, the hypothesis and considerations just mentioned makes its application in practice somewhat uncertain.

Our aim here is the development of an alternative procedure for the estimation of the maximum tensile strain induced in a building by a given settlement and horizontal displacement profile. Again, the procedure is based on the assumption that the building is equivalent to a Timoshenko’s beam, but no additional hypotheses are included. Thus, we consider the problem of calculating the maximum tensile strain in a deep beam for a given deflected shape of its lower edge. In this section we use Timoshenko’s beam theory to obtain an analytical expression for the strain field at each point on the beam. In Section 3 we present several application examples of this procedure and review some of the hypotheses adopted in the classical procedure.
As usual, we represent the building by a rectangular beam of length $L$ and height $H$. We assume that the longitudinal axis that defines the geometry of the beam is located in the middle of the beam (Figs. 4 and 9). We fix the origin of the axes at the left-hand side of the building in such a way that axis $x$ coincides with the longitudinal axis and axis $z$ is vertical. The tunnel advances in the direction of axis $y$. The displacements of a point of the beam in the direction of axes $x,y,z$ are denoted by $u, w, v$, respectively, and are positive if they go in the direction given by the corresponding axis. The sign of the stresses is such that the cross-sectional forces in Figure 5 are positive. We assume that all geometric and constitutive properties of the beam are independent of $x$. If $A$ represents the area of a section of the beam and $I$ is its second moment (inertia), then we can assume that $A \propto H$ and $I \propto H^3$, thus $\frac{A}{I} = \frac{\alpha}{H^2}$, where $\alpha$ is a dimensionless geometric parameter that characterizes the beam. Finally, if $g$ is a function in the variable $s$, then we denote by $g_s, g_{ss}$ the first and second derivatives of $g$ with respect to $s$.

### 2.1 Timoshenko's beam theory

We work with the following set of hypotheses for the building (Timoshenko, 1955):

- **Displacements**: $v(x, y, z) = v(x), w(x, y, z) = 0$.
- **Navier’s hypothesis**: plane sections that are normal to the longitudinal axis remain plane after deformation.
- **Small strains**.
- **Constitutive hypothesis**: anisotropic linear elasticity with homogeneity in directions $y,z$ (we also assume homogeneity in direction $x$). Parameters $E, G, \nu$ represent Young modulus in direction $x$, shear modulus in direction $z$ and Poisson’s ratio for strains in direction $z$ due to strains in direction $x$.

Under those conditions, a generic section $SS'$ becomes section $TT'$, and the displacements of the beam can be described by means of the horizontal and vertical movements of the longitudinal axis, $u_0(x), v_0(x)$, and the rotation $\theta(x)$ (Fig. 4). In particular, we have

$$u(x, y, z) = u(x, z) = u_0(x) - z\theta(x)$$  \hspace{1cm} (3)

and the relationship between $u_0, v_0$ and the horizontal and vertical displacements of the lower extreme fiber of the beam, $\bar{u}, \bar{v}$, is

$$u_0 = \bar{u} - \theta \frac{H}{2}, \quad v_0 = \bar{v}$$  \hspace{1cm} (4)

The strains are given by

$$\varepsilon_x = \bar{u}_x - \theta \left( z + \frac{H}{2} \right), \quad \gamma_{xz} = \bar{v}_x - \theta, \quad \varepsilon_y = \varepsilon_z = \gamma_{yx} = \gamma_{yz} = 0$$  \hspace{1cm} (5)
Figure 4. Variables characterizing the deformation of the beam.

The forces acting on a section are obtained as

\[
N = \frac{\mu}{2} \int_{-H/2}^{H/2} \sigma_x \, dz = EA \left( u - \frac{\theta H}{2} \right),
\]

\[
M = \frac{\mu}{2} \int_{-H/2}^{H/2} z \sigma_x \, dz = EI \theta_x,
\]

\[
Q = \frac{\mu}{2} \int_{-H/2}^{H/2} \tau_{xy} \, dz = kGA \left( v - \theta \right)
\]

The coefficient \( k \) is introduced to take into account the variation of the tangential stresses through the section of the beam. This point is considered in subsection 2.4.

### 2.2 Equilibrium equations

Let us consider the differential element of the beam showed in Figure 5. The element is in equilibrium under the action of the cross-sectional forces, its own weight \( \gamma \) and the interaction forces between the soil and the building. These forces can be decomposed in a vertical reaction \( r \) and a tangential stress \( t \).

The equilibrium equations of the element are

\[
\sigma_x = E \varepsilon_x, \quad \tau_{xz} = G \gamma_{xz}, \quad \sigma_y = \sigma_z = \tau_{xy} = \tau_{yz} = 0
\]
Estimation of building damage due to tunnelling: an analytical approach

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\[ N + dN + dX = N \Rightarrow N = -E A \left( \frac{H}{2} - \bar{u}_x \right) \]

\[ Q + dQ + (r - \gamma) dx = Q \Rightarrow Q = \gamma - r \]

\[ M + dM + (Q + dQ) dx + (r - \gamma) dx \frac{dx}{2} + t dx \left( \frac{H}{2} \right) = M \Rightarrow \]

\[ M_x + Q + \frac{H}{2} = 0. \]

\[ \frac{\partial}{\partial x} \left( \frac{H x}{2} \right) \]

\[ \frac{\partial}{\partial x} \left( \frac{H x}{2} \right) \]

\[ \frac{\partial}{\partial x} \left( \frac{H x}{2} \right) \]

Figure 5. Forces acting on a differential element of the beam.

The combination of equations (7) and (8) leads to the following set of differential equations:

\[ k G A \left( \bar{v}_{xx} - \bar{u}_x \right) = \gamma - r \]

\[ \bar{u}_{xx} - k \frac{4 \alpha}{4 + \alpha} \frac{G}{E H^2} \theta = \frac{4 \alpha}{4 + \alpha} \left( \bar{u}_{xx} - k \frac{G}{E H^2} \bar{v}_x \right) \]

We focus now in the latter equation and introduce the dimensionless variable \( \hat{x} = x/L \), \( 0 \leq \hat{x} \leq 1 \), and the function \( \hat{\theta} \), defined as \( \hat{\theta}(\hat{x}) = \theta(x) \). In this case, we have \( L \theta_x(x) = \hat{\theta}(\hat{x}) \) and \( L^2 \theta_{xx}(x) = \hat{\theta}_{\hat{x}\hat{x}}(\hat{x}) \), and the equilibrium equation for the bending moments can be rewritten in the form

\[ \hat{\theta}_{\hat{x}\hat{x}} - K_1 \hat{\theta} = f(\hat{x}) \]

where

\[ K_1 = k \frac{4 \alpha}{4 + \alpha} \frac{G}{E H^2} \left( \frac{L}{H} \right)^2 \]

and
2.3 Solving the bending moment equilibrium equation

The general solution of Equation (10) is

\[ \hat{\theta}(\hat{x}) = C_1 \text{sh} \sqrt{K_1} \hat{x} + C_2 \text{ch} \sqrt{K_1} \hat{x} + \frac{1}{\sqrt{K_1}} \int_0^{\hat{x}} f(s) \text{sh} \sqrt{K_1}(\hat{x} - s) \, ds \]  

see, for instance, Spiegel et al. (2000).

For the determination of the coefficients \( C_1, C_2 \) it is necessary to prescribe some boundary conditions for \( \hat{\theta} \). Note that only two boundary conditions can be prescribed. Therefore, it is not possible in general to prescribe that the bending moments and also the shear forces are zero at \( x = 0, x = L \).

If we assume that the bending moments at \( x = 0, x = L \) are zero, then we obtain

\[ \hat{\theta}(\hat{x}) = \frac{1}{\sqrt{K_1}} \left( \int_0^{\hat{x}} f(s) \text{sh} \sqrt{K_1}(\hat{x} - s) \, ds - \frac{\text{ch} \sqrt{K_1} \hat{x}}{\text{sh} \sqrt{K_1}} \int_0^{\hat{x}} f(s) \text{ch} \sqrt{K_1}(\hat{x} - s) \, ds \right) \]

\[ \hat{\theta}''(\hat{x}) = \int_0^{\hat{x}} f(s) \text{ch} \sqrt{K_1}(\hat{x} - s) \, ds - \frac{\text{sh} \sqrt{K_1} \hat{x}}{\text{sh} \sqrt{K_1}} \int_0^{\hat{x}} f(s) \text{ch} \sqrt{K_1}(\hat{x} - s) \, ds \]  

\[ \hat{\theta}''''(\hat{x}) = K_1 \hat{\theta}'(\hat{x}) + f(\hat{x}) \]  

However, those expressions must be handled with care when the quotient \( \frac{E}{G} \) increases. If we take \( \frac{G}{E} \to 0 \) then Equation (10) becomes

\[ \hat{\theta}''''(\hat{x}) = \frac{4\alpha}{4 + \alpha} \frac{u_{xx}(\hat{x}L)^2}{2H} \]  

This implies that \( \hat{\theta}''''(\hat{x}) = \frac{4\alpha}{4 + \alpha} \frac{u_{xx}(\hat{x}L)^2}{2H} + B \). In this case, if \( u_{xx}(0) \neq 0 \) or \( u_{xx}(L) \neq 0 \), then it is not possible to find a constant \( B \) compatible with both conditions \( \hat{\theta}''''(0) = \hat{\theta}''''(1) = 0 \).

This problem can be avoided by prescribing null shear forces at \( x = 0, x = L \), which leads to

\[ C_1 = \frac{-\bar{v}_x(L) - \bar{v}_x(0) \text{ch} \sqrt{K_1} - \frac{1}{\sqrt{K_1}} \int_0^1 f(s) \text{ch} \sqrt{K_1}(\hat{x} - s) \, ds}{\text{sh} \sqrt{K_1}}, \quad C_2 = \bar{v}_x(0) \]
In any case, for the range of variation of \( \frac{E}{G} \) that is normally considered; i.e., \( 0.5 < \frac{E}{G} < 12.5 \), Equations (14) can be used without difficulty. In the examples presented in this paper, we have employed the values of \( \hat{\theta} \) given by Eqs. (14), as it appears more realistic to consider null bending moments at the extreme beam boundaries.

### 2.4 Shear stress and strain distribution

According to the hypotheses of Timoshenko's theory, the angular strain \( \gamma_{xz} \) (and hence the shear stress \( \tau_{xz} \)) does not depend on \( z \). In general this assumption does not agree with the fact that the continuity conditions \( \tau_{xz} \left( x, \frac{H}{2} \right) = 0 \), \( \tau_{xz} \left( x, -\frac{H}{2} \right) = -t \) must hold. Actually, the distribution of the shear stresses is not uniform in each section, and the value \( G \left( \bar{v}_e - \hat{\theta} \right) \) that appears in Timoshenko's equations can be interpreted as a kind of average measure of this distribution.

![Figure 6. Equilibrium of horizontal forces for a rectangular section.](image)

The real distribution of \( \tau_{xz} \) depends on each particular section. If we consider the case of a rectangular section \( (\alpha = 12) \) and we assume that \( \tau_{xz} \) does not depend on \( y \), we can write the equilibrium equation (see Fig. 6)

\[
\int dx + \tau_{xz} dx + \int_{-\frac{H}{2}}^{\frac{H}{2}} (\sigma_x + d\sigma_x) dz = \int_{-\frac{H}{2}}^{\frac{H}{2}} \sigma_x dz,
\]

which leads to the parabolic distribution
\[ \tau_{xz} = E \left( \frac{H}{2} - z \right) \left( -u_{xz} - \frac{\theta_{xz}}{2} \left( z + \frac{3H}{2} \right) \right). \] (18)

The coefficient \( k \) that has been introduced in Equation (7) modifies the area of the section to take into account that the real distribution of shear stresses is not constant. There are different methods to define this parameter. One of the most commonly used is that of computing the area \( A^* \) of a virtual section that, under the action of a uniform shear stress distribution, would have the same shear deformation energy than the actual section \( A \) has under the action of the variable shear stress. The shear stress that acts on the virtual section is taken in such a way that the shear force \( Q \) is the same for \( A \) and for \( A^* \). Then, the coefficient \( k \) is defined as \( k = \frac{A^*}{A} \).

Let us consider a rectangular section with \( A = 1 \) that is under the action of a parabolic shear stress distribution \( \tau_{xz} = (0.5 - z)(z + a) \), see Figure 7 (left). Then, the average shear stress acting on \( A \) is \( \frac{6a - 1}{12} = Q \) and the shear stress acting on \( A^* \) must be \( \frac{6a - 1}{12A^*} \). The shear deformation energy of \( A \) and \( A^* \) are,

\[ \frac{1}{2} \int_0^1 \tau_{xz}^2 \, dz = \frac{10a^2 - 5a + 1}{30G} \quad \text{and} \quad \frac{36a^2 - 12a + 1}{144G A^*}, \]

respectively, which leads to

\[ k = \frac{15 \cdot 36a^2 - 12a + 1}{72 \cdot 10a^2 - 5a + 1}, \] (19)

When \( a = -0.5 \) we obtain the well-known result \( k = \frac{5}{6} \). The variation of \( k \) is strong around \( a = 0 \) (Fig. 7, right), and \( k \rightarrow \frac{3}{4} \) when \( |a| \rightarrow \infty \).

Figure 7. Variation of the coefficient \( k \) with \( a \).
In general, we have \( a = \frac{2u_{xx}}{\theta_{xx}} - \frac{3H}{2} \) -Equation (18)- and the coefficient \( k \) varies with \( x \), which is not consistent with the original hypotheses. Taking \( k(x) = \frac{3}{4} \) can be a reasonable solution, although when the sections lie in the range \(-0.5 \leq a \leq 0.3\) there could be considerable energy deviations (see subsection 3.2).

Again, the case of very high \( \frac{E}{G} \) ratios requires a specific comment. Note that the angular strain \( \gamma_{xz} \) given by Equation (18) is proportional to that ratio, so that \((\epsilon_1)_{\text{max}} \rightarrow \infty\) when \( \frac{G}{E} \rightarrow 0 \). In these cases, a reasonable solution could be that of computing the angular strain as \( \gamma_{xz} = v_x - \theta \) -Equation (5)-.

### 2.5 The maximum tensile strain

Timoshenko’s hypotheses imply \( \epsilon_z = 0 \). However, once the distribution of horizontal strains has been obtained, we can consider Poisson’s effect to compute the major principal strain at every point. Under the assumption of plane stress we have \( \epsilon_z = -\nu \epsilon_x \) and, as a consequence,

\[
\epsilon_1 = \frac{1}{2} \left( \epsilon_x (1-\nu) + \sqrt{\epsilon_x^2 (1+\nu)^2 + \gamma_{xz}^2} \right),
\]

see Figure 8.
Thus, in the case of rectangular section we have

\[
\varepsilon_x (\hat{x}, \hat{z}) = \bar{u}_x (\hat{x}L) - \hat{\theta}_{x} \left( \frac{H}{L} \left( \frac{\hat{z} + \frac{1}{2}}{2} \right) \right)
\]

\[
\gamma_{xz} (\hat{x}, \hat{z}) = \frac{E}{G} \left( \frac{H}{L} \right) \left( \frac{1}{2} - \hat{z} \right) \left( \frac{L^2 \bar{u}_x (\hat{x}L)}{H} - \hat{\theta}_{xi} \left( \frac{\hat{z} + \frac{3}{2}}{2} \right) \right),
\]

(21)

where the dimensionless variable \( \hat{z} = \frac{z}{H} \) has been introduced: For each particular building represented by the set of parameters \( \left( E, G, L, H, \nu, \alpha = 12, k = \frac{3}{4} \right) \) and for each foundation displacement distribution \( (\bar{u}, \bar{v}) \), the application of Equations (11,12,14,20,21) provides us with the complete field of major principal strains. The maximum tensile strain in the building can be then obtained just by varying \( \hat{x}, \hat{z} \in [0,1] \).

### 2.6 Surface displacements due to tunneling

As it is generally the case in the application of this type of methods, it is assumed that the building foundation follows precisely the movements of the ground. The expressions presented above can be applied to any assumed or measured distribution of horizontal and vertical ground displacements.

However, for damage evaluation exercises before a tunnelling excavation is performed, it is necessary to estimate the distribution of vertical and horizontal movements. The proposals of Peck (1969) for the settlement trough and of O’Reilly & New (1982) for horizontal displacements caused by tunnelling have proved practical in this respect. They are summarised here for completeness.

Peck’s (1969) estimate establishes that the distribution of vertical surface displacements can be described by means of a Gaussian function (Fig. 9):

\[
\bar{v}(x) = -\delta_{\text{max}} e^{-\frac{1}{4} \left( \frac{x+x_i}{R} \right)^2}
\]

(22)

The parameter \( i \) can be estimated as \( Kz_0 \), where \( z_0 \) is the depth of the tunnel axis. The coefficient \( K \) lies in the range between 0.4 and 0.7 for different types of soil, see for instance Kimura&Mair (1981), O’Reilly&New (1982).

Under those conditions, the area bounded by the settlement trough is \( \sqrt{2\pi i \delta_{\text{max}}} \). In undrained conditions, this area equals the contraction that the tunnel section suffers during the excavation; i.e. the volume loss \( V_L \), that is usually expressed as a percentage of the theoretical excavation section. Thus, if the tunnel is circular with diameter \( D \) we have

\[
\delta_{\text{max}} = \sqrt{\frac{\pi V_L D^2}{32 \ 100i}}
\]

(23)
Figure 9. Variables associated with ground movement estimates by Peck (1969) and O’Reilly & New (1982).

If we assume that the displacement vectors of the points of the soil surface point towards the center of the tunnel (O’Reilly & New, 1982), we obtain

\[ \overline{u}(x) = \frac{\sigma_{\text{max}}}{z_0} (e - x)e^{-\frac{(x-x)^2}{2\alpha^2}} \]  

(24)

Finally, introducing the dimensionless variables \( \hat{e} = \frac{e}{L} \), \( \hat{i} = \frac{i}{L} \), we obtain

\[ \overline{v}_x(\hat{x}L) = \frac{\pi}{32} \frac{V_L}{100} \left( \frac{D}{L} \right)^2 \hat{x} - \hat{e} \frac{e^{-\frac{(x-x)^2}{2\alpha^2}}}{i^2} \],

\[ \overline{u}_x(\hat{x}L) = \frac{\pi}{32} \frac{V_L}{100} \left( \frac{D^2}{L} \right) \frac{1}{i} \left( \frac{\hat{x} - \hat{e}}{i} - 1 \right) e^{-\frac{(x-x)^2}{2\alpha^2}} \],

\[ \overline{u}_{xx}(\hat{x}L) = \frac{\pi}{32} \frac{V_L}{100} \left( \frac{D^2}{L} \right) \left( \frac{\hat{x} - \hat{e}}{i^3} \right) \left( 3 - \left( \frac{\hat{x} - \hat{e}}{i} \right)^2 \right) e^{-\frac{(x-x)^2}{2\alpha^2}} \]  

and

\[ f(\hat{x}) = \frac{\pi}{32} \frac{4\alpha}{4 + \alpha} \frac{V_L}{100} \left( \frac{\hat{x} - \hat{e}}{i} \right) \left( \frac{D^2}{L} \right)^2 \left( 3 - \left( \frac{\hat{x} - \hat{e}}{i} \right)^2 \right) - k \frac{G}{E} \left( \frac{D}{H} \right)^2 e^{-\frac{(x-x)^2}{2\alpha^2}} \].

(26)

This type of ground surface movement distribution will be called Gaussian in the rest of the paper. In this case, the deformation of the building is governed by the following geometric dimensionless parameters: 

\[ \frac{L}{H}, \frac{D}{H}, \frac{D^2}{Hz_0} \].
3 EXAMPLES OF APPLICATION

In this section we present some application examples of the approach presented. In some cases, they are compared to the results from the classical approach given by Equations (1) and (2). For the application of those equations to a rectangular beam with neutral axis in the middle (sagging) we take \( I = \frac{H^3}{12} \), \( h = \frac{H}{2} \) and obtain

\[
\varepsilon_{b,\text{max}} = \frac{\Delta / L}{0.167 \frac{L}{H} + 0.25 \frac{E}{G} \frac{H}{L}}, \quad \varepsilon_{d,\text{max}} = \frac{\Delta / L}{1 + 0.67 \frac{G}{E} \left( \frac{L}{H} \right)^2}.
\]  

(27)

For the case of hogging it is assumed that the neutral axis is the lower extreme fibre and \( h = H \). In this case, it is usual to take \( I = \frac{H^3}{3} \), that is the second moment of inertia of a rectangular beam with respect to an extreme fiber. This implies

\[
\varepsilon_{b,\text{max}} = \frac{\Delta / L}{0.083 \frac{L}{H} + 0.5 \frac{E}{G} \frac{H}{L}}, \quad \varepsilon_{d,\text{max}} = \frac{\Delta / L}{1 + 0.167 \frac{G}{E} \left( \frac{L}{H} \right)^2}.
\]  

(28)

When \( \frac{E}{G} = 2.6 \), one obtain the expressions

\[
\varepsilon_{b,\text{max}} = \frac{\Delta / L}{0.083 \frac{L}{H} + 1.3 \frac{H}{L}}, \quad \varepsilon_{d,\text{max}} = \frac{\Delta / L}{1 + 0.064 \left( \frac{L}{H} \right)^2}
\]  

(29)

as presented by Boscardin & Cording (1989) and Franzius (2003).

Alternatively, it could be argued that the second moment of inertia is a geometric parameter, and it should not depend on the loading state. For a rectangular beam it would be \( I = \frac{H^3}{12} \) irrespective of the position of the neutral axis. When the neutral axis is in the lower fibre (hogging), this gives

\[
\varepsilon_{b,\text{max}} = \frac{\Delta / L}{0.083 \frac{L}{H} + 0.125 \frac{E}{G} \frac{H}{L}}, \quad \varepsilon_{d,\text{max}} = \frac{\Delta / L}{1 + 0.67 \frac{G}{E} \left( \frac{L}{H} \right)^2}
\]  

(30)

3.1 Computed strain field

For illustration purposes, let us start by considering a generic arbitrary case in which the settlement trough is given by

\[
\bar{v}(x) = \frac{0.02}{L'} \left( 5x^3 - 9Lx^2 + 4L^2 x \right), \quad \bar{u}(x) = \frac{0.01}{L'} x^2
\]  

(31)
see Figure 10 (bottom left). We choose the set of parameters $E = G, L = 40, H = 20, \nu = 0.3$. Lengths are expressed in meters.

Figure 10 (top left) displays the contours of major principal strains that were obtained by applying the above procedure. The maximum tensile strain in the building is $(\varepsilon_1)_{\text{max}} = 0.09954\%$ (category of damage: 2, Slight) and it occurs at point $(x, y) = (0, -10)$. Similar maps can be plotted for horizontal strains, angular strains, etc. The presentation of the results by means of three-dimensional diagrams may also be of interest (Fig. 10, right).

The inflexion point of the settlement trough is placed at $x = 24 \, m$. This point determinates the hogging and sagging zones for the application of the conventional method. For the sagging zone we obtain a deflection ratio of 0.0154\% and a maximum tensile horizontal strain of 0.0497\%, which gives a maximum tensile strain of 0.083\% -Eq. (27)-; that is, category of damage 2, Slight. For the hogging zone we have a deflection ratio of 0.0346\% and a maximum tensile horizontal strain of 0.0297\%. With these data Equation (28) gives a maximum tensile strain of 0.113\% (Category 2, Slight), whereas Equation (30) gives a maximum tensile strain of 0.183\% (Category 3, Moderate).

If we consider, for instance, a Gaussian surface displacement distribution (typical of tunnel excavation) and adopt the parameters $E/G = 2.6, L = 30, H = 30, D = 10, z_0 = 25, e = 40, i = 0.5z_0, V_e = 1\%$, $\nu = 0.3$, we obtain the contour map showed in Figure 11. The maximum tensile strain in the building is $(\varepsilon_1)_{\text{max}} = 0.1758\%$ (category of damage: 3, Moderate) and it occurs at point $(29.1, -15)$. Equation (28) gives for this case $(\varepsilon_1)_{\text{max}} = 0.0578\%$ (Category 1, Very slight) and Equation (30) gives $(\varepsilon_1)_{\text{max}} = 0.089\%$ (Category 2, Slight).

Thus, in general, one can find some differences between the results obtained with our analytical approach and those obtained with the conventional method. This can be explained by the fact that the superposition of two generic curves of vertical and horizontal displacements in the lower edge may induce strain states in a beam that are not necessarily
similar to those corresponding to a beam under either a central point load or under a uniformly distributed load (see Figs. 10 and 11).

\[
\varepsilon_1(xL) = \frac{q}{E} \left( \frac{L}{H} \right)^2 \left( 6x^2 - 4x^3 - \frac{1}{2} \right), \quad \varepsilon_2(xL) = \frac{q}{2GH} \left( \frac{L}{H} \right)^2 \left( x - \frac{1}{2} \right), \quad \varepsilon_3(xL) = \frac{3q}{E} \left( \frac{L}{H} \right)^2 \left( 1 - x \right),
\]

(32)

We have adopted the parameters \( q = 100, E = 260000, G = 100000, L = 40, H = 30, \nu = 0.3 \).

In this context, Figure 12 shows the field of horizontal strains, angular strains and major principal strains for the case of a beam under uniformly distributed load. The figure has been obtained by applying our procedure with the following displacement input (and \( k = \frac{2}{3} \))

\[
\hat{\varepsilon}_1(xL) = \frac{q}{E} \left( \frac{L}{H} \right)^2 \left( 6x^2 - 4x^3 - \frac{1}{2} \right) + \frac{3q}{2G} \left( \frac{L}{H} \right)^2 \left( x - \frac{1}{2} \right),
\]

\[
\hat{\varepsilon}_2(xL) = \frac{3q}{E} \left( \frac{L}{H} \right)^2 \left( 1 - x \right),
\]

(33)

Figure 12. Horizontal strains (left), angular strains (center) and major principal strains (right) in a beam under uniformly distributed load. The neutral axis is in the middle.

In this case, the computed distribution of \( \hat{\varepsilon}(\hat{x}) \) coincides exactly with the theoretical one:
Therefore, the strains computed with the proposed method coincide, as required, with the classical results for a Timoshenko beam under uniform applied load.

### 3.2 The neutral axis and the coefficient $k$

Equations (21) allows us to obtain the location of the neutral axis, $z_N$, and the fibre where the shear strain vanishes at each section of the building, $a$. The corresponding expressions are

$$
z_N = \frac{\bar{u}_x(\hat{x}L)}{\hat{\theta}_x(\hat{x})} L - \frac{H}{2}, \quad a = \frac{2L^2 \bar{u}_{xx}(\hat{x}L)}{\hat{\theta}_{xx}(\hat{x})} - \frac{3H}{2} \tag{34}
$$

 Naturally, the position of the neutral axis can exhibit a singularity at the points where $\hat{\theta}_x = 0$, or, equivalently, $M = 0$.

Figure 13 (left) shows the variation of $z_N, a$ with $x$ for the building corresponding to the first case analyzed in the previous subsection; i.e., $L = 40, H = 20$. It can be observed that the neutral axis is at the lower extreme fiber only for $x = 0$, where $(u_x)_x(0) = 0$. The neutral axis is below the lower edge of the beam in the rest of the zone under hogging, and it is above the upper edge of the beam in the zone of sagging. The curve described by $a$ has two singularities, which correspond to points where $\hat{\theta}_{xx} = 0$. Only a small fraction of the sections are in the region $-0.5H \leq a \leq 0.3H$, so the hypothesis $k = \frac{3}{4}$ can be considered valid for this building.

![Figure 13](image)

**Figure 13.** Position of the neutral fibre (solid line) and the fibre where the shear strain vanishes (dashed line) for the case $L = 40, H = 20$ (left) for the case $L = 30, H = 30$ (right).

Figure 13 (right) shows the variations of $z_N, a$ with $x$ for the building corresponding to the second case analyzed in the previous subsection; that is, $L = 30, H = 30$. In this case,
$M = 0$ only in $x = 0, x = L$, and the position of the neutral axis has no singularities inside the building. A considerable portion of the building satisfies the condition $-0.5H \leq a \leq 0.3H$. This suggests the introduction of a coefficient $k$ variable in the analysis, which would imply the use of iterative methods to solve the problem. Although this could open a new interesting research avenue, the choice $k = \frac{3}{4}$ is probably sufficient in practice. For this example, we have computed the maximum tensile strain for different choices of the value of the coefficient $k$, assumed to be constant in the entire building. For the values $k = \frac{1}{3}, \frac{1}{2}, \frac{2}{3}, \frac{3}{4}, \frac{5}{6}, 1$ we obtained the following tensile strains: $(\varepsilon_i)_{\text{max}} = 0.1513\%, 0.1614\%, 0.1711\%, 0.1758\%, 0.1804\%, 0.1896\%$. In this case the result is quite insensitive to the value of the coefficient $k$, and, moreover, the selection of high values for this coefficient appears to be a safe choice. For the first example the maximum tensile strains corresponding to the same values of $k$ were $(\varepsilon_i)_{\text{max}} = 0.06011\%, 0.07457\%, 0.09153\%, 0.9954\%, 0.1073\%, 0.1221\%$. In this case the influence of $k$ is greater, but, again, the high values of $k$ are conservative in relation to the estimation of the building damage.

### 3.3 Influence of the settlement curve shape

Burland & Wroth (1974) used Timoshenko’s beam theory to study the relationships between the deflection ratio and the maximum bending and shear strains in a beam under the action of a mid-point load and a uniformly distributed load. They observed that in both cases the expressions obtained were similar, and concluded that those relationships were likely to be insensitive to the form of loading and the precise deflected shape. In this subsection we study the variation of the maximum tensile strain obtained with our approach for buildings under different settlement curves but with the same deflection ratio.

Let us consider the case given by

$$\tilde{\nu}(x) = \delta \left(1 - \left[\left(\frac{2x}{L} - 1\right)^{2}\right]^p\right), \quad \tilde{u}(x) = 0, \quad p = \frac{\ln 0.5}{\ln \left[\frac{2x_m}{L} - 1\right]}, \quad 0.75 < x_m < 1.$$  \hspace{1cm} (35)

Figure 14 (left) displays the settlement trough for $\frac{x_m}{L} = 0.75, 0.8, 0.85, 0.9, 0.94, 0.97$. Parameter $x_m$ indicates the position of the points where the vertical displacement is a half of the maximum vertical displacement $\delta > 0$. If we take the parameters $E = 2.6G$, $L = H = 30$, $\delta = 0.01, \nu = 0.3$ and we compute the value of the maximum tensile strain in the building for different values of $\frac{x_m}{L}$, we obtain the diagram showed in Figure 14 (right).

Similar results can be also obtained for asymmetric settlement troughs. Figure 15 corresponds to taking
\[
\bar{v}(x) = C_1 + C_2 \frac{x}{L} + \sqrt{C_3 \left(\frac{x}{L}\right)^2 + C_4 \frac{x}{L} + C_5}, \quad u(x) = 0. \tag{36}
\]

Figure 14. A family of curves of vertical displacements and the variation of the maximum tensile strain with the parameter \(x_m\).

The vertical displacements correspond to a parabolic function of the form \(g(x) = -x^2\) composed with a translation and a rotation \(\alpha\). Coefficients \(C_1, C_2, C_3, C_4, C_5\) depend on the rotation \(\alpha\) and the value of the maximum vertical displacement \(\delta > 0\), that is located at \(x = x_{\text{max}}(\alpha)\). The curves displayed in Figure 15 (left) correspond to the cases \(\alpha = 0, 0.25, 0.6, 1\), or, equivalently, \(x_{\text{max}} / L = 0.5, 0.59, 0.68, 0.74\). Figure 15 (right) shows the variation of \((\epsilon_t)_{\text{max}}\) with \(x_{\text{max}} / L\) for \(E = 2.6G, L = H = 30, \delta = 0.01, \nu = 0.3\).

So, there appears to be some significant effect of the shape of the vertical displacement distribution on the magnitude of the maximum tensile strain.

### 3.4 Separate consideration of hogging and sagging zones

In this subsection we analyze the hypotheses that the hogging and sagging zones can be considered separately. In a case with Gaussian displacement profile we have considered
the set of parameters $E = 2.6G$, $L = H = 30, e = 30$, $D = 10$, $V_L = 1\%$, $\nu = 0.3$. The hogging and sagging zones are described by the sets of parameters $L_h = 15$ and $L_s = 15, e = 15$, respectively. If we consider the entire building we obtain a maximum tensile strain of $0.1981\%$, whereas the hogging and sagging zones give maximum tensile strains of $0.1447\%, 0.2481\%$, respectively.

Thus, in general, the compatibility condition that exists between the zones of the building under hogging and under sagging can produce a substantial modification of the value of the maximum tensile strain compared to that obtained by assuming two separate buildings. As the analytical method allows the consideration of the entire building, it seems that more realistic results should be obtained avoiding the artificial division of the structure.

3.5 Application diagrams

In the case of the frequently used Gaussian displacement profile, it can be useful to organize the information in diagrams suitable for direct application. For instance, for each set of parameters $E, H, D, \frac{D^2}{L}, \frac{D^2}{H}, \frac{D^2}{Lz}, V_L, V$, it is easy to plot the maximum tensile strain in the building as a function of $i, e$. Figure 16 shows the diagram corresponding to the parameter set $E = 2.6, H = 1, D = 1, \frac{D^2}{L} = 0.3, \frac{D^2}{H} = 9, V_L = 1\%, \nu = 0.3$. Contours that separate different categories of damage are indicated. As the maximum tensile strain is proportional to $V_L$, it is sufficient to consider diagrams corresponding to a volume loss of $1\%$

Figure 16. Diagram for the determination of the category of damage with a Gaussian displacement distribution.

Figure 17 displays the diagram obtained with our approach for a Gaussian settlement profile, but in this case we have assumed that the horizontal displacements are zero to take into account the strong reduction of horizontal strains that result from the consideration of the soil/structure interaction. It can be observed that damage estimates decrease considerably.
4 CONCLUSIONS AND PERSPECTIVES

We have described a new analytical procedure to estimate the maximum tensile strain due to an arbitrary distribution of vertical and horizontal displacements applied to the lower boundary of a thick beam. This parameter is regularly used to estimate likely damage in buildings following the proposal of Burland & Wroth (1974). The analytical solution has been solved adopting the hypotheses of Timoshenko’s beam theory. This approach has several advantages over the conventional method: it is not necessary to specify beforehand the location of the neutral axis, the building can be analyzed without dividing it in hogging and sagging zones, the effect of horizontal displacements is considered in a more appropriate manner, and the specific shapes of the displacement distributions are automatically taken into account. In addition, the full field of maximum principal strains can be readily obtained allowing the identification of the location of likely damage appearance in each particular case. Although this paper has focused mainly on the effects of tunnelling, the method is, of course, of general applicability.

We have presented several application examples that illustrate the performance of the method. We have also used the new procedure to examine some of the hypotheses that are usually assumed in the classical procedures. We have observed that the map of strains that appear in a beam under the prescription of arbitrary vertical and horizontal displacements can sometimes be quite different from that of a mid-point or uniformly loaded beam. In some cases, the maximum tensile strains provided by the analytical method appear to be larger than those provided by the classical approaches.

The availability of an analytical solution makes it possible to used spreadsheets (or other simple computing tools) to obtain the maximum tensile strength in a quick and straightforward manner. Application diagrams for parameter sets of specific interest can also be readily produced.
The possibility of using analytical tools for the estimation of building damage opens a variety of research lines for further progress. The following can be mentioned:

- Since the reaction forces can be easily computed from the analytical solution, the presence of tensile forces in the ground/building contact can be accounted for by assuming lost of contact in those areas. This inevitably requires, however, an iterative procedure for solving the equations.

- Variation of the properties of the building can be taking into account. If the variation is piecewise, compatibility conditions between different parts of the building should be ensured; but the general analytical solution described above can still be used. If the variation of the properties were continuous, then the differential equations that govern the problem should be rewritten and solved by means of numerical methods.

In any case, it should not be forgotten that the analytical solution has been obtained assuming that the building can be represented by a Timoshenko beam. Although the use this hypothesis is very widespread and has proved very useful in practice, it can only be an approximate description of an actual building, especially for low values of $L/H$.

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**MADRID.**
T. 91 7280524 F. 91 7290499.
madrid@paymacotas.com

**BARCELONA**
P.I. La Ferrería. Av. Ferrería,
57. 08110 Montcada i Reixac (Barcelona).
T. 93 5752834 / F. 93 5648900.
barcelona@paymacotas.com