Geotechnical Aspects of Underground Construction in Soft Ground, Mair & Taylor (eds)
© 1996 Balkema, Rotterdam. ISBN 90 5410 856 8

Prediction of ground movements and assessment of risk of building damage due to bored tunnelling

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ABSTRACT: Ground movements are inevitably caused by bored tunnel construction in soft ground. In urban areas their potential effects on buildings are an important consideration in the design and construction of tunnels. This paper summarizes the methodology adopted for prediction of ground movements and assessment of the risk of building damage adopted for three major urban tunnelling projects in the UK. The approach combines studies on classification of damage to masonry structures with methods of ground movement prediction and calculations of tensile strains induced in buildings. The assessment methodology involves a three stage process in which buildings are eliminated from further stages, depending on the potential degree of damage predicted.

1. INTRODUCTION

The construction of bored tunnels in soft ground inevitably causes ground movements. In urban areas such movements may affect buildings. Prediction of ground movements and assessment of the risk of damage is therefore an essential part of the planning, design and construction of a tunnelling project in the urban environment.

This paper summarizes the methodology adopted for prediction of ground movements and assessment of the risk of building damage for the London Underground Jubilee Line Extension project, presently under construction, and for the proposed CrossRail and Channel Tunnel Rail Link projects. All three projects involve tunnelling beneath numerous buildings in the London area. The methodology draws on work on classification of damage to masonry buildings by Burland et al (1977), and combines this with empirical methods of predicting ground movements due to tunnelling together with calculation of distortions and strains potentially induced in buildings. Based on this methodology, rational procedures for assessment of risk of building damage have been developed.

2. PREDICTION OF GROUND MOVEMENTS

2.1 *Settlement*

Initially, the case of a single tunnel in "green field" conditions will be considered. The procedure adopted generally follows that outlined by Peck (1969), O'Reilly and New (1982) and extended by New and O'Reilly (1991). Figure 1 shows the general case of an advancing tunnel with its axis at depth z_0 below ground level. Construction of the tunnel results in ground movements with a settlement trough developing above and ahead of the tunnel. Analysis of a considerable number of case records has demonstrated that the resulting transverse settlement trough immediately after a tunnel has been constructed is well described by a Gaussian distribution curve (as shown in Figure 2) as

$$S_v = S_{max} \exp\left[\frac{-y^2}{2i^2}\right] \qquad (1)$$

where S_v is settlement
S_{max} is the maximum settlement on the tunnel centre line
y is the horizontal distance from the centre line
i is the horizontal distance from the tunnel centre line to the point of inflexion on the settlement trough.

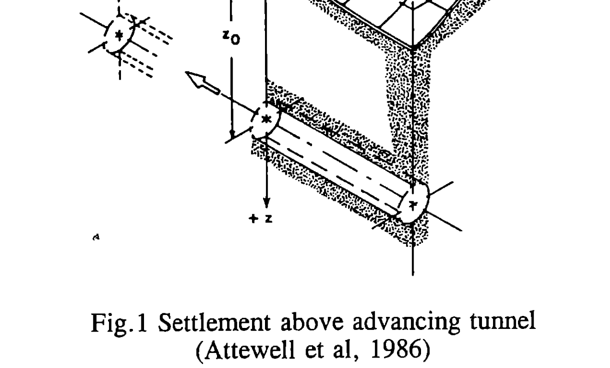

Fig.1 Settlement above advancing tunnel (Attewell et al, 1986)

For near surface settlements O'Reilly and New (1982) showed that the dimension i in Figure 2 was an approximately linear function of the depth z_0 and broadly independent of tunnel construction method. This was also confirmed by Rankin (1988) for tunnels both in the UK and worldwide. They assumed that the simple approximate relationship

$$i = Kz_o \quad (2)$$

can be adopted and that values of the trough width parameter K for tunnels in clay, and sands or gravels, may be taken as approximately 0.5 and 0.25 respectively. The choice of an appropriate value of K may require some judgement, since it depends on whether the ground is primarily cohesive or granular, and in the latter case on whether or not the tunnel is above or below the water table. Generally, for tunnels in clay strata, the full width of the transverse settlement trough is about three times the depth of the tunnel. Although the value of K for *surface* settlements is reasonably constant for tunnels at different depths in the same ground, it has been shown by Mair et al (1993) to increase with depth in clays for *subsurface* settlements.

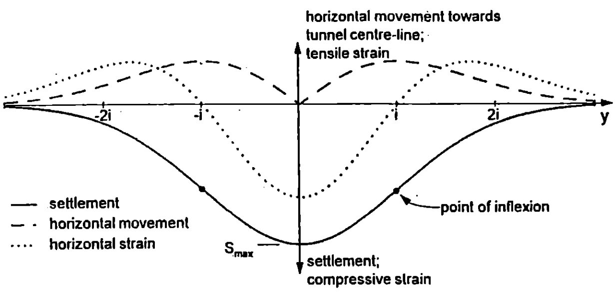

Fig. 2 Transverse settlement trough

The volume of the settlement trough (per metre length of tunnel), V_s, can be evaluated by integrating equation (1) to give

$$V_s = \sqrt{2\pi}\ i\ S_{max} \quad (3)$$

The volume loss is usually expressed as a percentage fraction, V_l, of the excavated area of the tunnel, i.e. for a circular tunnel

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

For non-circular or inclined tunnels, it is reasonable to replace the term $\pi D^2/4$ in equation (4) by the area of the tunnel intersected by a vertical plane. The volume loss is a key parameter, its magnitude depending principally on the type of ground and on the tunnelling method. For example, volume losses of up to 3% (excluding consolidation settlements) were recorded in the soft marine clays of Singapore using earth-pressure balance (EPB) tunnelling machines or compressed air (Shirlaw and Doran, 1988), whereas only about 0.2% was recorded using an EPB machine in gravels below the water table in Tokyo (Kanayasu et al, 1995). For tunnels in London Clay volume losses are generally likely to be in the range 1-2% for shield tunnelling (O'Reilly and New, 1982). Tunnels constructed in London Clay using sprayed concrete linings (often referred to as New Austrian Tunnelling Method - NATM) generally give rise to similar volume losses; values in the range 1-1.5% were reported by New and Bowers (1994), and similar results have been obtained during construction of the Jubilee Line Extension.

Equations (2) to (4) can be combined to give

$$S_{max} = \frac{0.31 V_l D^2}{K z_o} \quad (5)$$

2.2 *Horizontal Movements*

Building damage can also result from horizontal tensile strains, and therefore a prediction of horizontal movement is required. There are few case histories where horizontal movements have been measured. The data that exist show that the assumption of O'Reilly and New (1982) that the resultant vectors of ground movement are directed towards the tunnel axis is generally conservative but reasonable. The vector of ground movement has vertical and horizontal components S_v and S_h respectively. Assuming that the vector is directed towards the tunnel axis, then

$$S_h = \frac{y}{z_o} S_v \quad (6)$$

and this allows a simple assessment of horizontal movement.

Figure 2 shows the relation between the settlement trough, horizontal movements and horizontal strains occurring at ground level for a single tunnel. The horizontal ground strain, ϵ_{hg}, is determined by differentiating equation (6) with respect to y. In the region i > y > -i, horizontal strains are compressive. At the points of inflexion (y = ±i), the horizontal movements are greatest and the horizontal strain is zero. For i < y < -i, the horizontal strains are tensile.

2.3 *Longitudinal Settlement Trough*

There may be cases where a building close to or directly above the tunnel centre-line might experience more damage from the progressive longitudinal settlement trough generated ahead of the tunnel face, as shown in Figure 1, than from the final transverse settlement profile after the tunnel face has passed beneath the building. The ground movements and associated building strains should then be determined from the longitudinal settlement trough, which can be assumed to have the form of a cumulative probability curve as described by Attewell and Woodman (1982) and summarised by New and O'Reilly (1991) and New and Bowers (1994). A result of this assumption is that the settlement directly above the tunnel face corresponds to $0.5S_{max}$.

2.4 *Multiple Tunnels*

When two or more tunnels are constructed it is generally assumed that the predicted ground movements for each tunnel acting independently can be superimposed. For tunnels in close proximity (where the clear separation is less than one tunnel diameter) this assumption may be unconservative. Interaction can be taken into account by assuming a greater volume loss for the second tunnel and superimposing the resulting ground movements. In some cases a building might be more adversely affected by a single tunnel than by a later combination of multiple tunnels.

3. CLASSIFICATION OF DAMAGE

Unless objective guidelines based on experience are employed, extreme attitudes and unrealistic expectations towards building performance can develop. It is important to note that most buildings experience a certain amount of cracking, often unrelated to foundation movement, which can be easily repaired during routine maintenance and decoration. In the UK the development of an objective system of classifying damage has proved to be most beneficial in creating a logical and realistic framework for assessment of risk of damage to buildings.

The system of classification for masonry structures is summarized in Table 1. This was first put forward by Burland et al (1977), who drew on the work of Jennings and Kerrich (1962), the UK National Coal Board (1975) and MacLeod and Littlejohn (1974). It has since been adopted with only slight modifications by BRE (1981 and 1990), the Institution of Structural Engineers (1978, 1989 and 1994) and the Institution of Civil Engineers (Freeman et al, 1994).

The classification system in Table 1 is based on 'ease of repair' of the visible damage. When classifying visible damage, therefore, it is necessary during the survey to

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

Category of damage	Normal degree of severity	Description of typical damage (Ease of repair is underlined) 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.
0	Negligible	Hairline cracks less than about 0.1mm.
1	Very Slight	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 1mm.
2	Slight	Cracks easily filled. Re-decoration probably required. Recurrent cracks can be masked by suitable linings. Cracks may be visible externally and some repointing may be required to ensure weathertightness. Doors and windows may stick slightly. Typical crack widths up to 5mm.
3	Moderate	The cracks require some opening up and can be patched by a mason. Repointing of external brickwork and possibly a small amount of brickwork to be replaced. Doors and windows sticking. Service pipes may fracture. Weathertightness often impaired. Typical crack widths are 5 to 15mm or several greater than 3mm.
4	Severe	Extensive repair work involving breaking-out and replacing sections of walls, especially over doors and windows. Windows and door frames distorted, floor sloping noticeably[1]. Walls leaning[1] 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.
5	Very Severe	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.

[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.

assess the type of work required to repair the damage. It is particularly important to note that the strong temptation to classify the damage solely on crack width must be resisted; the ease of repair is the key factor.

Table 1 defines six categories of damage, numbered 0 to 5 in increasing severity. It also lists the 'normal degree of severity' associated with each category. The division between damage categories 2 and 3 ('slight' and 'moderate') is particularly important. Case records show that damage up to category 2 can result from a variety of causes, often in combination, either from within the building itself (eg. shrinkage or thermal effects) or associated with ground movement. The division between damage categories 2 and 3 represents an important threshold and is therefore of particular significance in the process of risk assessment described later.

Table 2: Relationship between category of damage and limiting tensile strain, ϵ_{lim} (after Boscardin and Cording, 1989)

Category of damage	Normal degree of severity	Limiting tensile strain (ϵ_{lim}) (%)
0	Negligible	0 - 0.05
1	Very Slight	0.05 - 0.075
2	Slight	0.075 - 0.15
3	Moderate*	0.15 - 0.3
4 to 5	Severe to Very Severe	> 0.3

*Note: Boscardin and Cording (1989) describe the damage corresponding to ϵ_{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.

Burland and Wroth (1974) developed the concept of *'critical* tensile strain' as a fundamental parameter determining the onset of cracking; this was replaced by the concept of *'limiting* tensile strain', ϵ_{lim}, by Burland et al (1977). Boscardin and Cording (1989) analysed case histories of excavation induced subsidence, and showed that the damage categories in Table 1 are related to the magnitude of tensile strain induced in the building, and ranges of strain were identified, as given in Table 2. Thus there is an important link between the estimated tensile strain induced in a building and the potential damage category.

4. CALCULATION OF BUILDING STRAINS

4.1 *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 on Figure 3. The height H is taken as the height from foundation level to the eaves. The roof of the building 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 a building. Also, 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_v/S_{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

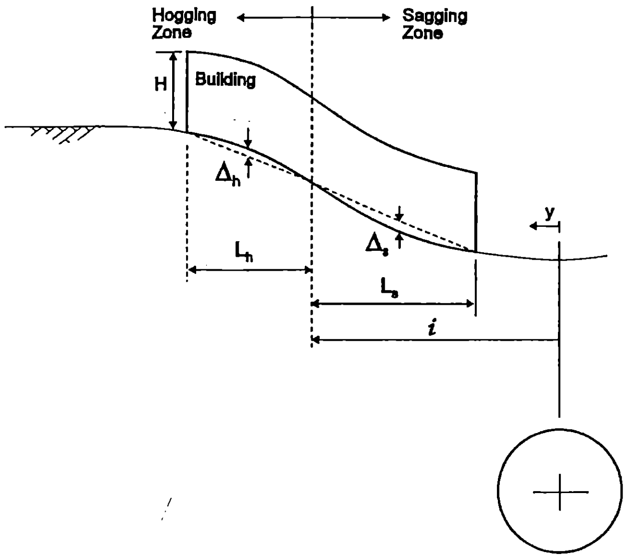

Fig. 3 Building deformation

hogging or sagging zone (shown as L_h or L_s on Figure 3) and limited by a point of inflexion or extent of settlement trough.

4.2 *Strains due to Ground Settlement*

Ground movements will usually generate tensile strains in buildings which can lead to cracking and damage. The subject of settlement damage to buildings was considered by Burland and Wroth (1974). They treated a building as an idealised beam with span L and height H deforming under a central point load to give a maximum deflection Δ. They argued that, for a building in the hogging mode, the restraining effect of the foundations would, in effect, lower the neutral axis which could therefore be taken to coincide with the lower extreme fibre of the 'beam'. For a building in the sagging mode, however, it is reasonable to assume that the neutral axis remains in the middle of the 'beam'. Burland and Wroth showed that these selections of the positions of the neutral axis were consistent with observations of building performance. Expressions were derived relating the ratio Δ/L for the beam to the maximum bending strain (ϵ_b) and diagonal strain (ϵ_d). The strains in a building with a maximum relative settlement Δ can also be determined from these expressions which were presented in a generalised form by Burland et al (1977) as

$$\frac{\Delta}{L} = \left\{ \frac{L}{12t} + \frac{3I\ E}{2tLH\ G} \right\} \epsilon_b \tag{7}$$

and

$$\frac{\Delta}{L} = \left\{ 1 + \frac{HL^2\ G}{18\ I\ E} \right\} \epsilon_d \tag{8}$$

where:

H is the height of the building

L is the length of the building (but limited by any point of inflexion or extent of settlement trough)

E and G are respectively the Young's modulus and shear modulus of the building (assumed to be acting as a beam)

I is the second moment of area of the equivalent beam (i.e. $H^3/12$ in the sagging zone and $H^3/3$ in the hogging zone)

t is the furthest distance from the neutral axis to the edge of the beam (i.e. H/2 in the sagging zone and H in the hogging zone).

The maximum bending strain ϵ_b and diagonal strain ϵ_d are likely to develop at the centre and quarter span points respectively. Although masonry is not an isotropic material, the ratio E/G is often taken as 2.6, which is consistent with an isotropic Poisson's ratio of 0.3.

It is conservatively assumed that the building follows the ground settlement trough at the foundation level. The point of inflexion of the settlement trough (defined by i for the case of a single tunnel) divides the building into two zones. In the hogging zone (-i > y > i), where the neutral axis is assumed to be at the bottom, all strains due to bending will be tensile. In the sagging zone, where the neutral axis is assumed to be at the centre of the building, bending will cause both compressive and tensile strains. Within each zone the maximum value of Δ can be determined and the deflection ratio Δ/L calculated, i.e. Δ_h/L_h in the hogging zone and Δ_s/L_s in the sagging zone, as shown on Figure 3. For a given ratio Δ/L, the hogging mode is likely to be more damaging than the sagging mode. This procedure essentially allows the building to be treated separately either side of a point of inflexion, which is considered a reasonable approach. The maximum values of Δ_h or Δ_s are unlikely to occur exactly at the centre of their respective span and in general it will be simplest to search for these numerically.

It should be noted that this direct method of evaluating Δ/L differs slightly from that suggested by Boscardin and Cording (1989) in which Δ/L is calculated from the maximum value of angular distortion (β) and which leads to a relative overprediction of tensile strains (typically by up to 20%).

4.3 *Superposition of Horizontal Ground Strain*

The horizontal ground strains due to bored tunnel construction may also contribute to potential building damage. The average horizontal strain across a section of building is more appropriate in the context of potential damage than local horizontal ground strains. In order to determine the average horizontal strain, ϵ_h, transferred to a building, equations (1), (5) and (6) are used to calculate the horizontal movement at either end of a building span under consideration; the difference between these divided by the span length then gives an average horizontal strain.

The average horizontal strain is combined with either the bending or diagonal strain obtained from equations (7) and (8), and the maximum combined tensile strain is used in the assessment of potential building damage (described later). The maximum combined tensile strain will usually occur in the hogging zone, where the horizontal strains are tensile.

The horizontal strain can be added directly to the bending strain giving

$$\epsilon_{bt} = \epsilon_h + \epsilon_b \tag{9}$$

where ϵ_{bt} is the total bending strain.

Diagonal (shear) strains and horizontal strains can be combined by making use of a Mohr's circle of strain. Assuming a value for Poisson's ratio of 0.3, the total tensile strain due to diagonal distortion, ϵ_{dt}, is given by

$$\epsilon_{dt} = 0.35\epsilon_h + \left[(0.65\epsilon_h)^2 + \epsilon_d^2 \right]^{0.5} \quad (10)$$

4.4 *Use of Simplified Charts*

As an alternative to calculating the tensile strains as indicated above, simplified charts can be used, such as the well-known one presented by Boscardin and Cording (1989) for cases of L/H=1 in terms of angular distortion β and horizontal strain ϵ_h. This chart ignores bending strains and it assumes that β is proportional to Δ/L; also, the evaluation of β is not always straightforward because the tilt of the building needs to be identified. An equivalent chart, directly in terms of Δ/L and ϵ_h, presented by Burland (1995), is shown in Figure 4 for the case of L/H=1 for the hogging mode. Other charts would be needed for different ratios of L/H.

4.5 *Framed Buildings*

Reinforced concrete framed structures are more flexible in shear than masonry structures, and are consequently less susceptible to damage. For the purposes of assessment of potential damage, framed buildings on shallow foundations can be considered using the same methodology as for masonry structures. It is more appropriate to adopt an E/G ratio of 12.5, rather than 2.6 used for masonry buildings (Burland and Wroth, 1974); this value should be used in equations (7) and (8), and the maximum strains calculated from equations (9) and (10).

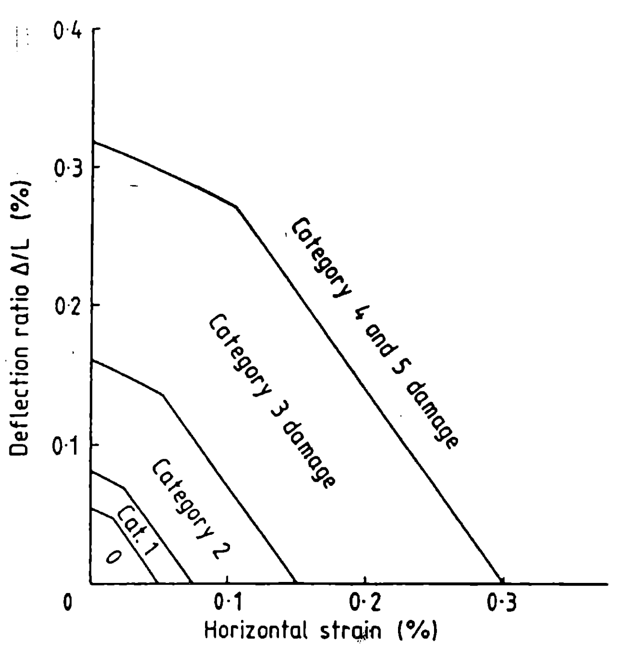

Fig. 4 Damage category chart for L/H = 1, hogging mode (Burland, 1995)

5. ASSESSMENT OF RISK OF DAMAGE TO BUILDINGS

5.1 *General Approach*

Most buildings are considered to be at low risk if the predicted degree of damage falls into the first three categories 0 to 2 (ie. 'negligible' to 'slight'). In Section 3 it was noted that the threshold between categories 2 and 3 is particularly important. A principal objective in design and construction of bored tunnels is to restrict the potential level of damage to below this threshold for all buildings. Special consideration has to be given to buildings judged to be of particular sensitivity. A staged process of assessing risk is adopted: preliminary assessment; second stage assessment; detailed evaluation.

5.2 *Preliminary Assessment*

In most situations it can be shown by the application of the equations given above that a building has a negligible risk of damage when subjected to settlements of less than 10mm. The preliminary assessment involves drawing contours of ground surface settlement and eliminating all buildings falling within this criterion. (An additional check is made that no eliminated building experiences a slope exceeding 1/500, but this is rarely critical.)

5.3 *Second Stage Assessment*

In this approach, the maximum tensile strains induced in the building are calculated using the methodology described in Section 4. The maximum tensile strain calculated from either equation (9) or (10), whichever is greater, is used to obtain the corresponding potential damage category from Table 2. The approach is usually very conservative, because the building is assumed to have no stiffness and to conform to the 'green field site' settlement trough. In reality the inherent stiffness of the building will tend to reduce both the deflection ratio and the horizontal strains. In most cases, therefore, the derived category of damage in the second stage assessment refers only to the *possible* level of damage; in practice, the *actual* damage will be less than the assessed category.

5.4 *Detailed Evaluation*

Detailed evaluation is undertaken for those buildings classified in the second stage assessment as being at risk of category 3 damage ('moderate') or greater. The sequence and method of tunnelling should be given detailed consideration, and full account taken of the three-dimensional aspects of the tunnel layout with respect to the building. (For the second stage assessment the simplified assumption is often made that the problem is two-dimensional, as shown in Figure 3.)

Details of the building should be taken into account. Buildings possessing structural continuity such as those of steel or concrete frame construction are less susceptible to damage than those without structural continuity such as load bearing masonry and brick buildings.

Soil-structure interaction effects are particularly important. The predicted displacements (and associated building strains) using the second stage assessment

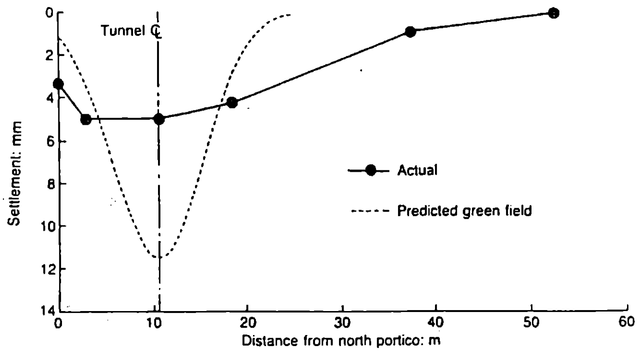

Fig. 5. Settlement of the Mansion House (after Frischmann et al, 1994)

approach, based on the 'green field site' assumption, will be modified by the stiffness of the building and its foundations. The beneficial effects of building stiffness can be very significant, as demonstrated for example by measurements on the Mansion House in the City of London during tunnelling beneath the building (Frischman et al, 1994); Figure 5 shows the actual building settlement profile to be much wider than the predicted 'green field' profile, with correspondingly much lower deflection ratios and distortions.

The foundations of a building will, in many cases, modify the horizontal ground movements so that the horizontal strain induced in the building is considerably reduced (Geddes, 1990). Buildings on continuous foundations such as strip footings or rafts are likely to experience negligible horizontal strain arising from bored tunnel construction.

In view of the inherently conservative assumptions adopted for the second stage assessment, the detailed evaluation will usually result in a reduction of the possible degree of damage. Following the detailed evaluation, consideration is given to whether protective measures are required. These are usually only required for buildings remaining in damage category 3 ('moderate') or higher.

SUMMARY

This paper describes a rational methodology for the assessment of risk of building damage due to bored tunnelling. The approach combines the use of a damage classification system for masonry structures with the concept of limiting tensile strain. The three stage assessment process, in which buildings are eliminated from further stages depending on the potential degree of damage predicted, provides a logical framework for determining the need for protective measures.

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