Settlement assessment of running tunnels – a generic approach

D.I. Harris & J.N. Franzius
Geotechnical Consulting Group, London, UK

ABSTRACT: This paper introduces a new generic approach to the second stage of the assessment of the potential for building damage associated with major tunnelling projects in urban areas. The generic methodology has been developed to improve the effectiveness and efficiency of the process whilst maintaining consistency with the 3 phase assessment methodology outlined by Mair et al. (1996). The aim is to reduce the effort expended in producing a large number of Phase 2 calculation reports for individual buildings which show, in most cases, that the potential damage is within acceptable limits. Implementation of the new method will enable resources to be targeted at more problematic areas in Phase 3. As an example the paper presents a study of a typical 3 km long section of twin running tunnel; it is shown that the generic assessment alone is sufficient to conclude that no unacceptable damage will occur. For the section of running tunnels presented, one figure replaces several hundred individual building assessment reports. The generic methodology therefore reduces significantly the cost of potential building damage assessment.

1 INTRODUCTION

In the late 1980's and early 1990's the phased approach to settlement or potential damage assessment due to tunnelling projects was developed into a formalised procedure. The method comprises 3 phases which use successively less conservative and more complex analysis. Buildings are eliminated from further consideration at each phase if it is demonstrated that the risk of unacceptable damage is small. Mair et al. (1996) describe the methodology applied on the Jubilee Line Extension (JLE), London: similar processes have been used on projects worldwide.

Phase 1 of the procedure is an example of a generic approach in which volume loss contours are used to identify buildings potentially at risk of damage from settlement. For major urban schemes, the number of buildings which continue into Phase 2 can number in the thousands. Phase 2 requires the production of an individual assessment report for each building: production of this number of buildings is very time consuming and expensive. The settlement of the vast majority of the buildings is due to running tunnels alone and hence Phase 2 assessments would be expected to be indicate unacceptable potential damage in a very limited number of locations. The potential to develop a more efficient method of identifying any such areas was identified by Geotechnical Consulting Group (GCG) as a result of a review of settlement issues arising from Jubilee Line Extension (JLE) Contract 102. A generic approach to Phase 2 assessments was recommended, and GCG have investigated and developed this alternative, generic approach, as described herein.

2 CURRENT SETTLEMENT ASSESSMENT

The settlement assessment procedure consists of three phases, schematically shown in Figure 1, which are referred to as "preliminary assessment", "second phase assessment" and "detailed evaluation".

In Phase 1 the presence of buildings is not considered and "greenfield" volume loss settlement contours are calculated using formulae described by Attewell et al. (1986). For buildings outside the 10 mm settlement contour and with a maximum slope of less than 1/500 no further action is required. For all other buildings a Phase 2 assessment has to be carried out.

In Phase 2 "greenfield" (denoted with the index "GF") settlements and horizontal displacements are calculated for each building individually. The building is assumed to follow the greenfield ground movements and is represented as an elastic beam, described by its length *L*, its height *H* and a ratio of Young's modulus over the shear modulus *E*/*G*(Burland & Wroth, 1974).

The deflection ratio, DR^{GF} (defined in Figure 2), and the average horizontal strain, ε_h^{GF}, along the building are evaluated. In this approach, zones of hogging/tension and sagging/compression are treated independently. As shown in Figure 2, these zones are defined by the points of inflection (that is the position of maximum slope), by the edges of the building or by the extent of the settlement trough. The deflection

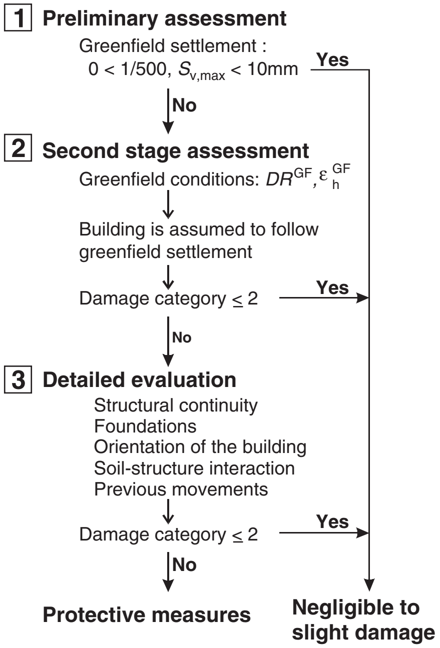

Figure 1. Phase 3 building risk assessment.

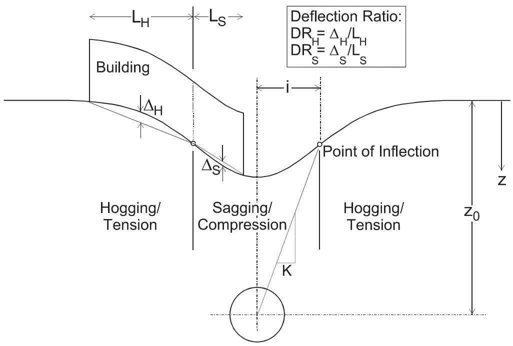

Figure 2. Definition of geometry.

ratio and the average horizontal strain are calculated for each hogging and/or sagging zone of the building; these can be related to categories of potential damage (Table 1) using interaction diagrams (Burland, 1995). Alternatively, the category of damage can be obtained by calculating the tensile strain developing in the building as outlined by Mair et al. (1996). For damage categories of 2 (Slight) or smaller only aesthetic damage is predicted and thus no further analysis is required. For buildings with a potential damage category of 3 (Moderate) or higher a detailed evaluation is required.

Although the Phase 2 calculation is more detailed than the preliminary assessment, it is conservative since the building is assumed to follow the greenfield settlement trough. Individual analyses are carried out for each building within the 10 mm contour of the settlement trough; this produces a large number of reports which is a time consuming, and therefore a costly, process.

Table 1. Damage categories (after Burland, 1995).

Category of damage	Limiting tensile strain [%]	Normal degree of severity
1	0–0.05	Negligible
2	0.05–0.075	Very Slight
3	0.075–0.15	Slight
4	0.15–0.3	Moderate
5	>0.3	Severe

In Phase 3 more details of the building and of the tunnel construction are taken into account. This includes the orientation of the building towards the tunnel, building features such as the foundation design and structural continuity and their effect on the soil-structure interaction. Mair et al. (1996) point out that, because of the conservative assumptions of the second phase assessment, the detailed evaluation will usually predict lower categories of damage than obtained from the previous phase. However, if the risk remains high (i.e. damage category of 3 or greater) it has to be considered whether protective measures are necessary.

3 THE GENERIC APPROACH

3.1 *Introduction*

Based on experience on JLE Contract 102, it was concluded that the 3 phase approach to settlement assessments was inefficient: for many of the Phase 2 assessments undertaken, a low damage category is evident a priori, and consequently the expense in producing individual reports for all buildings is not justified.

The proposal for generic Phase 2 assessments is to apply the same assumptions and calculation methods as used previously for the assessment of individual buildings, but to apply these to representative sections taken through the surface settlement contours determined in the Phase 1 assessment. Along each section, a high number of different building geometries are analysed and the worst case, i.e. the maximum tensile strain, for any building geometry located along the section is then determined. The main advantages of this approach are:

– it avoids the production of hundreds or thousands of Phase 2 reports for individual buildings;

- it gives improved insight into the variation of maximum tensile strain along the route alignment and identifies potentially problematic areas to be more efficiently targeted for further assessment;
- it increases the accessibility of results since the result for any particular property can be abstracted for a very limited number of figures (this paper will demonstrate that 1 figure can summarize results which apply to hundreds of buildings).

The main limitation with the proposed generic method is that it cannot take account of varying foundation depth. It is proposed that surface settlement contours are used. This is logical in that the values of trough width parameter, K (defined in Figure 2), are determined from empirical relationships derived from surface measurements, largely from greenfield sites. The application of such a value at foundation depth is illogical since it implies that the trough width is narrower for locations with buildings than for greenfield sites. It can be shown that for a foundation depth z to tunnel depth z_0 (see Figure 2) ratio of $z/z_0 = 0.2$ the (subsurface) value of i is 80% of i at the surface; with the assumption of a variable K with subsurface level (Mair et al., 1993) this value increases to 87%. Compared with the other simplifying assumptions incorporated in Phase 2 calculations, this effect is minor.

The proposed method is applicable to relatively shallow foundations (single or no basement), i.e. $z/z_0 < 0.2$ (with z being the foundation depth); the results are not applicable to buildings with deep basements or piled foundations which need to be considered separately. These buildings should be identified and a building specific assessment should be undertaken.

3.2 *Settlement calculation*

The settlement assessment presented in this paper was undertaken using a visual basic spreadsheet which computes the greenfield settlement for any tunnel configuration. The output can be obtained in the form of a grid to produce contours or in the form of a section line in any horizontal direction. From the latter it is possible to calculate the deflection ratio (both hogging and sagging) and the horizontal strain (tension and compression) for any given building geometry along the section line.

The tunnelling induced settlement contours are calculated in the same way as a standard Phase 1 assessment, using empirical formulae describing the settlement trough as a Gaussian curve (Attewell et al., 1986). The input required for this settlement analysis are the coordinates of the tunnels, their diameter and depth, the settlement trough width, K, and the volume loss, V_L.

The effect of the spacing of points along each section line, at which the movements were determined, was evaluated to optimise the precision whilst maintaining the calculation effort at a reasonable level. It was found that, for running tunnels, reducing the spacing below 2 m had only a negligible effect on the results obtained.

3.3 *Choice of sections*

The determination of the building distortion criteria, together with the position of the point of inflection, is dependent on the direction of the section along which they are calculated. It is therefore necessary to define the sections such that the maximum values of deflection ratio DR and horizontal strain ε_h are determined. This is generally the direction of the maximum gradient of the settlement trough.

For single tunnels these conditions are given by the transverse direction, i.e. perpendicular to the tunnel axis. For twin running tunnel systems, an average bearing of the two tunnels can be calculated and the section which is orthogonal to this bearing analysed. This is an approximation but, for the running twin tunnels analysed in this paper, the maximum difference in bearing was 5°. The change in deformation criteria due to the rotation of the section by 2.5° with respect to each single tunnel was less than 1% (when compared with a single tunnel analysis).

At stations, interchanges and ventilation/intervention shafts, the geometry is more complicated and the choice of sections is not as straight forward as for running tunnels. In these situations, settlement contours can be calculated and sections lines positioned orthogonal to the settlement contour lines. Of particular interest are sections which cross areas of high differential settlement. Intermediate construction stages need to be considered to ensure that the most critical cases are identified. The generic methodology has been successfully applied to shaft and station locations, however, this paper presents results for running tunnels only.

3.4 *Variation in building geometry*

In the generic methodology, no specific buildings are considered but a wide range of building length/position combinations are used; this effectively addresses all possible combinations of building length and position. The results can be represented in a matrix with the columns giving the building length L and the rows representing the position of the building along the section. A total of 8 different building lengths were included in the analysis (4, 8, 16, 24, 32, 40 and 96 m). The range of positions was chosen in order to cover the full length of each sagging or hogging zone, defined by the points of inflection. Also buildings which are only partly within these zones were considered. The position was varied in steps of 2 m within each hogging or sagging zone. The influence of the increments in building length and position on the generic approach were investigated and it was found that a reduction in these steps only marginally affects the results. The number of building geometries considered for each sagging or hogging zone varied between 280 and 384 cases.

3.5 *Calculation of deformation criteria*

The calculation of the deformation criteria is based on the procedure outlined by Mair et al. (1996) in which hogging/tension and sagging/compression zones are treated independently (see Figure 2). For the settlement due to running tunnels, building deformation within a zone defined by an offset of 2.5i from the tunnel centre line are often considered. However, for consistency with station and shaft locations where this definition is not meaningful, it is recommended that the 1 mm settlement contour is used.

The values of tensile building strain depend on the following parameters:

- Ratio of Young's modulus over shear modulus E/G: A value of $E/G = 2.6$ was adopted which is equivalent to an isotropic Poisson's ratio of $\nu = 0.3$. (This assumption is made in the standard Phase 2 methodology, Mair et al., 1996)
- Position of neutral axis: For hogging the neutral axis was modelled to be at the lower fibre of the building while for sagging the neutral axis was assumed to be in the middle of the structure (This assumption is made in the standard Phase 2 methodology, Mair et al., 1996).
- Ratio of building length over height L/H: Table 2 lists the values used to calculate the maximum tensile strain, in the analyses presented herein.

Although the fixed building geometries given in Table 2 were used in the example results presented herein, it is recommended that fixed, critical values of L/H are used to calculate the maximum tensile strain from the deflection ratio and horizontal strain. This is preferable because, as noted above, the actual length, L, used in the calculation is determined by the extent of the hogging or sagging zones. The same assumption is made in the standard Phase 2 methodology (Mair et al., 1996). If a fixed building height, H, is used, this results in variable, smaller values of L/H than those initially. The values of deflection ratio and horizontal strain can be used directly to assess the effect of changes in L/H for other ratios by using interaction diagrams.

Table 2. Building geometries analysed.

L [m]	4	8	16	24	32	40	48	90
H [m]	4	8	16	17.5	19	20	20	20
L/H	1	1	1	1.4	1.7	2	2.4	2.8

4 GENERIC ASSESSMENT OF RUNNING TUNNELS

The generic approach was applied to a 3 km length of twin tunnels with an excavated diameter of 7.8 m. The tunnel depth of the twin tunnels (right hand side vertical axis) and the horizontal separation between the two (left hand side axis) is shown in Figure 3. It can be seen that over the first 1700 m both tunnels have approximately the same tunnel depth z_0 increasing from 40 m to 24 m. From thereon the two tunnel diverge vertically with the westbound tunnel (WB) increasing its tunnel depth to 37 m while the Eastbound (EB) tunnel remains between 24 m and 29 m. The horizontal distance of the two tunnels is between 19 and 56 m.

Table 3 summarizes the parameters adopted for the settlement calculation. A volume loss (i.e. the ratio between volume of the surface settlement trough per running metre over the area of the tunnel face) of $V_L = 1.5\%$ with a settlement trough width parameter of $K = 0.5$ was used. These values are within the range of tunneling in stiff clays, such as London Clay for example.

The generic method was applied by positioning sections every 100 m in the transverse direction to the twin tunnel system as shown in Figure 4. The graph also shows the contour of surface settlement. The dotted lines connect the positions of points of inflection detected along each section line. Before section 1100 the settlement profile shows only two points of inflection and therefore exhibits a sagging zone above the tunnels, surrounded by two hogging zones at the edges of the settlement trough. In contrast, four points of inflection are marked for sections 1100 to 1300. In this area a separate sagging zone develops above each of the two tunnels with a central hogging zone between

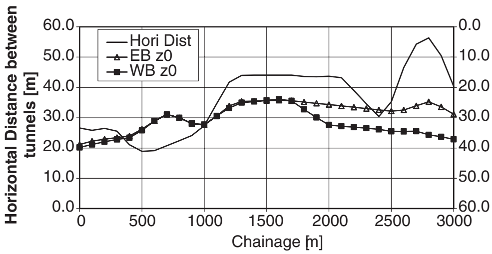

Figure 3. Geometry of tunnel route.

Table 3. Input parameters for settlement calculation.

D [m]	V_L [%]	K	max z_0 [m]	min z_0 [m]
7.8	1.5	0.5	39.8	23.9

them. Two further hogging zones are located towards the edges of the settlement trough.

It can be seen that settlement exceeds 10 mm in this area and consequently a Phase 2 assessment would be necessary for all individual buildings above the tunnels. Instead the generic assessment calculates the maximum value of potential tensile building strain along each section and these values are compared with limits of strain associated with damage categories listed in Table 1. Figure 5 summarizes the results by plotting the maximum value of tensile building strain detected for different building lengths along each section against their chainage. The horizontal lines define the upper boundaries of damage categories 0 ("Negligible"), 1 ("Very Slight") and 2 ("Slight"). The graph shows that the highest potential value of tensile building strain falls within the lower range of category 2 and develops between chainage 1300 and 1700. The following points can be noticed in Figure 5.

Firstly, all values of tensile building strain are well below category 3, and most values are within category 0. Consequently no further building assessment is required. This demonstrates that the trigger level of 10 mm settlement for a Phase 1 assessment can be conservative. Figure 5 replaces a few hundred individual building Phase 2 assessments.

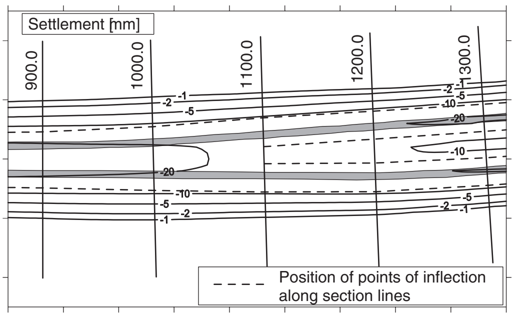

Figure 4. Settlement contour along running tunnels together with positions of sections and points of inflection.

Secondly, it can be seen that the rapid increase in tensile building strain around chainage 1300 in Figure 5 coincides well with the chainage were the central hogging zone develops, i.e. the position at which Figure 4 showed four points of inflection per section. This behaviour is further illustrated in Figure 6 which plots separate curves of maximum tensile building strain for each hogging zone (it was found that the maximum tensile building strain in the sagging zones was always lower than that in the hogging zones). Only maximum values are given; i.e. the upper envelope of this graph coincides with the maximum values shown on Figure 5. The graph shows no central hogging zone between chainages 0 and 1100; each section has only two hogging zones (referred to as "westbound" and "eastbound"). Above chainage 1100 each section consists of the two outer hogging zones and a central hogging zone. The graph not only demonstrates that in all sections the maximum strain develops in hogging zones, it further shows that it is the central hogging zone which yields the highest values of tensile building strain.

An interaction diagram plots the deflection ratio against horizontal strain and allows for the effect of the length/height ratio (L/H) of the building by variation in the damage category envelopes. Figure 7 shows the results from Chainage 1600 (where Figure 5 showed the highest values of tensile building strain along the route) and includes results for both hogging and sagging zones. The data points are the maximum values obtained for each building length at this particular chainage. The diagram also shows the boundaries of the damage categories for L/H ratios of 1 and 4. For the data points shown the L/H ratio was close to 1 (note that lower L/H ratios than those listed in Table 2 are obtained, as the length of the hogging or sagging zone is restricted by the position of the points of inflection). Comparing the data points with the corresponding boundaries shows that the building strain in the central hogging zone remains close to the lower boundary of category 2. It also shows that both sagging zones

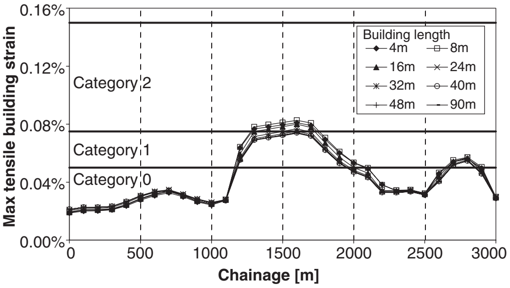

Figure 5. Maximum tensile building strain along tunnel route.

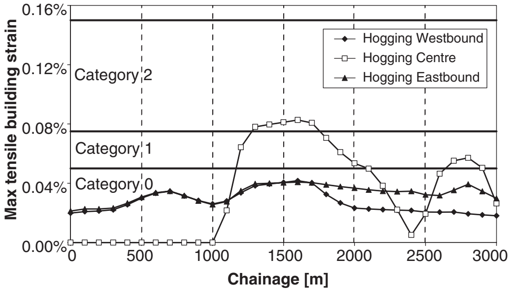

Figure 6. Maximum tensile building strain in hogging zones.

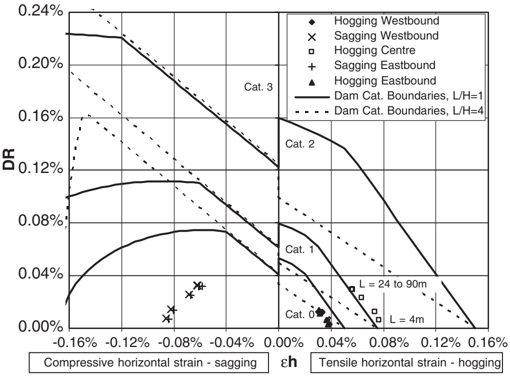

Figure 7. Interaction diagram for chainage 1600.

remain within category 0 while the two outer hogging zones are closed to the upper boundary of category 0.

The values of L/H listed in Table 2 are an assumption made for the calculation of tensile building strain and different ratios can lead to higher values of this strain. For hogging, $L/H = 4$ gives the highest value of tensile building strain. Figure 7 shows that with the conservative assumption of $L/H = 4$ the data points of the central hogging zone remain within damage category 2 and therefore no further settlement analysis is required. To ensure that the most onerous case is determined, $L/H = 4$ should be adopted in the calculation of tensile strain rather than the variable values given in Table 2.

5 CONCLUSIONS

This paper has presented a new generic approach to Phase 2 potential damage assessments and illustrated its application for a 3 km long section of running tunnels in stiff clay. In recent tunnelling projects in London, it was found that the individual building damage assessment was a very time consuming and therefore costly process. The generic approach, proposed in this paper, simplifies this procedure and therefore can lead to a substantial reduction in costs.

In the generic methodology, the settlement is calculated along sections perpendicular to the twin tunnel system. Sections were located every 100 m and along each the potential building damage of a wide variety of building geometries was analysed. The worst case for each section (or for each hogging or sagging zone) was extracted and compared to limits associated with damage categories. It was shown that in this "worst-case" scenario all building scenarios remain within or below damage category 2 for which normally no further action is required. The generic methodology significantly reduced both the costs and the time required for the settlement assessment.

This paper only presents results for running tunnels. However, the generic methodology has been extended to more complex geometries at shafts and stations. The methodology is essentially identical although different methods of presenting the results have been developed.

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