

Modelling tunnelling-induced settlement of masonry buildings

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■ **Current practice for assessment of settlement damage to buildings due to tunnel construction usually starts with a procedure in which greenfield settlements are imposed on a structural model of the building. This process ignores the important interaction effects that the weight and stiffness of the building have on the settlements. This paper describes a three-dimensional finite element analysis in which the tunnel, the soil and a building are all treated in a single analysis. Example calculations are described, and these indicate that interaction between the building and the ground can have a significant effect on the extent of the predicted damage. The performance of the building is seen to be highly dependent on whether the settlements induce a sagging or hogging mode of deformation. The analyses are used to plot contours of soil surface settlement and also expected crack patterns within the building.**

Keywords: brickwork & masonry; mathematical modelling; tunnels & tunnelling

Notation

c	residual tensile stress
D	tunnel diameter
G_0	small strain shear modulus
K_0	coefficient of earth pressure at rest
s_u	undrained shear strength
x	distance from façade centreline
z	distance below the ground surface
θ	inclination of the major principal strain direction
$\mu\epsilon$	microstrain

Introduction

Several recent tunnelling projects in the UK, most notably the construction of the Jubilee Line Extension in London, have involved the installation of relatively shallow tunnels close to important buildings. In these cases, the design process includes procedures to predict the probable extent of any settlement-induced damage to the buildings. An assessment of potential damage is particularly important when the buildings are of masonry, in which case relatively small differential settlements can lead to unsightly cracking in the walls and façades. If the extent of the predicted damage is

unacceptable, then appropriate action needs to be taken, for example modifications to the design or the specification of settlement control procedures such as compensation grouting.

2. Current assessment methods are generally based on a two-stage process. First, the ground settlements at an equivalent site where buildings are absent are estimated (these are termed 'greenfield' settlements). These displacements are then imposed on a structural model of the building to obtain an assessment of the expected damage. Burland and Wroth¹ described a general procedure of this sort for the prediction of settlement-induced damage to masonry buildings. Mair *et al.*² refer to the need to carry out more detailed assessments of structures that are found, from the initial two-stage assessment, to be at risk from settlement-induced damage. They give, however, no details as to how these assessments should be carried out.

3. Greenfield settlements are usually approximated empirically by a Gaussian curve in a direction perpendicular to the tunnel axis, and by a cumulative probability curve in the tunnelling direction. These curves are specified by two parameters. One defines the width of the settlement trough and depends on the soil type and the tunnel depth. The second is the value of the 'ground loss' that occurs during tunnel construction, and this influences the magnitude of the settlements. Ground loss is due to two separate mechanisms. Loss occurs at the tunnel heading by an amount that depends (among other factors) on the pressure applied to the tunnel face; it may also occur by approximately radial movement of the soil after excavation has taken place, and before the tunnel liner is installed and grouted.

4. The surface settlements caused by shallow tunnel construction at a greenfield site have been the subject of much research. As a result, greenfield settlement profiles can often be predicted with some confidence. However, surface settlements that develop in urban areas, where complex interaction mechanisms may act between buildings and the ground, are less well understood. Unfortunately, few published data are available that relate to field measurements of buildings subjected to tunnel-induced settlements. The data that do exist (for example, studies of the Mansion House during construction of the Docklands Light Railway³) suggest that the presence of surface buildings leads to

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profiles of surface settlement that are radically different from those that might be expected at a greenfield site. Such observations lead to questions about the merits of design procedures in which the effect of buildings on the settlement profile is ignored.

5. To assess the extent of likely damage to a masonry building, it is usually assumed that the damage due to cracking is related to the magnitude of the tensile strains developed within the structure. Burland and Wroth,¹ for example, describe a procedure in which individual façades of a building are modelled as elastic deep beams. Tunnel-induced settlements are imposed on the building façades and approximate expressions are used to estimate the induced maximum tensile strain. In this approach, the lateral strain at the ground surface is taken to be zero, and different analytical approaches are used to deal with portions of the building deforming in hogging and sagging modes. The Burland and Wroth¹ approach was intended to predict the performance of buildings subjected to general settlements; Boscardin and Cording,⁴ however, applied the method to the particular case of settlements caused by tunnelling, and included the effect of horizontal movements at the ground surface. Although assessment methods such as these that are based on the assumption of elastic structural behaviour are convenient to use, it should be noted that a masonry building is unlikely to behave elastically, particularly once significant cracking occurs.

6. It is clear that current design procedures (such as those described by Burland and Wroth¹ and Boscardin and Cording⁴) do not model detailed aspects of the mechanisms that cause settlement damage in masonry buildings. In response to this, a research project is under way at Oxford University on the development of new procedures to assess settlement-induced damage to buildings. These procedures are based on a three-dimensional finite element method, in which the building, the ground and the tunnelling processes are combined in a single numerical model. The purpose here is to describe this model and to show how it has been used to study an example problem. The example analyses illustrate that soil-structure interaction effects have an important influence on the predicted structural damage. These analyses were carried out using the finite element program OXFEM which has been developed at Oxford University for the analysis of problems in geotechnical engineering. Details of the relevant finite element formulations are given in the literature.⁵⁻¹⁰

Finite element modelling of settlement damage to buildings

7. Analysis of such a complex non-linear problem by finite element methods is a major

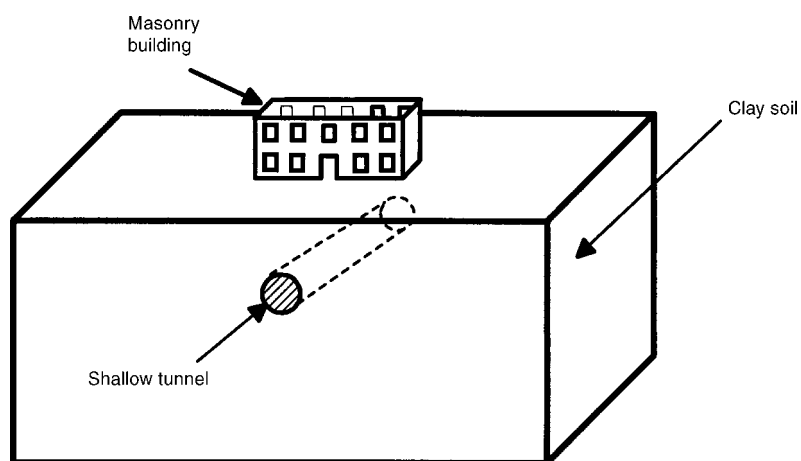


Fig. 1. Example analysis

undertaking, but finite element procedures have several advantages over the simplified predictive methods that are in current use. The interaction of the building and the soil, for example, is dealt with naturally by the use of a single mesh to represent the combined problem. The interaction between a building and a nearby tunnelling process is essentially a three-dimensional phenomenon and so a realistic study of the problem can only be carried out using a three-dimensional analysis. Finite element computations of this sort are highly complex, however, and require meshes with large numbers of elements to capture the geometry of the tunnel and any nearby structures. The analysis requires careful preparation, access to suitable mesh generation and finite element software and suitably powerful computing facilities. Although these procedures are undoubtedly complex, they may lead to results that are significantly more realistic than those obtained using conventional methods.

8. This paper describes the initial stages of development of new procedures to predict tunnel-induced damage to surface structures. This phase concentrated on a set of example problems. Further research on the application of these methods to case histories is currently under way.

Example finite element analyses

9. The example problem consists of a simple masonry structure (8 m high \times 20 m wide \times 10 m deep) on the surface of a clay soil as shown in Fig. 1. The structure consists of two identical façades, containing a set of openings to model windows and a door. The façades are connected by two plain gable walls. The roof, floors and internal walls of the building are not included in the model, and this is justified on the grounds that most of the mass and stiffness of a masonry building lies in the masonry itself. These structural elements could be added in a relatively straightforward way leading to an improved model, but this would be at the expense of additional complexity. Foundation



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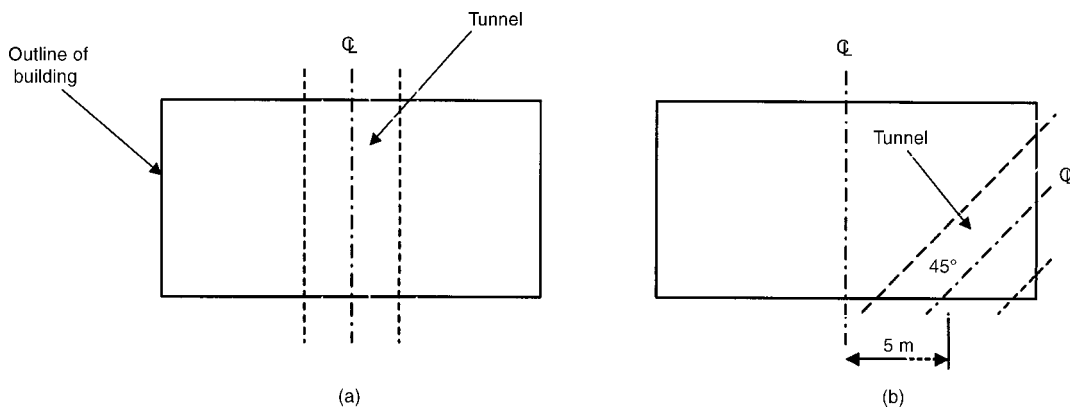


Fig. 2. Plan view of tunnel positions: (a) symmetric analysis; and (b) non-symmetric

details are not modelled, although clearly these would be an important consideration for an analysis of a particular building.

10. Two tunnel configurations have been studied. In one case the tunnel is installed directly below the centreline of the building (termed 'symmetric' analysis, Fig. 2(a)) and in the other the tunnel is installed obliquely beneath one corner of the building (termed 'non-symmetric' analysis, Fig. 2(b)). In both cases the diameter of the tunnel is 5 m, the cover depth to the crown is 7.5 m and the tunnel is horizontal. Numerical studies were carried out to determine the probable extent of structural damage in the building caused by the tunnelling processes. The project was concerned only with the prediction of short-term damage and so the soil was modelled as undrained.

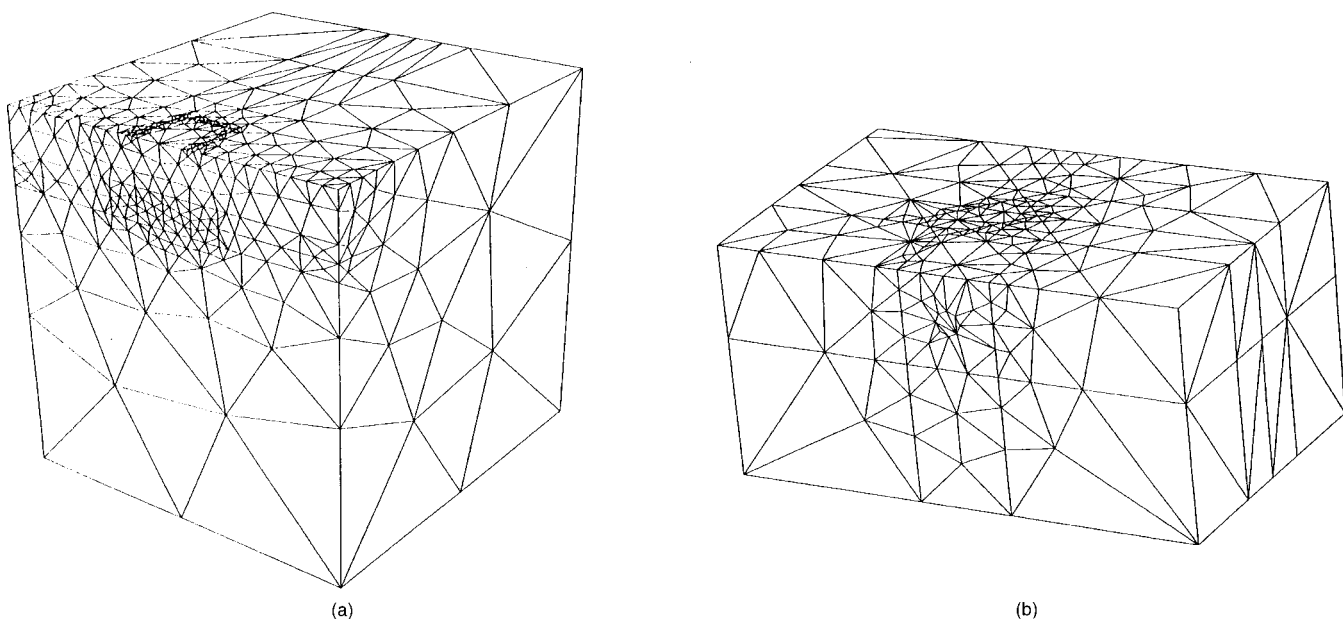
Finite element meshes

11. Finite element meshes for soil were generated using the commercial CAD package I-DEAS and for the building using in-house software. For the symmetric run, symmetry about a vertical plane through the tunnel axis was exploited by modelling half of the problem

and specifying suitable boundary conditions on the plane of symmetry. The soil was modelled as a 60 × 60 × 60 m block with full fixity applied to the base and also the boundary parallel to the plane of symmetry. Roller supports were applied to the other three vertical boundaries. For the non-symmetric analysis, however, it was necessary to mesh the complete problem. The soil, in this case, was modelled as a 60 × 120 × 60 m block with full fixity applied to the base and the two side boundaries. Roller supports were applied to the two vertical boundaries at each end of the tunnel.

12. The meshes for the ground for the symmetric and non-symmetric analyses are shown in Fig. 3. These meshes use ten-noded tetrahedral elements, which have certain computational advantages over the alternative of hexahedral (brick) elements, for cases such as this where the soil is incompressible. In particular, they are more efficient than brick elements because there are more free degrees of freedom per node.¹¹ They are also more adaptable when free meshing techniques are used; free meshing is adopted here so that mesh density can be increased near the tunnel and building. The meshes each

Fig. 3. Finite element meshes for the ground: (a) symmetric run; and (b) non-symmetric



contain a zone of elements defining the tunnel: a cylinder for the non-symmetric mesh and a semi-cylinder for the symmetric mesh. This zone is clearly visible, for example, in the symmetric mesh (Fig. 3(a)). The elements within this zone were incrementally removed during the finite element analysis to simulate the construction of the tunnel. This incremental construction procedure is necessary to study the response of the building as tunnel construction proceeds beneath it; the progressive nature of the problem cannot be captured by two-dimensional analysis. In the calculations described in this paper, the tunnel was constructed in four stages; more stages could be used but at the expense of additional computer time.

13. The building mesh used for the non-symmetric run is shown in Fig. 4. Each façade of the structure was modelled by a plane of six-noded plane stress triangular elements. It is thought that the stiffness of a building depends mainly on the in-plane stiffness of the façades, and that out-of-plane bending effects are less significant. These elements were therefore formulated in terms of in-plane displacements only and, as a consequence, each node has two (rather than three) degrees of freedom. The façades were connected using specially developed tie elements.^{8,12} The building mesh used for the symmetric run was similar to that for the non-symmetric analysis, except that it was necessary to mesh only half of the building.

14. To connect the ground and building meshes together, a set of nodes was generated on the surface of the ground mesh at precisely the same positions, in plan, as those at the base of the building. These nodes required the mesh to be refined in the region of the building foundation; zones of refined elements are seen clearly in Figs 3(a) and (b). The nodes at the base of the building were joined to the 'foot-print' nodes at the surface of the ground mesh using tie elements.^{8,12} These tie elements prevent slip between the soil and the building.

Modelling the behaviour of the ground

15. Recent research has shown that it is necessary to choose carefully the soil model to obtain realistic predictions of ground movement developed during tunnelling processes.^{13,14} In particular, the importance of the non-linearities that are known to occur in overconsolidated clay at low strain levels has been recognized. Gunn,¹³ for example, showed that the width of a settlement trough predicted by a model for a tunnel constructed at a greenfield site may be unrealistically wide unless proper account is taken of these small strain non-linearities. Similar conclusions were reached by Chow¹⁵ and Addenbrooke *et al.*¹⁴

16. The important feature of clay behaviour that must be modelled in this case is the variation of stiffness that occurs at strain levels that are

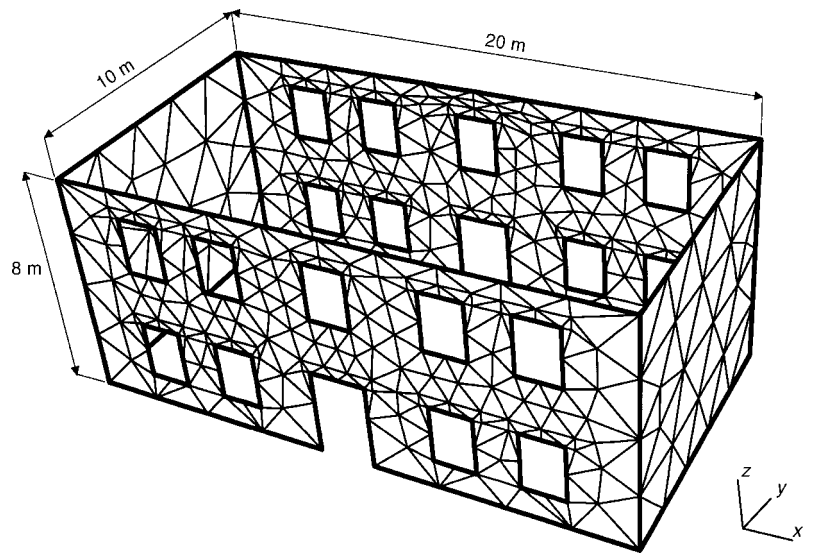


Fig. 4. Finite element mesh for the building (non-symmetric run)

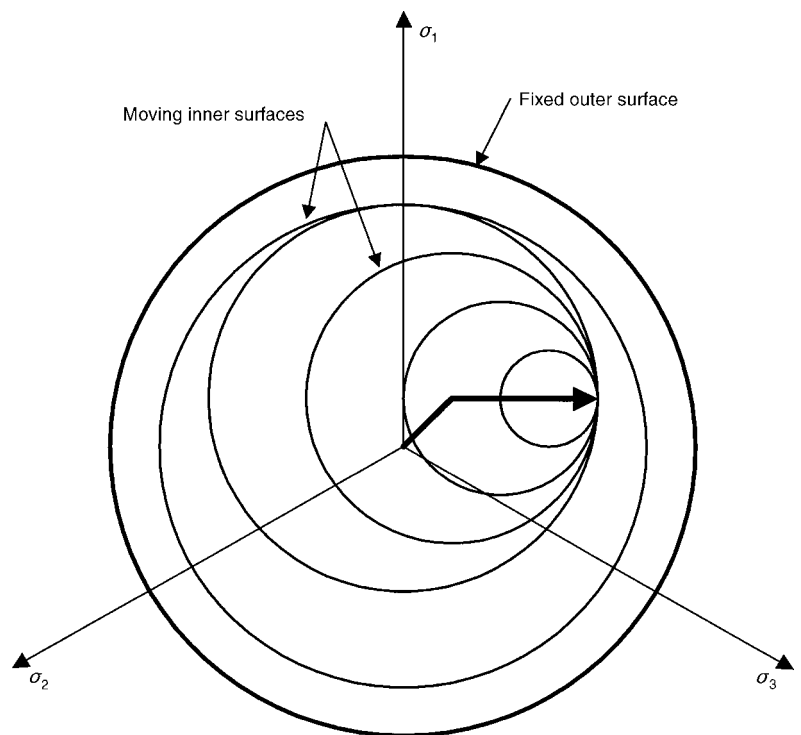


Fig. 5. Yield surfaces and stress path for the nested yield surface model

well below those that would lead to failure of the material. Several models have been proposed to deal with this effect, for example the 'bricks-on-strings' model described by Simpson,¹⁶ the three-surface kinematic yield hardening model developed by Stallebrass¹⁷ and the non-linear elastic model proposed by Jardine *et al.*¹⁸

17. The model adopted for this project¹⁹ is a multi-surface plasticity model. It is designed to model the undrained response of clays. The yield surfaces, which are cylinders parallel to the space-diagonal in total stress space, are

illustrated in Fig. 5, which shows sections of the yield surfaces on the octahedral plane. The surfaces translate in stress space according to a set of linear strain hardening relationships. As the stress point moves, those surfaces it contacts move with it, as illustrated in Fig. 5. Each yield surface is defined by two parameters, which specify the size of the surface and the magnitude of the strain associated with its movement. The size of the outermost surface, which is fixed, determines the undrained shear strength of the material. An attractive feature of the model is that as the size of a surface approaches zero the equations defining its behaviour reduce to the elastic equations, and as the stiffness associated with it approaches zero they approach the perfect plasticity equations. The surfaces effectively interpolate between an initial elastic response and perfect plasticity at large strains. The model reflects some of the features of the work by Simpson¹⁶ but is formulated in a slightly different way. The model assumes that the tunnelling process can be treated as undrained.

18. In these analyses the soil was modelled using nine inner yield surfaces and the bounding surface. The undrained shear strength, s_u , was

$$s_u = 60 + 6z \text{ (kPa)} \quad (1)$$

where z is distance (in metres) below the ground surface. The small strain shear modulus, G_0 , was set to $500s_u$. The remaining model parameters were selected to give a variation of stiffness with strain that is representative of the behaviour of London Clay; the variation of tangent stiffness against shear strain is shown in Fig. 6. Note that the stiffness reduces in steps, which are associated with the yield of individual surfaces in the model. The curve could be made smoother by choosing more surfaces, but this would add to the computation time. The use of ten surfaces is a compromise between accuracy and efficiency.

Modelling the behaviour of masonry

19. Masonry has a high compressive strength and a relatively low strength in tension. It is therefore expected that the dominant mode of settlement-induced damage in masonry structures is associated with tensile failure. A suitable model for masonry may therefore ignore the possibility of compressive failure, but must model failure of the material in tension.

20. A relatively straightforward masonry model has been adopted, in which the material has a low tensile strength and infinite compressive strength. This model is illustrated in Fig. 7. When both principal stresses are compressive then the material is elastic with Young's modulus 10 000 MPa. If the strains in the material cause the minor principal strain to

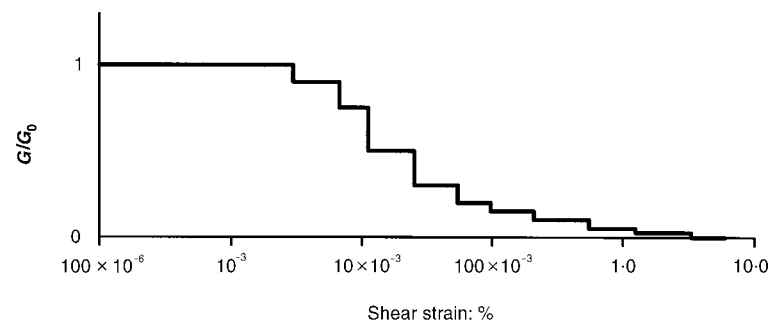


Fig. 6. Variation of tangent shear stiffness of the soil with strain

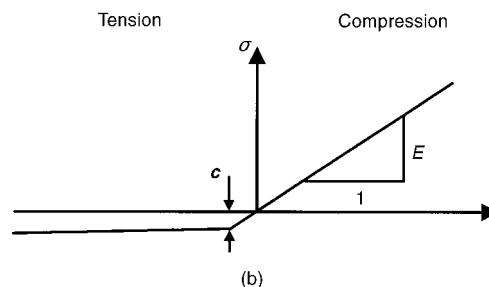
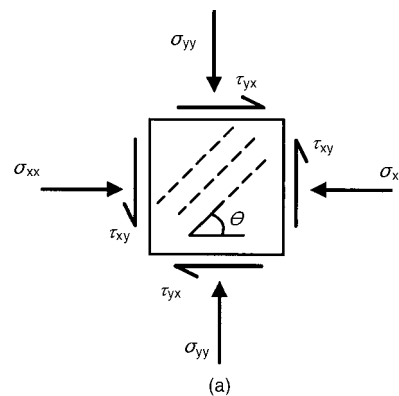


Fig. 7. Elastic no-tension masonry model: (a) cracked elastic no-tension material; and (b) uniaxial behaviour

become tensile, however, then tension cracks will form at an angle θ , which is the inclination of the major principal strain direction (taking compressive strains to be positive). In this case, the stiffness of the material in the direction perpendicular to the crack is reduced to a small value, and the tensile stress acting across the crack remains set at small residual value (c). The residual tension was set to 10 kPa in the calculations described in this paper. Residual tension and stiffness are necessary to avoid computational instability. The tensile strain normal to the crack is termed the 'crack strain'; this represents in an averaged way the intensity of cracking. For instance, a crack strain of $2000 \mu\epsilon$ could represent one 2 mm wide crack every metre or two 1 mm cracks every metre. The nature of cracking, which depends to a

Table 1. Damage categories⁴

Maximum principal tensile strain: %	Expected severity of damage
0-0.05	Negligible
0.05-0.15	Slight
0.15-0.3	Moderate
> 0.3	Severe

large extent on the heterogeneity of masonry, means that it is not regarded as possible to make a prediction of individual cracks and their widths. The masonry model has been validated against closed form solutions⁸ and also against field measurements of a large masonry building.²⁰

21. Elastic lintels were included in the finite element mesh to prevent failure of the masonry above the window and door openings. These lintels are visible in the mesh shown in Fig. 4.

22. Burland and Wroth¹ and Boscardin and Cording⁴ related crack damage to the level of maximum principal tensile strain developed in the structure. Boscardin and Cording,⁴ for example, propose the correlation shown in Table 1. A similar approach is adopted in this paper, in which the magnitude of computed crack strain is assumed to indicate the severity of the damage. The definition of crack strain used in this model is different from that of tensile principal strain in a more conventional continuum model such as that used by Boscardin and Cording.⁴ It is thought, however, that these strain definitions are sufficiently similar for the numerical correlations given in Table 1 to be a useful guide in interpreting the results of this numerical model.

Initial stresses

23. At the start of each analysis the weight of the building (when present) is set to zero, and a set of initial stresses is assigned to the soil elements. These stresses are based on a soil unit weight of 20 kN/m³ and a value of K_0 of 1.0. Next, the self-weight of the building is applied; in these calculations the masonry walls were taken to be 1 m thick with a unit weight of 20 kN/m³. This leads to the modification of the stresses in the ground and the development of stresses within the building. This procedure also generates strains within the masonry structure. To provide a suitable baseline from which the cracking caused by tunnel installation can be determined, the *strains* in the building are reset to zero before the start of the tunnel installation phase of the analysis. It is important, of course, that the *stresses* within the building are retained to ensure that the mesh remains in equilibrium. All displacements are also reset to zero at this stage. This procedure ensures that all building damage referred to hereafter is solely a result of the

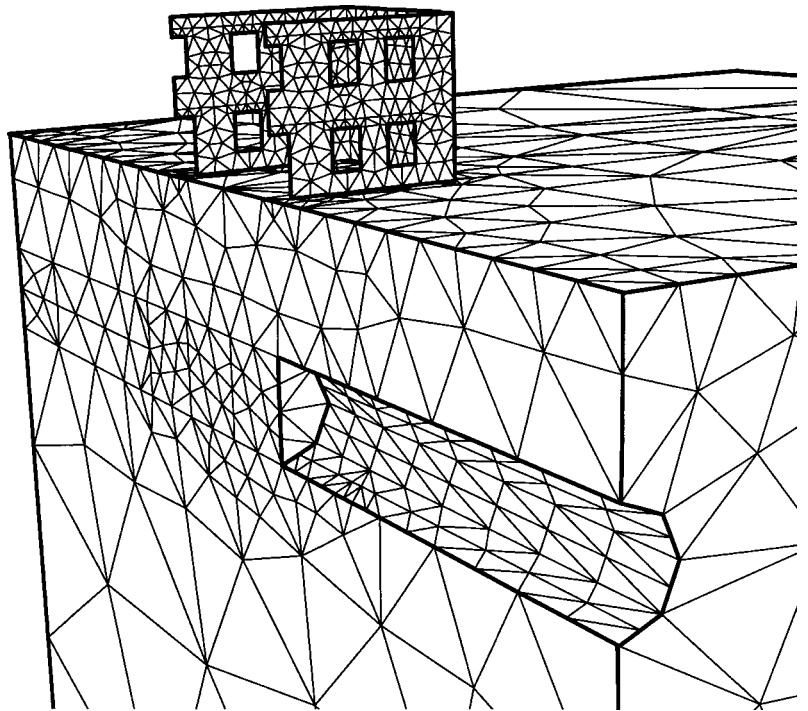


Fig. 8. Mesh with partially installed tunnel (symmetric run)

tunnel construction, and does not relate to the original construction and settlement of the building.

24. When the stresses have been initialized, the analysis proceeds by modelling tunnel excavation as described in the following subsection.

Tunnel installation procedures

25. Tunnel construction is simulated by incremental removal of elements within a pre-defined zone. This process involves the removal of terms in the global stiffness matrix that are associated with the excavated elements. Careful account also needs to be taken of the gravity loading applied to the elements to ensure equilibrium of nodal force on the tunnel surface. This aspect of the model is discussed by Augarde *et al.*⁶ and Augarde,⁷ and is based on work by Brown and Booker.²¹ A finite element mesh with a partly completed tunnel is shown in Fig. 8. The analyses described in this paper were for an unlined tunnel. While this is clearly a simplification, the magnitude of the ground movements are determined principally by volume loss. The parameters were chosen for the soil model to result in a volume loss of approximately 2%, which is typical of real tunnelling operations; the overall magnitude of the ground movements, therefore, should also be typical. After each analysis the amount of ground loss was evaluated by integration of the settlements over the surface of the mesh. These values are reported in Table 2.

26. Tunnel construction generally involves the installation of a structural liner to maintain the stability of the tunnel. Calculations in which shell elements were used to model the

tunnel liner are described by Augarde.⁷ The use of liner elements provides an improved means of modelling the tunnel installation process and allows the value of ground loss associated with the tunnel construction to be controlled with more precision. Liner elements add considerably to the complexity of the analysis, however, and a discussion of their use is beyond the scope of this paper.

Results of combined analyses

27. A variety of analyses were carried out during this research project; each of these analyses produced a large amount of output data. This paper, however, is concerned with just four analyses as indicated in Table 2.

28. Two sets of runs were carried out for each of the symmetric and non-symmetric cases. Coupled runs refer to the case where the building is coupled directly to the ground as previously described. Additional uncoupled runs were carried out, in which the surface settlements and associated horizontal movements were calculated for the case where the building was absent. These computed ground movements were then imposed directly on the finite element model of the building.

29. The purpose of the coupled analyses was to investigate the general mechanisms of soil-structure interaction that occur in this type of problem. The uncoupled analyses were intended to investigate the shortcomings of methods in which soil-structure interaction effects are not considered.

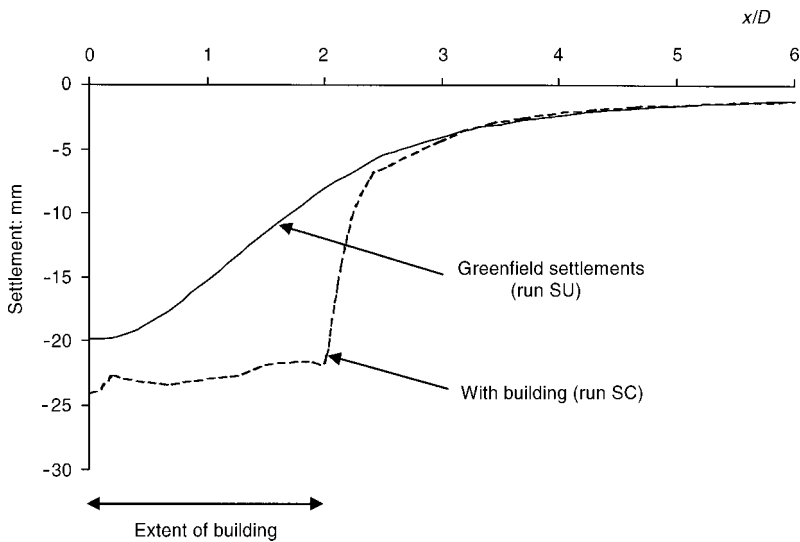
30. The tunnel was installed in four stages. Data on surface settlements and structural damage were computed at the end of each stage and were used to study the development of cracking within the building during tunnel installation. The crack damage results described in this paper, however, all relate to the case where tunnel installation is complete. Details of the earlier stages are reported by Liu.⁸

Symmetric analyses

31. The computed settlement profiles beneath the front façade (i.e. the façade facing the approaching tunnel) of the building for the two symmetric runs are shown in Fig. 9, where x is the distance from the façade centreline and D is the tunnel diameter. The building appears to act as a stiff beam spanning the settlement trough and, as a result, reduces substantially the magnitude of the differential settlements beneath it. The settlement gradients are increased immediately adjacent to the building, however, suggesting that any smaller structures in this region may suffer appreciable damage. Fig. 9 shows that, for the coupled analysis, the presence of the central door in the façade causes the settlement gradient to increase significantly near the plane of symme-

Table 2. Specification of runs

Reference	Calculation type	Coupled/uncoupled	Actual ground loss: %
SC	Symmetric	Coupled	2.4
SU	Symmetric	Uncoupled	2.29
NC	Non-symmetric	Coupled	1.91
NU	Non-symmetric	Uncoupled	1.8



try of the building. For a greenfield site the cross-sectional area of the final settlement trough at the ground surface would be expected to be constant with distance in the tunnelling direction. However, the weight and stiffness of a building causes the area of the settlement trough to vary in the tunnelling direction (although the total settlement volume, for undrained conditions, would be expected to be equal to the total volume loss in the tunnel). Fig. 9 shows that the presence of the building acts to increase significantly the area of the settlement trough beneath the front façade; this increase in local settlements is caused by the weight of the building.

32. The maximum greenfield settlement of 20 mm shown in Fig. 9 is about 70% of the value that would be suggested by current semi-empirical methods; the width of the settlement trough is also greater than might be expected. This discrepancy is typical of finite element models of tunnelling and is an indication of the complex nature of the soil strains that are developed around the tunnel. It should be noted that less sophisticated constitutive models than the one adopted here would lead to considerably larger discrepancies.

33. Figure 10 shows plan views of vertical settlement contours for the two symmetric runs; these plots also indicate the building outline. Two of these plots relate to the case where the tunnel heading is immediately beneath the building and the other two show the settlements when tunnel installation is complete. In

Fig. 9. Settlements of front façade (symmetric building, where SU = symmetric uncoupled analysis; SC = symmetric coupled analysis)

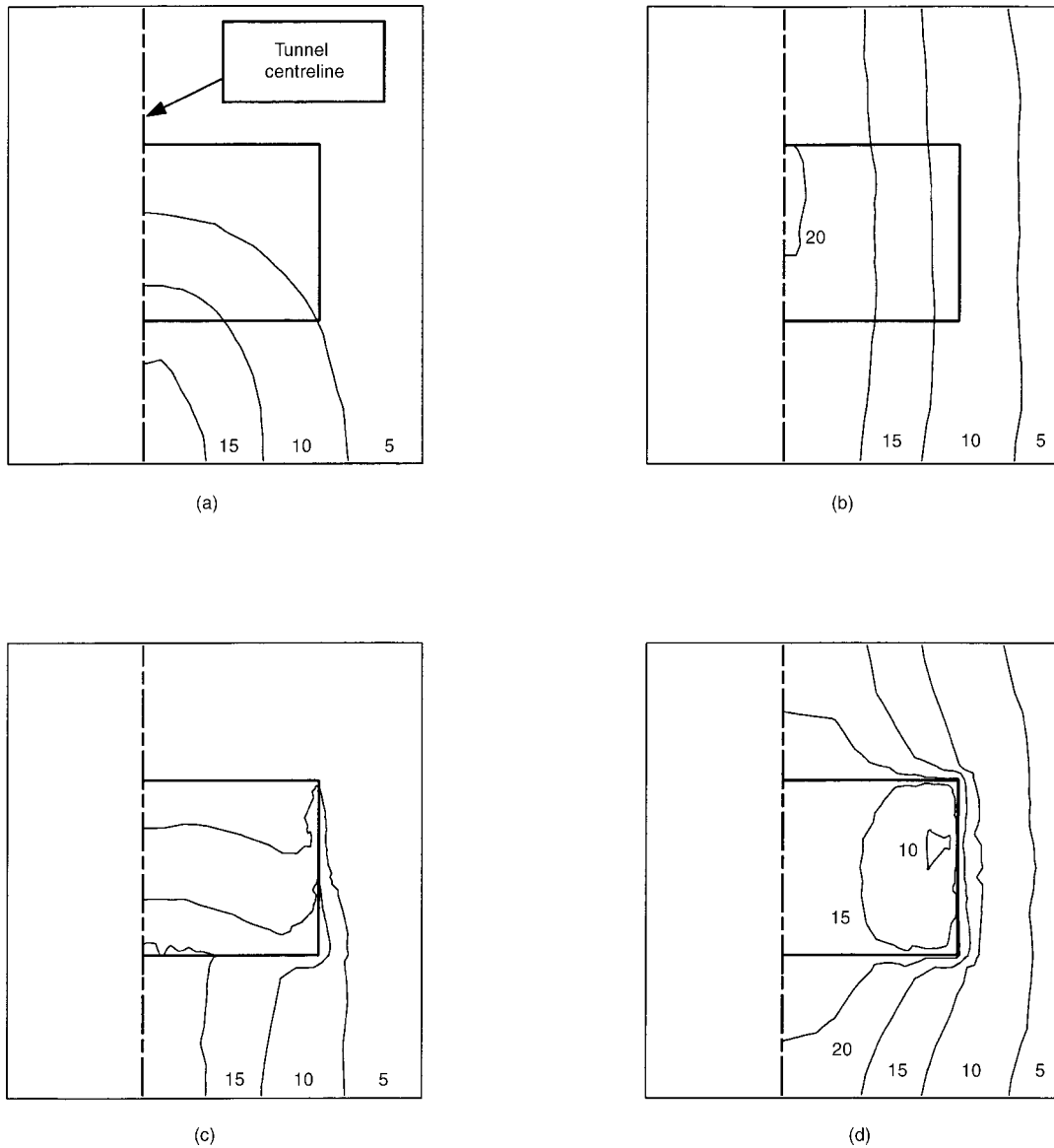


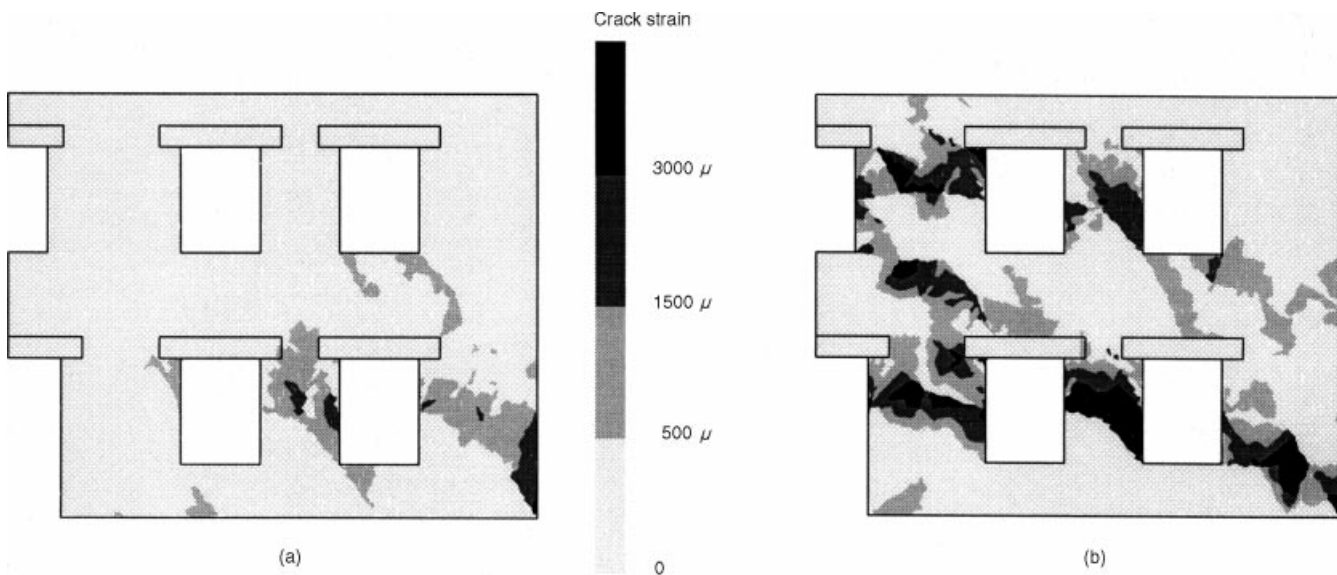
Fig. 10. Settlement contours (symmetric analysis): (a) uncoupled analysis—tunnel partially complete; (b) uncoupled analysis—tunnel complete; (c) coupled analysis—tunnel partially complete; and (d) coupled analysis—tunnel complete. All settlements are in mm

the absence of the building, the settlements at the end of tunnel installation are seen to be reasonably uniform along the tunnel axis (Fig. 10(b)); the slight non-uniformities are associated with mesh discretization effects. Fig. 10(d) confirms the pattern seen in Fig. 9 that the presence of the building tends to reduce the differential settlements beneath it.

34. Contours of crack strain in the front façade for the symmetric coupled run are shown in Fig. 11(a). It is thought that damage in areas where the crack strain is less than $500 \mu\epsilon$ would be negligible, and that larger values of crack strain indicate areas of more significant damage. Fig. 11(b) shows the crack strain plot for the equivalent uncoupled analysis (SU). The coupled analysis indicates a small zone at the bottom right-hand corner of the building where the shear stresses in the masonry lead to an area of severe cracking. In the uncoupled analysis, however, the predicted damage is substantially greater. Fig. 12 illustrates an

alternative approach to visualize the computed settlement damage, for the front façade in the coupled analysis. In this plot a single line is drawn at each stress point for which the crack strain exceeds $500 \mu\epsilon$. The direction of the line indicates the expected crack direction. At stress points where the strain exceeds $1000 \mu\epsilon$ then two parallel lines are drawn; when the strain exceeds $1500 \mu\epsilon$ three lines are drawn, and so on. This produces a plot in which the inclination of the lines indicates crack directions and the density of the lines gives a qualitative indication of the severity of cracking.

35. If the surface settlements given in Fig. 9 were imposed on an elastic model of the front façade of the building then, for both the uncoupled and coupled cases, the magnitude of maximum principal tensile strains would be significantly less than the computed crack strains. This is because the reduced stiffness in cracked masonry tends to attract further movement. Note that when strain becomes localized



(as predicted by the masonry model) then the largest strains are substantially greater than estimated average values. Elastic models do not exhibit this localization and are therefore regarded as less realistic.

Non-symmetric analysis

36. Surface settlement contour plots for the non-symmetric analyses are given in Fig. 13. Figs 13(a) and (b) show the settlement contours for the uncoupled analysis when tunnel installation is partially complete and at the end of tunnel installation respectively. The magnitudes of the settlements differ slightly from the equivalent symmetric analysis; this may be attributed to differences in mesh density. Corresponding plots for the coupled analysis are shown in Figs 13(c) and (d). The building is seen to increase the magnitude of the predicted settlements in the area immediately above the tunnel; these additional local movements are caused by the weight of the building. In contrast to the symmetric analysis, the structure has only a minor effect on the overall magnitude of the differential settlements at the

base of the building, although their distribution changes (see later).

37. Figure 14 shows the computed settlements along the bases of the front and rear façades for the coupled and uncoupled analyses. Both uncoupled analyses show a typical approximately Gaussian trough centred on the tunnel and with a maximum settlement of approximately 15 mm. The maximum settlements for the coupled analyses are significantly larger, principally because of the weight of the building. The front façade lies partly within the sagging portion of the uncoupled analysis trough and partly within a region of very mild hogging. Under these circumstances, the building is very stiff and the settlement variation along the front façade becomes almost linear. While the curvature is much reduced, the differential settlement is, however, significantly increased (because of the weight of the building). The rear façade lies entirely within the hogging region. In these circumstances, the building is very flexible and follows the hogging profile, but with larger differential settlements and curvatures, because of the weight of the building. Regions of large differ-

Fig. 11. Cracking in front façade (symmetric analysis): (a) coupled analysis; and (b) uncoupled analysis

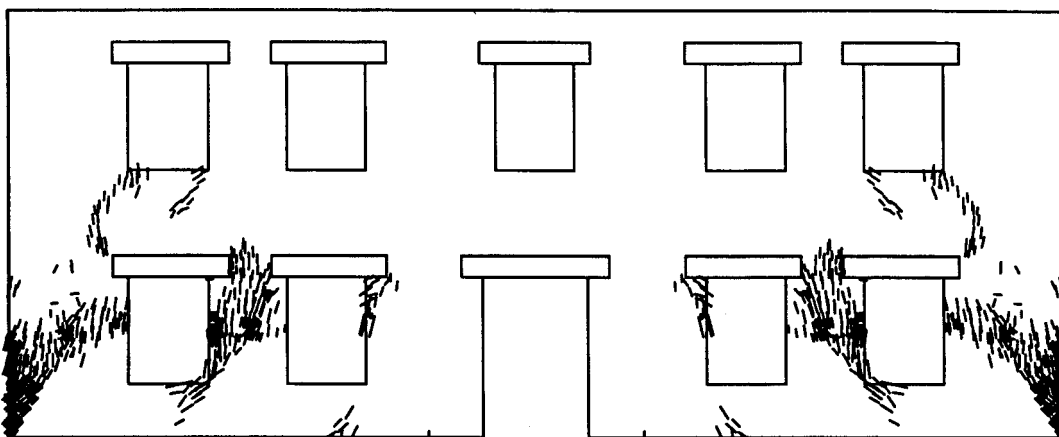


Fig. 12. Predicted crack patterns in front façade (symmetric coupled analysis)

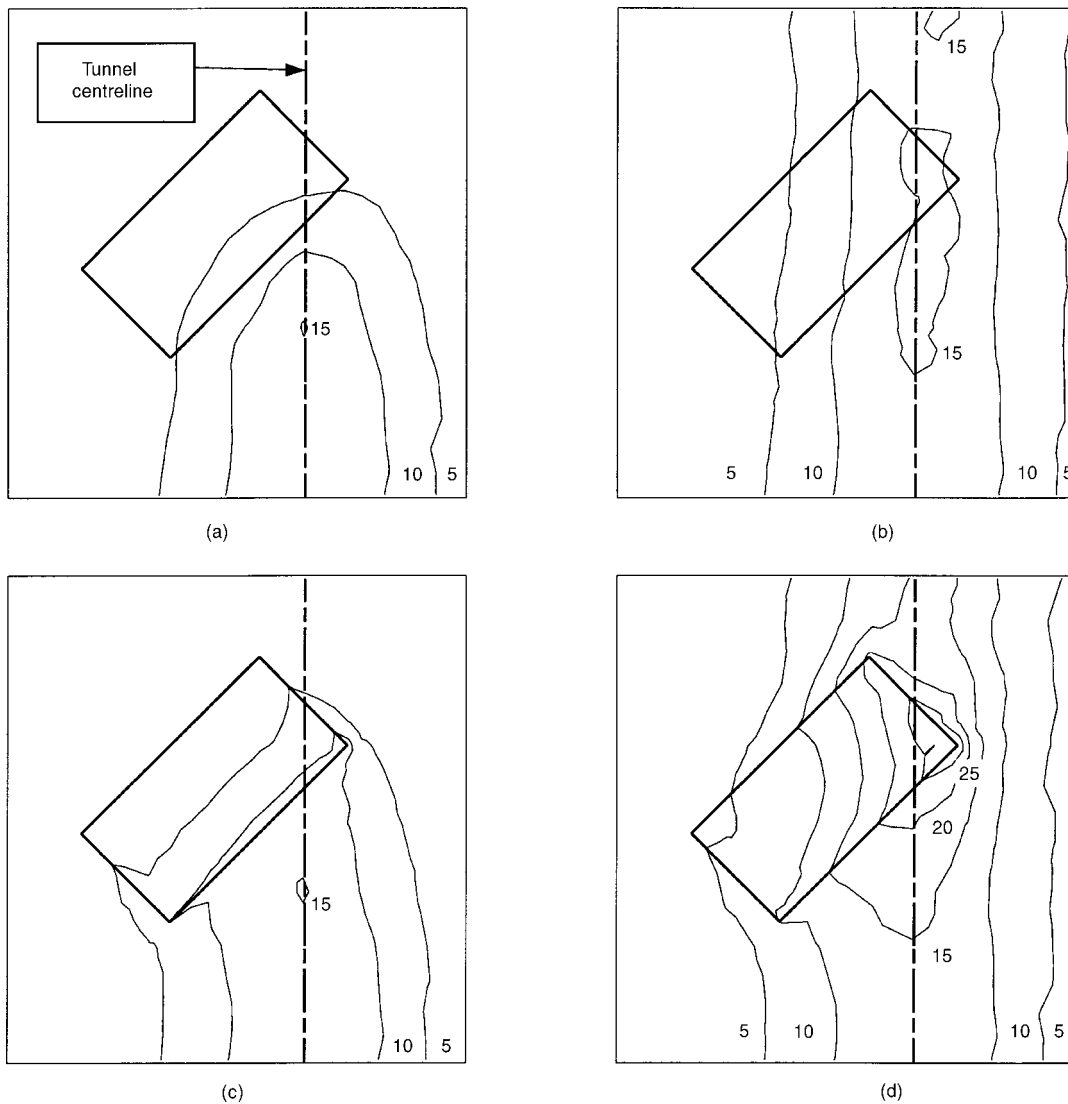


Fig. 13. Settlement contours (non-symmetric analysis): (a) uncoupled—tunnel partially complete; (b) uncoupled—tunnel complete; (c) coupled—tunnel partially complete; and (d) coupled—tunnel complete. All settlements are in mm

ential settlements immediately adjacent to the building are clear from the figure.

38. Crack strain contours (see Fig. 11 for the damage scale), and crack patterns, for the front and back façades of the building for the coupled analysis are shown in Fig. 15. Figs 15 (a) and (b) show that damage to the front façade is relatively minor although a small amount of cracking is evident at the top of the building. This is associated with tensile strains caused by the hogging displacements imposed on the façade. The hogging deformation applied to the back façade is more severe than that applied to the front, and this leads to a more extensive pattern of predicted damage.

39. Predicted crack strains for the uncoupled analysis are shown in Fig. 16. These results differ from those obtained from the coupled analysis (Fig. 13) in two important respects. First, damage to the back façade is more severe for the coupled than the uncoupled analysis. This is contrary to the normal expectation that uncoupled analysis is conservative. It appears that, in the coupled analysis, the

building weight acts to increase the local settlements, particularly near the corner of the building above the tunnel axis (see Fig. 13(d)). Overall, however, the ground loss is increased by a factor of only about 1.05. These settlements are transmitted to the rear façade by the stiff gable wall and act to increase the magnitude of the predicted damage. Second, it is seen that damage to the right-hand half of the front façade is less severe for the coupled than the uncoupled analysis. This portion of the façade is subjected to a sagging mode of deformation; significant loss of bending stiffness does not, therefore, occur and the local differential settlements are therefore reduced.

Performance of the building

40. These analyses show that the building interacts with the ground in a highly complex way. The weight of the building tends to increase the general magnitude of the settlements that are developed beneath it; the stiffness of the building may act to reduce differential settlements. The performance of the

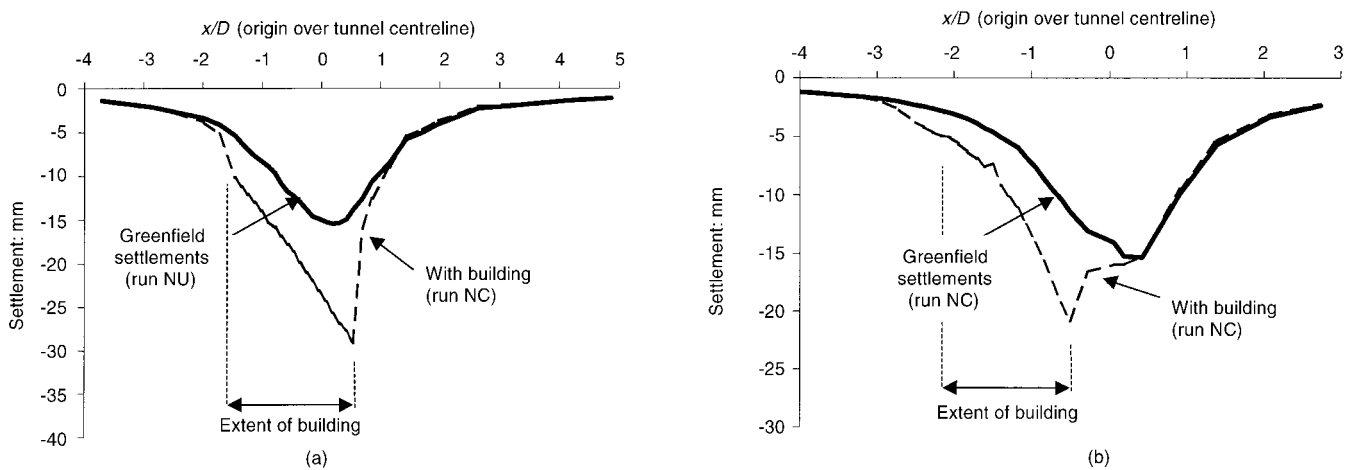


Fig. 14. Settlement of façade bases (non-symmetric analysis) for: (a) front façade; and (b) rear façade (where NU = non-symmetric uncoupled; NC = non-symmetric coupled)

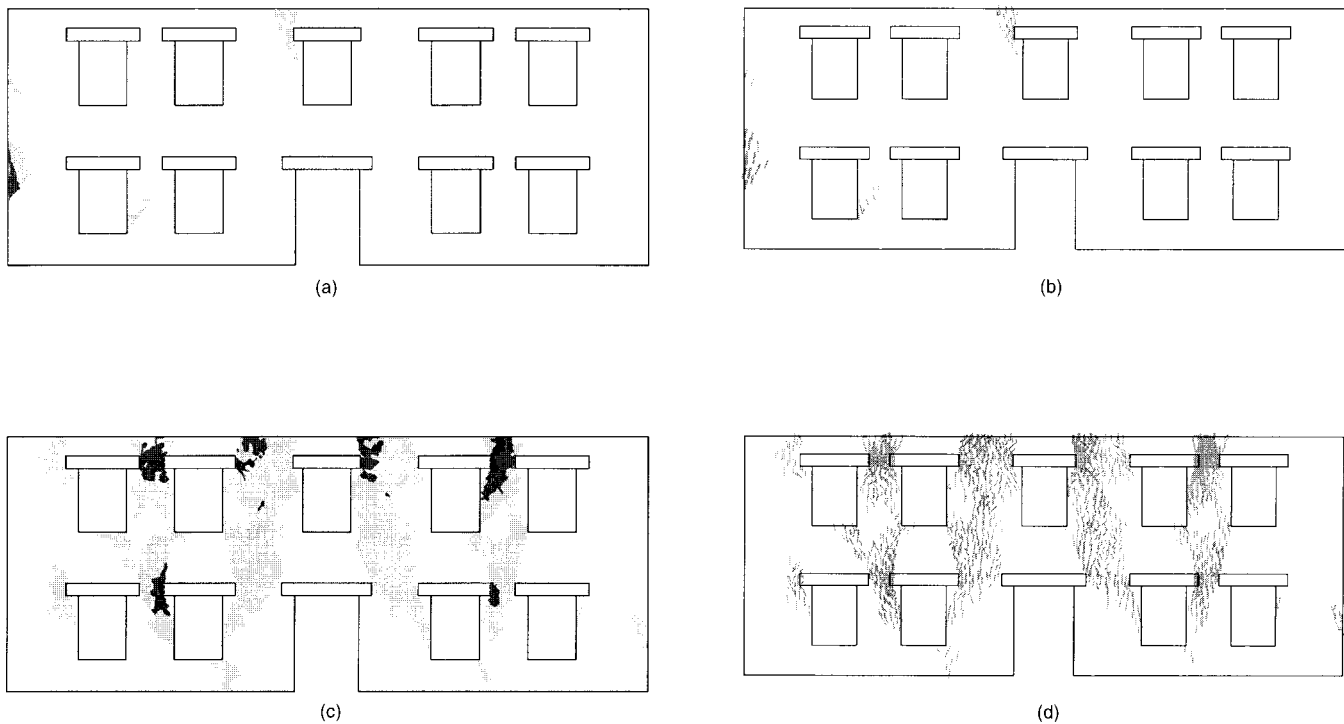


Fig. 15. Crack damage (coupled, non-symmetric analysis): (a) damage contours—front façade; (b) predicted crack patterns—front façade; (c) damage contours—back façade; and (d) predicted crack patterns—back façade

building depends critically on its position and orientation relative to the tunnel. Details of the foundations would further modify the building response.

41. Portions of the building subjected to sagging deformation are seen to exhibit a relatively stiff response. This aspect of building performance is discussed by Burland and Wroth,¹ who suggest that, in a sagging mode, the ground provides a certain amount of lateral restraint to the building foundations, thus lowering the position of the neutral axis and

ensuring that the lateral stresses in the façade are predominantly compressive. Under these conditions, only minor tensile cracking (associated mainly with shear effects) will occur and the façade will retain a substantial proportion of its bending stiffness. Predictions of crack damage based on a conventional uncoupled approach will therefore be excessively conservative (because the building stiffness reduces the differential settlements) and a coupled analysis is required to obtain improved damage and settlement predictions. Although the use of

a non-linear model for the building is preferable, interaction analyses based on an elastic structure (such as those described by Potts and Addenbrooke²²) may yield useful results for buildings deforming in a sagging mode.

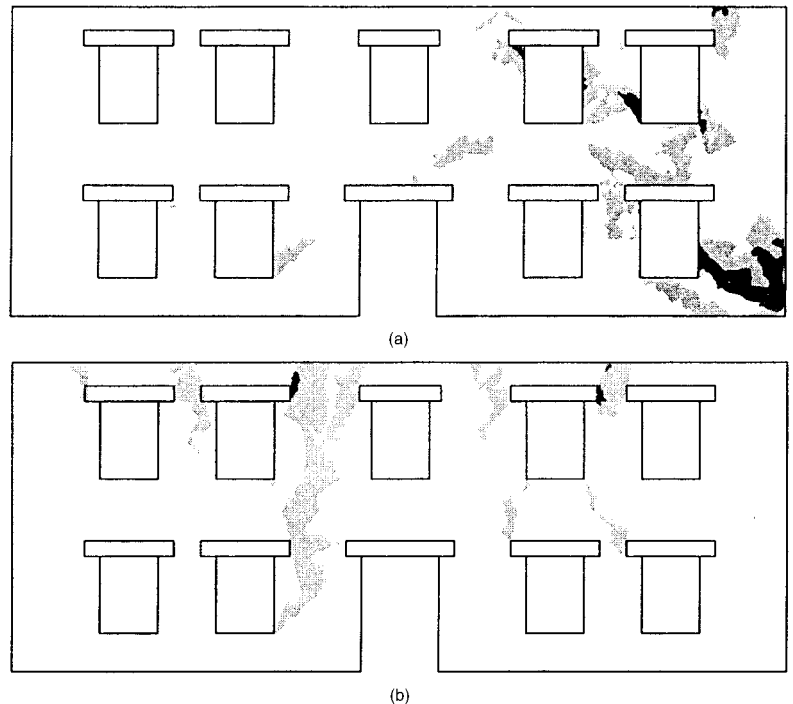
42. Lateral restraint provided by the ground reduces the extent of tensile stresses in the building for sagging deformation. For buildings or façades subjected to hogging, however, lateral ground restraint does not have this beneficial effect. Façades subjected to hogging are seen to develop patterns of vertical cracking and, as a result, suffer extensive loss of bending stiffness. This stiffness loss decreases substantially the effectiveness of the building in reducing the local differential settlements. Although the building stiffness is reduced, the results presented here show that important soil-structure effects still operate; for example, the weight of the building is seen to modify the profile of ground surface settlements. It is clear, however, that soil-structure interaction effects are less important for buildings subjected to hogging than sagging deformation. It is possible that a conventional uncoupled approach may give reasonable predictions for buildings that are subjected to hogging, although a coupled analysis of the sort described in this paper would generally be preferable. This is particularly the case when (as for the non-symmetric analysis presented here) different parts of the building are subjected to hogging and sagging deformations.

Computing hardware requirements

43. The analyses described in this paper were carried out using fast workstations configured with either 170 MHz processors and 128 Mbytes of RAM, or 200 MHz processors and 256 Mbytes of RAM. With these machines a typical run time for each of these analyses was eight days for the 256 Mbyte machine and twelve days for the 128 Mbyte machine. This was achieved using solution algorithms that were designed to achieve accurate and robust results rather than to minimize the run times. These lengthy run times ruled out the possibility of further calculations with refined meshes to study mesh density effects. Sets of analyses similar to those described in this paper have recently been run on the Silicon Graphics Cray Origin2000 supercomputer, OSCAR, at the Oxford Supercomputing Centre. Runs, comparable to those described here, take about five hours to complete on OSCAR when eight parallel processors are used. Further work is in progress to implement new solution procedures that are expected to lead to additional improvements in the speed of the analyses.

Conclusions

44. Three-dimensional finite element methods may be used to estimate the extent of



crack damage caused to masonry structures by nearby shallow tunnelling. The computations are complex and require dedicated computing facilities. This approach does, however, have the important advantage of being able to provide assessments that are based on a rational procedure to model the interaction effects between the building and the ground.

45. The numerical model described here was developed during an initial phase of research at Oxford University on settlement damage to masonry structures. The results presented here are intended to illustrate the mechanisms of interaction between buildings and the ground for the case of one example building and two tunnel positions. Work is currently under way on the comparison of the model with field data although a discussion of this is beyond the scope of this paper.

46. It is often assumed that soil-structure interaction effects will reduce the predicted tendency of a building to suffer settlement-induced damage. This trend is evident in the results presented in this paper for façades subjected to sagging deformations. When the building deforms in a hogging mode, however, the building is seen to be less effective in reducing differential settlements.

47. Current assessment methods are generally based on an uncoupled approach. For buildings subjected to sagging deformations it is seen that an uncoupled approach is likely to be excessively conservative. Soil-structure interaction effects when the building deforms in a hogging mode, although important, are less significant than for buildings subjected to sagging. For buildings subjected to hogging,

Fig. 16. Crack damage (uncoupled, non-symmetric analysis): (a) damage contours—front façade; and (b) damage contours—back façade

therefore, it is possible that a conventional uncoupled approach may give useful results.

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