Twist behaviour of buildings due to tunnel induced ground movement

J.N. Franzius
Geotechnical Consulting Group, London; (formerly Imperial College, London)

D.M. Potts & J.B. Burland
Imperial College, London

ABSTRACT: The assessment of building deformation due to tunnel induced ground movement has become an essential part in the planning and construction process of any tunnelling project in an urban environment. In current engineering practice the building is normally represented by a plane strain structure on which a settlement profile is imposed. This approach does not account for any three-dimensional (3D) deformation mode of the building. Recently published case studies, however, have demonstrated that 3D deformation such as twist occurs in buildings subjected to tunnel construction. Results from a 3D Finite Element (FE) parametric study are presented in which a building is represented by elastic shell elements. All buildings were in plan perpendicular to the tunnel axis while their stiffness, their size (width and length) and the tunnel depth were varied. It is shown how these different parameters influence the twist deformation. The study reveals a similar trend between twist and building size, building stiffness and tunnel depth as found by Potts & Addenbrooke (1997) for deflection ratio and horizontal building strain.

1 INTRODUCTION

The settlement assessment of buildings influenced by the construction of new tunnels normally assumes a two-dimensional (2D) deformation of the building. Case studies, however, have shown that buildings can be distorted three-dimensionally (3D) during the process of tunnel construction. The mechanisms of such 3D deformation are, however, not well understood.

In 1997, Potts & Addenbrooke introduced a relative stiffness approach to quantify the effects of the interaction between soil and building for 2D deformation. This method, despite its many simplifications, has been used in engineering practice since then (Mair & Taylor, 2001). Burland et al. (2001) pointed out the importance to extend this approach to include 3D building distortion.

This paper presents a 3D Finite Element (FE) parametric study of twist deformation of buildings. Modification factors, similar to those adopted by Potts & Addenbrooke (1997) are introduced to quantify the twist deformation. This study was part of a research project, carried out at Imperial College, London, to investigate the behaviour of buildings due to tunnelling induced subsidence. In this research 2D and 3D Finite Element (FE) analysis was adopted to model the tunnel-soil-structure interaction problem (Franzius, 2004).

2 CASE STUDIES

There are only a limited number of case studies available which address the problem of twisting. Standing & Selman (2001) present precise levelling data along the tunnels of the Northern Line (London Underground) which were subjected to movement caused by the construction of the Jubilee Line Extension (JLE) near Waterloo Station, London. The route of the JLE tunnel was approximately perpendicular to the existing tunnels of the Northern Line.

They introduced the term 'relative twist rotation' to quantify the twist measured in the tunnels. The twist was calculated from the difference in settlement between a pair of levelling points installed opposite each other on the cross section of the tunnel. This difference divided by the distance between the two points leads to an angle (more precisely to the sine of this angle which is the angle itself for small values) expressed in terms of arc minutes.

Standing & Selman (2001) showed that relative twist rotation occurred when the JLE tunnel construction passed beneath the existing Northern Line tunnels. However, their results demonstrate that relative twist rotation was only temporarily and some counter twist was observed as the construction of the tunnel advanced beyond the existing tunnel.

The study demonstrated that structures which are perpendicular to the route of the new tunnel construction experience twist mainly during the construction of the tunnel and twist deformation reduces or even vanishes when the tunnel face moves away from the structure.

Not all existing structures are, however, perpendicular to the tunnel to be constructed. Cooper & Chapman (2000) present measurements from the running tunnel of the Picadilly Line in Heathrow, London, which was affected by the construction of the Heathrow Express tunnel. The centre lines of the tunnels crossed at a skew angle of approximately 70°.

Cooper & Chapman (2000) expressed rotation by the differential settlement of a pair of opposite measurements points across the tunnel cross section (noting that the distance between the points was constant). Such pairs were installed over a certain length of the existing tunnel to study the longitudinal distribution of 'rotation'. In these definitions the longitudinal change of differential settlement was not considered.

As Standing & Selman (2001), Cooper & Chapman (2000) report rotation of the existing tunnel towards the new tunnel occurred as the new tunnel face approached. As tunnel construction passed beyond the existing tunnel, rotation reduced. As the new tunnel was subsequently enlarged (from the same direction) the rotation increased again followed by a reduction as the enlargement passed beyond the existing tunnel. However, some rotation remained in the Picadilly Line tunnel and Cooper & Chapman (2000) concluded that this behaviour is due to the skew angle between the two tunnel routes.

Similar conclusions were drawn by Franzius (2004) when calculating the twist for Elizabeth House, affected by the construction of the JLE, London. The tunnels had a skew angle with respect to the building of approximately 35°. Interpreting the precise levelling measurements reported by Standing (2001) he showed that the twist in the building changed during the construction of the twin tunnels beneath the 10-storey structure and that the structure remained twisted as tunnel construction moved beyond the building.

These different case studies showed that there are two different mechanisms of twist behaviour: the permanent twist which is generated by tunnel construction with a skew angle between the tunnel and the existing structure; and temporary twist which develops due to the three-dimensional progress of the tunnel excavation. Depending on the geometry both effects can occur together. The case studies, however, also highlighted the problem of defining twist deformation. Before presenting a parametric FE study a definition of twist will be proposed.

3 DEFINITION OF BUILDING TWIST

Two of the above case studies used the rotation of existing structures to quantify the twist deformation. Cooper & Chapman (2000) pointed out that this definition includes tunnel rotation caused by the overall tunnel settlement and distortion of the tunnel circumference. For a long structure (such as an existing tunnel) this approach is applicable. When buildings of different geometry are subjected to tunnel induced settlement, both dimensions, building width and length have to be considered.

The twist expression commonly used in structural engineering incorporates both geometric parameters. The definition is based on the torsion of a constant circular cross section of a rod (Timoshenko, 1955) as shown in Figure 1. A section of this circular rod with a diameter d and a height dx shows a rotation $d\varphi$ of its upper cross section with respect to its base. If the shaft shown in this figure is only twisted by a torque at its end the quantity $d\varphi/dx$ is constant over x. This ratio is the 'angle of twist per unit length' and will be denoted with the symbol

$$\Theta = d\varphi / dx \tag{1}$$

From the above definition it follows that Θ is not dimensionless but has the unit [1/*length*].

This definition can be extended to more general geometries such as shells and plates. Timoshenko & Woinowsky-Krieger (1959) show that the twist of a shell, shown in Figure 2, can be expressed as

$$\Theta_{xy} = \frac{1}{r_{xy}} = \frac{\partial^2 w(x,y)}{\partial x \partial y} \tag{2}$$

were $w(x, y)$ is the displacement normal to the shell, r_{xy} is the curvature of the surface with respect to the x and y axes defined in the figure and Θ_{xy} is the twist with respect to these axes. With this definition the twist is the rate of change of slope in the x-direction as

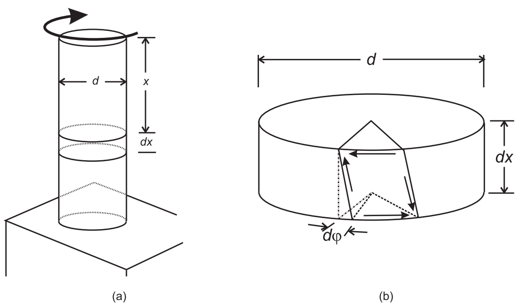

Figure 1. (a) Torsion of a single rod; (b) definition of 'angle of twist per unit length'.

one moves in the y-direction. It should be noted that this expression is, strictly speaking, only applicable to thin shells with small deformation (Timoshenko & Woinowsky-Krieger, 1959).

In this equation the displacement $w(x, y)$ depends on the coordinates x and y. This function is normally not known when analysing field measurements. Instead measurements are available for discrete points. Figure 3 shows this situation where the displacement is measured only at 4 corner points. If a linearly varying displacement is assumed on the edges of the rectangle and on any line parallel to these edges then the twist can be calculated from the differential settlement of the corners as

$$\Theta_{BL} = \frac{\frac{S_{v,a} - S_{v,b}}{L} - \frac{S_{v,c} - S_{v,d}}{L}}{B} \tag{3}$$

where $S_{v,a}, S_{v,b}, S_{v,c}, S_{v,d}$, are the settlement of the four corners, B and L are the width and the length of the building, respectively. The index 'BL' indicates that the twist is calculated along axes parallel to the building width B and length L. It will be omitted in the following paragraphs.

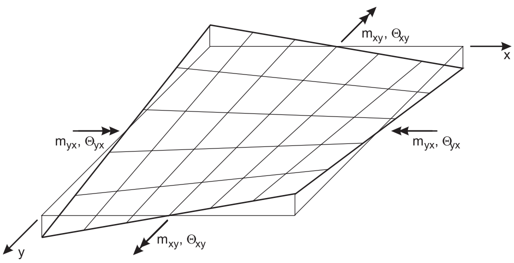

Figure 2. Twist of a thin shell caused by moments m_{xy}.

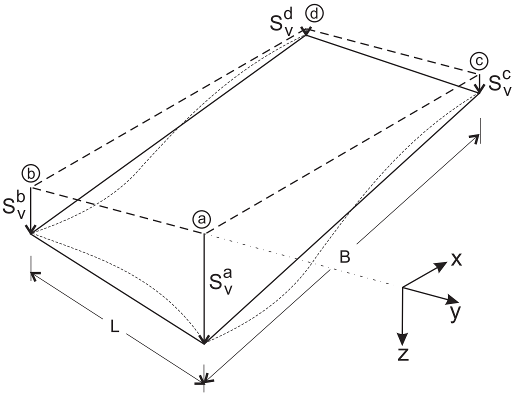

Figure 3. Deformation of shell and definition of displacement of corner nodes.

It has been stated earlier that the above definition of twist is not dimensionless in contrast to other building deformation criteria such as deflection ratio or horizontal strain which have no dimension. The dimension [1/*length*] is a consequence from twist being a one dimensional deformation of a two dimensional structure.

4 FE ANALYSIS

The 3D FE parametric study was performed using the Imperial College Finite Element Programme (ICFEP). Reduced integration was used, with an accelerated modified Newton-Raphson scheme with a substepping stress point algorithm for solving the nonlinear FE equations (Potts & Zdravkovic, 1999, 2001).

A typical mesh is shown in Figure 4 with a tunnel depth of 20 m and a diameter of 4.1 m. The soil was modeled by 20-node solid elements while the tunnel lining and the surface structure were represented by 8-node elastic shell elements (Schroeder, 2003). Due to symmetry only half of the problem was modeled. The mesh shown in the figure includes half of a 30 m × 10 m building. The nodes a and b, marked on the surface structure lie on the center line of the building. All vertical boundaries were modeled as planes of symmetry while on the bottom plane the displacement was restricted in all directions.

Tunnel construction was modeled by successively excavating 2.5 m long 'slices' of soil elements in front of the tunnel face and activating one ring of tunnel lining after each excavation step (i.e the soil remained unsupported over 2.5 m behind the face). The tunnel excavation was modeled over 100 m, which required 40 excavation steps. The longitudinal distance to the remote vertical boundary of the mesh was 55 m (i.e. 13 × D) while the mesh extended laterally 100 m (24 × D). Both the length of tunnel excavation and the dimension of the mesh were larger than in most 3D FE studies published by other authors in recent years.

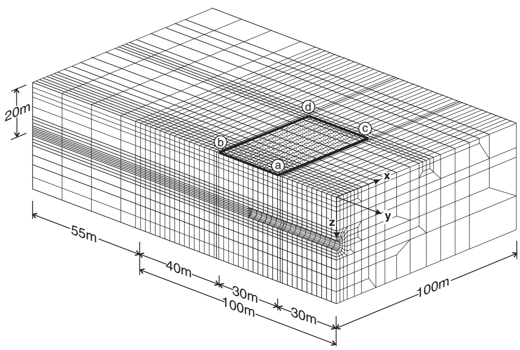

Figure 4. Example of 3D finite element mesh.

Table 1. Building stiffness.

Building	Bending stiffness *EI* [kN/m^2/m]	Axial stiffness *EA* [kN/m]
Slab	6.47×10^3	3.45×10^6
1-storey	2.00×10^7	6.90×10^6
3-storey	2.00×10^8	1.38×10^7
5-storey	6.98×10^8	2.07×10^7
10-storey	4.39×10^9	3.80×10^7

The soil profile consisted of London Clay represented by a non-linear elastic plastic constitutive model. The non-linear pre-yield behaviour was based on a model described by Jardine et al. (1986) while the plastic yield surface was described by a Mohr-Coulomb model. The parameters for these models were the same as adopted by Potts & Addenbrooke (1997) who performed a 2D study of a similar building-tunnel interaction problem.

The initial stresses in the ground were controlled by the assumed bulk unit weight of 20 kN/m^3 and a hydrostatic pore water profile with a water table 2 m below ground surface and a zone of suction in the top 2 m of soil. The initial coefficient of earth pressure at rest was $K_0 = 1.5$.

The soil was modeled to behave undrained by using an uncoupled analysis, i.e. by specifying effective stress soil parameters and a high value of the bulk stiffness of the pore water.

The stiffness parameters of the elastic shell elements modeling the building were chosen to represent different numbers of storeys (Potts & Addenbrooke, 1997). 1-, 3-, 5- and 10-storey buildings were included in the parametric study. The building stiffness values (per running metre) are summarized in Table 1. Corresponding values of the Young's modulus *E* and the thickness *t* were calculated from these values and used as input parameters for the elastic shell model. The building was modeled to be weightless. While this is a simplification, it allows for the investigation of the behaviour of twist independently from other parameters. Furthermore, Franzius et al. (2004) have shown that when considering realistic combinations of building stiffness and weight, the weight only has a small influence compared with the building stiffness.

Three different building geometries were included in the study varying both the building length and width (parallel and transverse to the tunnel axis). All buildings were perpendicular to the tunnel and had no eccentricity with respect to the tunnel axis.

5 PARAMETRIC STUDY

To investigate the behaviour of building twist 8 different building scenarios were analysed. The results

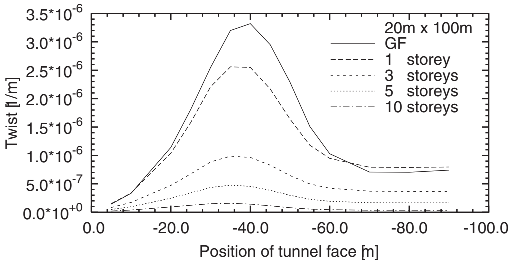

Figure 5. Development of twist with tunnel progress. Results from FE analysis of 20 m × 100 m geometries.

of these analyses were compared with the result of an analysis modeling Greenfield conditions.

For a 20 m × 100 m (in plan dimension) building, 1-, 3-, 5- and 10-storeys were modeled. For the 3-storey structure 2 further building geometries (30 m × 100 m and 20 m × 66 m) were included in the study. The tunnel depth was 20 m with a tunnel diameter of 4.1 m. One additional analysis was performed with a 3 4 m deep tunnel.

The above proposed definition (Equation 3) of twist was used and the settlement at the 2 building corners (i.e. nodes c and d in Figure 4) and at the center line (nodes a and b) was used to calculate the twist. The sign convention adopted is when looking in the transverse direction towards the route of the tunnel (i.e. from node c to a in Figure 4) a positive sign denotes an increase in anticlockwise rotation. For the building half analysed this means that, moving towards the tunnel, a positive twist increases the rotation towards the side where the tunnel construction comes from.

The influence of building stiffness on twist is illustrated in Figure 5 for the 20 m × 100 m building geometry. Twist is plotted against position of the tunnel face. The tunnel face encounters the plan of the building at $y = -30$ m passes beneath its centre line at -40 m and from -50 m it is beyond the building. Different curves are given for the greenfield and the different building cases. The twist for the greenfield conditions are calculated in the same manner as for buildings (i.e. adopting the same geometry). For the Greenfield case twist increases as the tunnel approaches the site. The maximum twist is reached at a face position of $y = -40$ m i.e. beneath the transverse centre line of the site. The twist reduces but then remains approximately constant for a tunnel face position beyond $y = -70$ m. Consequently the high values of twist are only of temporary nature. However, in this plane strain situation (i.e. no building present) no twist should remain after the tunnel face has moved to a certain distance from the site. The final value of twist deformation at the end of the analysis ($y = -90$ m) indicates that the longitudinal dimension of the FE mesh is not sufficient to develop this plane strain situation.

As part of this research project, Franzius (2004) investigated the influence of the mesh dimension on the settlement result of 3D FE analysis. Similar to the transverse settlement profile which has been found by several authors to be too wide when compared with field data, Franzius (2004) concluded that the longitudinal settlement trough is too wide to develop 'steady state' settlement conditions behind the tunnel face towards the end of the analysis (i.e. no further change of settlement at a certain point as excavation continues). For $K_0 = 0.6$, Vermeer et al. (2002) showed that in a 3D analysis the tunnel has to be constructed approximately over a distance of $10 \times D$ before steady state settlement established $5 \times D$ behind the tunnel face. Franzius (2004) showed that for a high K_0-regime, such as modeled in this study, this distance increases. No steady state conditions developed at the end of the analyses included in this parametric study despite the fact that the dimension of the FE mesh were larger than in most 3D studies recently published by other authors. It should be noted that an increase in mesh size, and therefore number of elements was not possible with the computational resources available at the time of the study. However, this study shows general trends in the building behaviour.

Analyses including a building give a similar picture as the greenfield analysis. Figure 5 shows that the peak twist is reached when the tunnel face is beneath the building. The influence of the structure's stiffness can clearly be seen in this graph when comparing the different curves: buildings with a higher stiffness show less twist deformation. To compare the building cases with the Greenfield situation a twist modification factor M^{Θ} can be introduced which is defined as the ratio between the peak twist of the building case over the maximum twist in the corresponding Greenfield scenario. This definition is similar to the modification factors introduced by Potts & Addenbrooke (1997) for 2D building deformation (however, noting that the twist modification factor does not consider whether the corresponding peaks occur at different positions).

The 1-storey building gives a modification factor of $M^{\Theta} = 0.77$. This value reduces with increasing building stiffness and the 10-storey building shows $M^{\Theta} = 0.047$. In all cases the twist in the building remains approximately constant after the tunnel face passes beyond $y = -70$ m.

Figures 6 and 7 investigate the influence of the in plan building geometry on the twist behaviour. 3-storey buildings of 20 m × 100 m, 30 m × 100 m and 20 m × 66 m are compared with corresponding greenfield situations. Figure 6 presents the results for the greenfield analyses by plotting the twist for each geometry against the position of the tunnel face. It can be seen that the largest geometry (30 m × 100 m) develops the smallest twist deformation. The curve for the 20 m × 100 m geometry is narrower and shows

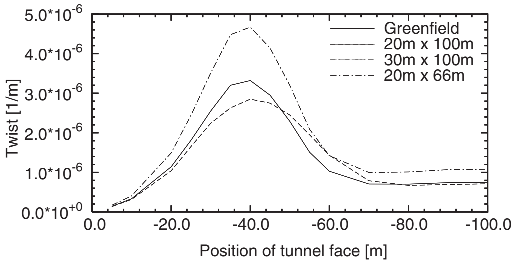

Figure 6. Development of twist with tunnel progress for different greenfield geometries.

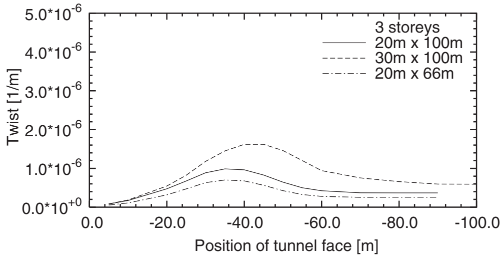

Figure 7. Development of twist with tunnel progress for different building geometries.

a higher peak twist while the 20 m × 66 m geometry exhibits the highest value of twist deformation.

It should be noted that the different curves for different greenfield geometries are a consequence of the application of Equation 3, i.e. calculating twist from the settlement at the 4 corners of half of the geometry. If the twist was calculated locally (for example by using the settlement at each surface node of the FE mesh) the peak twist found within each geometry would be identical. However, this would not be the case for the building cases where the twist depends on both geometry and building stiffness.

Figure 7 shows results from 3-storey buildings with the same geometries. The graph reveals the opposite trend to the previous graph. The largest structure (30 m × 100 m) develops the highest value of twist while the smallest building (20 m × 66 m) exhibits the smallest peak value. This trend of smaller structures leading to smaller values of twist deformation shows that smaller buildings show a stiffer response than large structures. In contrast to the small twist deformation, it was found that the stiffer building response leads to an increase rigid-body motion of the whole building, i.e. the entire structure rotates towards the tunnel face as it approaches and rotates back as the excavation passes beyond the buildings.

Decreasing twist for reducing building size in combination with an increasing peak twist for

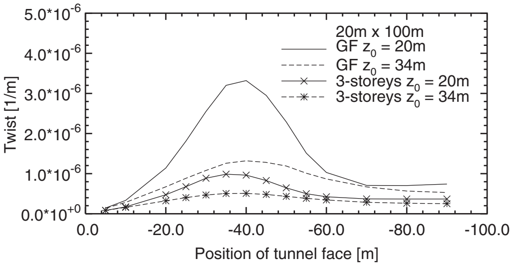

Figure 8. Development of twist depending on tunnel depth.

corresponding greenfield situation leads to a reduction in M^{Θ} with decreasing geometry: The 3-storey $30\,\mathrm{m} \times 100\,\mathrm{m}$ case shows $M^{\Theta} = 0.57$ which reduces to 0.15 for the $20\,\mathrm{m} \times 66\,\mathrm{m}$ geometry.

The influence of the tunnel depth z_0 is investigated in Figure 8 which plots the results of a $20\,\mathrm{m} \times 100\,\mathrm{m}$ building geometry (both greenfield and a 3-storey structure) subjected to tunnel excavation at $z_0 = 20\,\mathrm{m}$ and 34 m. It can be seen that for both cases, greenfield and building, the shallower tunnel generates higher values of twist. However, when calculating the modification factors it is the 34 m deep tunnel which leads to a higher value ($M^{\Theta} = 0.38$) compared with the shallower $z_0 = 20\,\mathrm{m}$ tunnel ($M^{\Theta} = 0.30$).

This study reveals that the twist modification factor M^{Θ} exhibits similar trends as the modification factors for deflection ratio and horizontal strain defined by Potts & Addenbrooke (1997): An increase in building stiffness (due to an higher number of storeys) reduces M^{Θ} while increasing building length and/or width increases it. Increasing z_0 also increases M^{Θ}. Because of the limited number of analyses included into this study it is, however, not possible to develop any design charts as Potts & Addenbrooke (1997) did for 2D building deformation.

6 CONCLUSIONS

This paper investigated the development of twist in structures which are affected to tunnel construction. Recently published case studies show that twist deformation can occur temporarily in a structure if its in plan geometry is approximately perpendicular to the tunnel axis. In structures whose centre line has a skew angle to that of the tunnel twist deformation remains after the tunnel has passed the structure.

There is no consistent definition of twist used in previously published case studies. For this paper a definition of twist derived from expressions used in structural engineering was adopted. By considering both building length and width this expression is not dimension-less but has the dimension [1/*length*].

A parametric 3D FE study was presented which modeled a surface structure, such as a building. The building was perpendicular to the tunnel and its stiffness and its size were varied.

The study highlighted problems which occur in 3D FE modeling where no ‘steady state’ settlement conditions develop during the analysis. Similar problems have been reported by other authors.

For the different building scenarios analysed, the twist behaviour was quantified by introducing twist modification factors M^{Θ} which normalize the building’s twist deformation against the corresponding Greenfield behaviour. The study revealed that these factors reduce with increasing building stiffness and increase with increasing building geometry and increasing tunnel depth. This study, therefore, revealed a similar behaviour in 3D building deformation as previously observed for 2D building distortion.

ACKNOWLEDGEMENTS

This work was part of a research project funded by the Engineering and Physical Sciences Research Council (EPSRC) with industrial collaboration with the Construction Industry and Research Information Association (CIRIA), the Geotechnical Consulting Group (GCG) and London Underground Limited.

REFERENCES

Burland, J.B., Standing, J.R. & Jardine, F.M. (2001). Assessing the risk of building damage due to tunneling – lessons from the Jubilee Line Extension, London. *Geotechnical Engineering. Meeting society’s need*. Vol. 1, 17–44.

Cooper, M.L. & Chapman, D.N. (2000). Settlement, rotation and distortion of Piccadilly Line tunnels at Heathrow. *Proc. of the International Symposium on Geotechnical Aspects of Underground Construction in Soft Ground*. Balkema, Rotterdam.

Franzius, J.N., Potts, D.M., Addenbrooke, T.I. & Burland, J.B. (2004). The influence of building weight on tunneling-induced ground and building deformation. *Soils and Foundations* 44(1), 25–38.

Franzius, J.N. (2004). Behaviour of buildings due to tunnel induced subsidence. *PhD Thesis*, Imperial College, University of London.

Jardine, R.J., Potts, D.M., Fourie, A.B. & Burland, J.B. (1986). Studies of the influence of non-linear stress-strain characteristics in soil-structure interaction. *Geotechnique*, 36(3), 377–396.

Mair, R.J. & Taylor, R.N. (2001). Elizabeth House: settlement predictions. Chapter 14 of: Burland, J.B., Standing, J.R. & Jardine, F.M. (eds), *Building response to tunneling*. Vol. 2.

Potts, D.M. & Addenbrooke, T.I. (1997). A structure’s influence on tunneling-induced ground movement. *Proc. Instn. Civ. Engrs. Geotech. Engineering*, 125, 109–125.

Potts, D.M. & Zdravkovic, L. (1999). *Finite Element analysis in geotechnical engineering: Theory*. Vol. 1. Thomas Telford, London.

Potts, D.M. & Zdravkovic, L. (2001). *Finite Element analysis in geotechnical engineering: Application*. Vol. 2. Thomas Telford, London.

Schroeder, F.C. (2003). The influence of bored piles on existing tunnels. *PhD Thesis*, Imperial College, University of London.

Standing, J.R. (2001). Elizabeth House, Waterloo. Chapter 30 of: Burland, J.B., Standing, J.R. & Jardine, F.M. (eds), *Building response to tunneling*. Vol. 2.

Standing, J.R. & Selman, R. (2001). The response to tunneling of existing tunnels at Waterloo and Westminster. Chapter 29 of: Burland, J.B., Standing, J.R. & Jardine, F.M. (eds), *Building response to tunneling*. Vol. 2.

Timoshenko, S. (1955). *Strength of materials; Part 1: Elementary theory and problems.* 3. edn. D. Van Nostrand, New York.

Timoshenko, S. & Woinowsky-Krieger, S. (1959). *Theory of plates and shells*. 2 edn. McGraw Hill, New York.

Vermeer, P.A., Bonnier, P.G. & Möller, S.C. (2002). On a smart use of 3D-FEM in tunneling. *Proc. of 8th Int. Symp. on Num. Models in Geomech – NUMOG VIII*, 361–366. Balkema, Rotterdam.