Geotechnical Aspects of Underground Construction in Soft Ground, Kastner, Emeriault, Dias, Guilloux (eds)
© 2002 Spécifique, Lyon. ISBN 2-9510416-3-2

The influence of building weight on the relative stiffness method of predicting tunnelling-induced building deformation

J. N. Franzius & T. I. Addenbrooke
Imperial College of Science, Technology & Medicine, London, UK

ABSTRACT: In urban areas, the ground movements induced by tunnelling can distort and damage overlying buildings. Potts & Addenbrooke (1997) presented new design charts to assist in the assessment of building damage in response to tunnelling. The charts were based on the results of two-dimensional plane-strain finite element analyses of tunnelling beneath buildings. Their analyses considered variations in the bending and axial stiffness of a building relative to the soil stiffness, the width of the building, its position relative to the tunnel axis, and the tunnel depth. In all cases weightless elastic beams represented the building. This paper presents the results of analyses that additionally consider building weight in order to investigate its influence on the tunnelling-induced deformation of a structure.
The effect of the self weight of buildings with a range of axial and bending stiffness is quantified. This paper demonstrates that the trend in deflection ratio with building stiffness is very similar for the wide range of weights investigated. Following the approach of the earlier publication the deflection ratios are then compared with the greenfield values to give modification factors. The results reveal an increase in these factors as weight increases. But when realistic combinations of weight and stiffness are identified the modification factors are shown to lie very close to the "weightless" design lines provided by Potts & Addenbrooke (1997).

1 INTRODUCTION

The prediction of settlement induced by tunnel construction has become an important issue. The current design practice is an empirical approach based on data of settlement troughs from recent tunnelling projects (O'Reilly and New, 1982). For the prediction of subsidence of buildings the characteristics of the structure are neglected and the expected deformed shape of the greenfield site is applied in order to assess the damage of a building using limiting criteria such as deflection ratio and horizontal strain (Burland and Wroth, 1974; Boscardin and Cording, 1989). This approach can be conservative as it assumes the building to be infinitely flexible and to follow the greenfield settlement trough.

Potts & Addenbrooke (1997) introduced the possibility of including the building/soil relative stiffness into this design approach. The basis of the new method was an extensive parametric suite of finite element analyses. The main simplification in their work was that the buildings were modelled as weightless. This paper explores the influence of building weight on their work.

2 THE RELATIVE STIFFNESS APPROACH

Potts & Addenbrooke (1997) proposed a method which includes the stiffness of a structure in the prediction of its deformation due to tunnelling induced subsidence. They presented the results of a parametric study including over 100 two-dimensional plane strain analyses. The structure was modelled by an elastic beam with a Young's modulus E_{beam}, a second moment of area I_{beam}, and a cross sectional area A_{beam}. For a n storey building these parameters were calculated assuming that the building consists of n+1 slabs with a vertical spacing of 3.4 m. The second moment of area for the equivalent single beam was calculated using the parallel axis theorem assuming the neutral axis is at the mid-height of the building. Axial straining was assumed along each structure's full height to give the axial stiffness. In their parametric study, structures with a wide range of stiffness parameters and geometries were investigated. The stiffness of the building was then related to the stiffness of the soil using relative stiffness parameters which are defined as:

$$\rho^* = \frac{EI}{E_S H^4} \qquad \alpha^* = \frac{EA}{E_S H} \tag{1}$$

where E_S is the secant stiffness of the soil that would be obtained at 0.01% axial strain in a triaxial compression test performed on a sample retrieved from half of the tunnel depth z_0. H is the half width of the structure as shown in Figure 1. The relative bending stiffness is given by ρ^*, while α^* is a measure for the relative axial stiffness. In plane strain analysis α^* is dimensionless while ρ^* has the dimension [1/m].

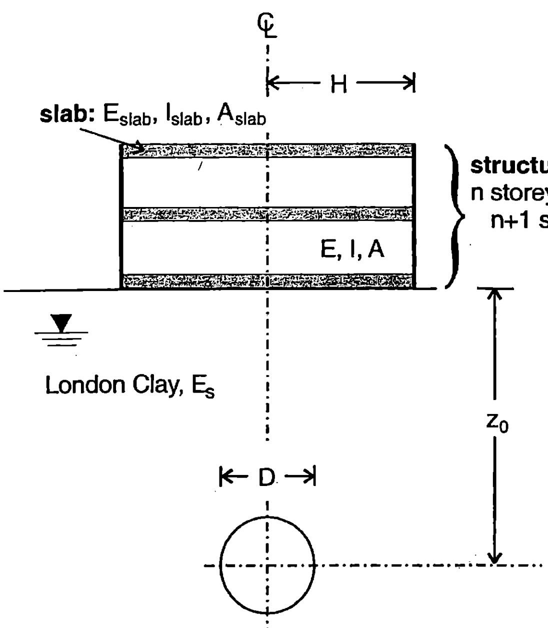

Figure 1. Geometry of the problem.

The parameter ρ^* was related to the deflection ratio of the building and α^* to the horizontal strain. This paper focuses on the deflection ratio DR_{sag} and DR_{hog} for sagging and hogging respectively. Potts and Addenbrooke (1997) related these to greenfield conditions using modification factors:

$$M^{DR_{sag}} = \frac{DR_{sag}}{DR^{g}_{sag}} \qquad M^{DR_{hog}} = \frac{DR_{hog}}{DR^{g}_{hog}} \tag{2}$$

where the index 'g' denotes the corresponding parameter for greenfield conditions. The results of the analyses were presented as plots of modification factor M^{DR} versus relative bending stiffness ρ^*. All the results were within a narrow range in these graphs. Therefore, in order to obtain design curves upper bounds to these data were drawn to provide a conservative estimate of the variation of the modification factors with respect to the structure's relative stiffness.

3 ANALYSIS INCLUDING SELF WEIGHT

The work described above did not consider the weight of the structure. This was justified in order to investigate the influence of the stiffness uncoupled from other factors. However, the stress a structure applies to the soil changes the stress state beneath the foundation significantly and it is arguable that this influences the deformation due to tunnel construction for both the soil and the structure.

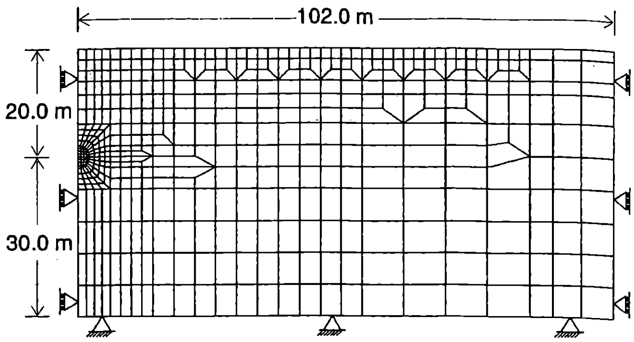

Figure 2. Finite Element mesh for the 20 m deep tunnel.

Therefore, an additional parametric study has been undertaken using the Imperial College Finite Element Programme (ICFEP). The mesh is shown in Figure 2. The soil model which is adopted in this analysis is the same as in the analysis of Potts & Addenbrooke (1997) using the same parameters. The soil profile consists of London Clay which is represented by a nonlinear elastic plastic constitutive model. The model described by Jardine et. al. (1986) is used to model the nonlinear elastic pre-yield behaviour while the yield surface and the plastic potential are described by a Mohr-Coulomb model. The initial stresses in the ground are controlled by the assumed unit weight of 20 kN/m^3 and by the water table at a depth of 2 m. A hydrostatic pore water pressure distribution is applied to the whole profile with a zone of suction in the 2 m above the water table. The initial earth pressure at rest K_0 is set to 1.5.

An uncoupled analysis is used. During the construction of the building the soil is modelled as a drained material. As the pore water pressure is in equilibrium after the load is applied to the structure it is not necessary to simulate any consolidation period. For the tunnel construction the soil model is then set to give undrained behaviour.

Tunnel excavation is modelled by the incremental removal of the solid elements within the tunnel boundary. This is achieved over 15 increments. The data presented are taken from increment 7 (i.e. 47% unloading). This increment is chosen because it gives a volume loss of V_L = 1.5% for a greenfield excavation. It should be noted that a consequence of the decision to present results for 47% unloading in all cases means that the volume loss is not the same for each analysis. The volume loss varied from 1.5 % for greenfield conditions down to 1.3 % for the stiffest structure applied with a high stress.

This paper only focuses on the behaviour of the deflection ratio. In order to concentrate on the influence of the building's weight only one geometry is used: a 100 m long structure with its centreline coinciding with that of the tunnel. The tunnel with a diameter of $D = 4.146$ m was at a depth z_0 of either 20 m or 34 m. A uniform stress of 10 kPa, 30 kPa, 50 kPa and 100 kPa was applied to the structure corresponding to a 1-, 3-, 5-, and 10-storey building respectively. In addition, the zero-weight cases and a structure with a negligible stiffness (zero-stiffness) were included in the study. The stiffness values of the different buildings are calculated as described in the previous section and are listed in Table 1.

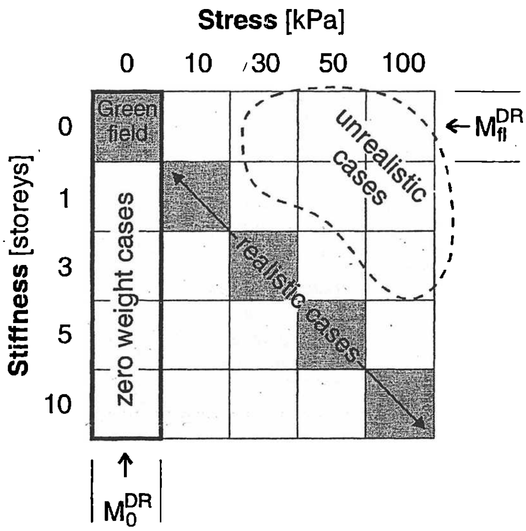

Figure 3. Matrix of stiffness/stress combinations used in the parametric study.

Combining the 5 weight options with the 5 stiffness values gives 25 variations. These are represented in a 5x5 matrix in Figure 3. The matrix contains some unrealistic cases, for instance structures with a low stiffness but loaded with a high stress. On the other hand, the leading diagonal of this matrix represents realistic cases: 10 kPa applied to a 1-storey building, 30 kPa applied to a 3-storey building etc. If basement construction were considered for a given stiffness the net loading would be reduced. This, arguably, could be represented by the cases below the leading diagonal. Combining the zero-stiffness structure with the zero-weight case represents the greenfield case. Considering two different tunnel depths 50 analyses were performed.

Table 1. Stiffness of buildings. A n-storey building consists of n+1 slabs.

Building	Bending stiffness EI [kNm^2/m]	Axial Stiffness EA [kN/m]
Slab	6.47E+03	3.45E+06
1-storey	2.00E+07	6.90E+06
3-storey	2.00E+08	1.38E+07
5-storey	6.98E+08	2.07E+07
10-storey	4.39E+09	3.80E+07

4 RESULTS

For all 50 cases the deflection ratios DR_{hog} for hogging and DR_{sag} for sagging were determined. Dividing these values by the corresponding greenfield case gives the modification factors M^{DR}. In order to investigate the influence of the applied stress the results of each stiffness are normalized against the corresponding zero-weight modification factor $M^{DR}{}_0$ as shown in Figure 3. Figures 4 and 5 show the results of this normalization. In these figures the normalized factors $M^{DR}/M^{DR}{}_0$ are plotted against the stress applied to the structure. Figure 4 shows for each stiffness in the hogging case a steady increase in the modification factor $M^{DR}{}_{hog}$ as the applied stress increases. This effect is small for structures with a low stiffness (0- and 1-storey) and a high stiffness (10-storeys).

It is appropriate to focus on the realistic cases (i.e. those on the leading diagonal). The data for these cases are marked with a thick line and black squares. The biggest increase is 36% for the 5-story building and a tunnel depth $z_0 = 34$ m. For the sagging case shown in Figure 5, the change of modification factor $M^{DR}{}_{sag}$ with stress is smaller (note the different scale between Figures 4 and 5). The increase is below 15%.

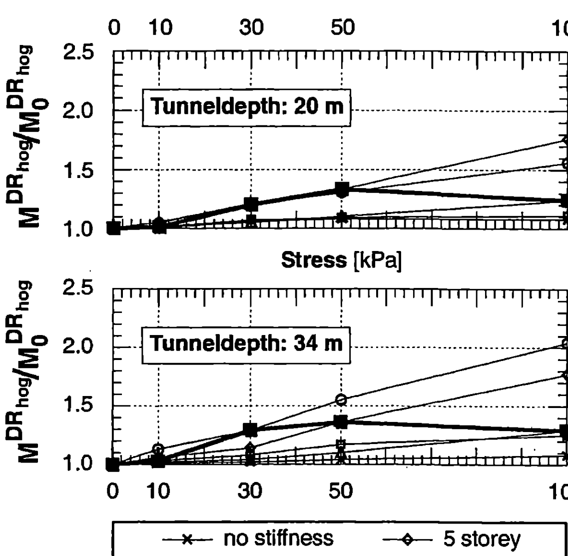

Figure 4. Change of hogging modification factor M^{DRhog} with applied stress. M^{DRhog} normalized against corresponding zero-weight case.

Figures 4 and 5 give no indication about the absolute values of the modification factors. Furthermore the increase for the hogging case seems to be significant. It has however to be considered that the deflection ratio decreases rapidly as building stiffness increases. This can be seen in Figure 6 where the modification factor for each magnitude of applied stress is normalized against the corresponding result of the zero-stiffness case $M^{DR}{}_{fl}$ ('fl' for flexible as shown in Figure 3). These data are plotted against the structure's stiffness. The data for the hogging case of the 34 m deep tunnel are presented. They lie in a very narrow range and show a significant decrease to 0.15-0.25 as the stiffness increases up to the equivalent of 3 storeys. For high stiffness values the results are stable and the deflection ratio is about 10% compared to the corresponding deflection ratio for the flexible zero-stiffness structure.

Combining these results with the flat gradient of the curve for the zero-stiffness case in Figure 4 reveals that despite the increase in $M^{DR}{}_{hog}$ due to weight for the 3- and 5-storey structures in Figure 4, the modification factors for these cases remain very small.

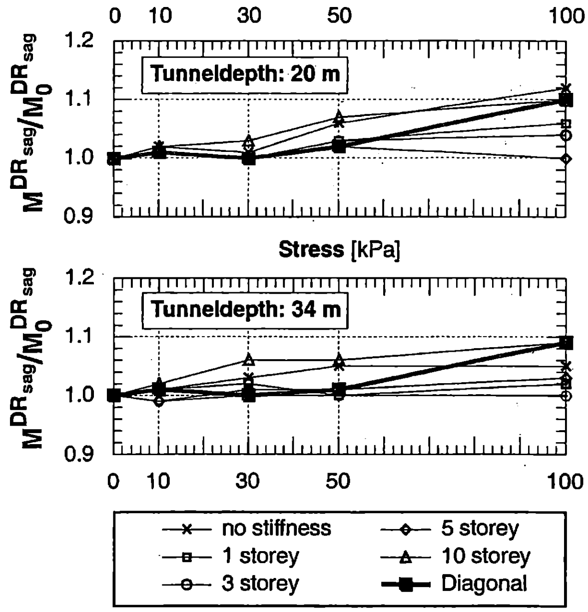

Figure 5. Change of sagging modification factor M^{DRsag} with applied stress. M^{DRsag} normalized against corresponding zero-weight case.

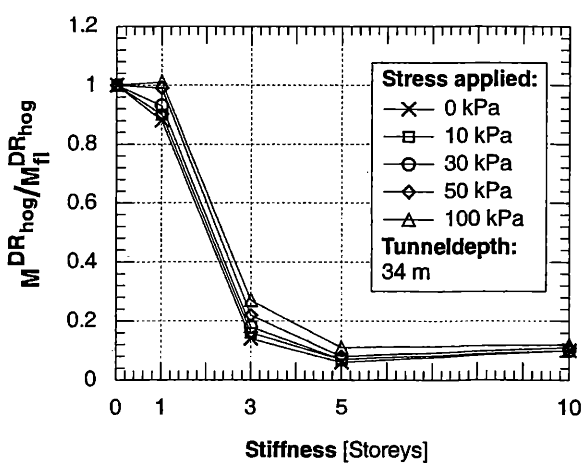

Figure 6. Change of modification factor M^{DRhog} with stiffness. M^{DRhog} normalized against corresponding zero-stiffness case.

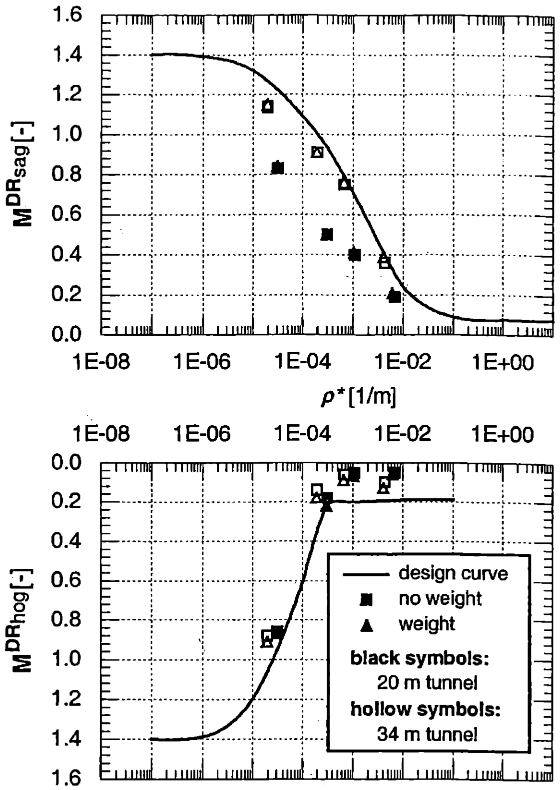

Figure 7. Modification factors M^{DR} compared with the design curves by Potts and Addenbrooke (1997).

This is further illustrated in Figure 7 where the modification factors for hogging and sagging are plotted against the relative bending stiffness ρ^*. For all stiffness values the results for the zero-weight (square symbols) and for the 'realistic' case (on the leading diagonal, triangle symbols) are plotted. The corresponding design curves by Potts and Addenbrooke (1997) which were an upper bound to their design data, are shown for comparison.

The soil stiffness E_S used for describing the relative bending stiffness (Eq. 1) is taken at a depth half way between the tunnel axis and the ground surface. In the analyses of Potts & Addenbrooke (1997) this value was only dependent on the depth of the tunnel. In the data presented in Figure 7 the increase in E_S due to the increased effective stress p' under the building weight was taken into account. The difference in relative stiffness for each data couple in this graph is, however, small.

The data points follow the pattern of the curve given by Potts and Addenbrooke (1997) although the modification factors for sagging of the 20 m deep tunnel lie well below this curve. It can be seen that the modification factors for the zero-weight case and for the corresponding 'realistic' case are close together. Even the increase of modification factor of the 5-storey building applied with 50 kPa, z_0 = 34 m (36% compared to the corresponding zero-weight case) is small when plotted in this context. The increase is from $M^{DR}{}_{hog}$ = 0.064 for the zero-weight case to $M^{DR}{}_{hog}$ = 0.088 for the case considering self weight. Following the design approach by Potts and Addenbrooke (1997) this means for engineering practice that DR^{hog} for a 5-storey building affected by tunnelling induced ground subsidence is only 0.088 times the hogging ratio for the corresponding greenfield situation.

The small change of M^{DR} with applied stress on a structure, and the well defined decrease in M^{DR} with increasing ρ^* show that the structure's stiffness dominates the hogging and sagging deformation behaviour of the structure.

5 CONCLUSIONS

This paper shows the influence of the weight of a structure on its deformation behaviour caused by tunnelling induced ground subsidence.

Using the Finite Element Method, stress of up to 100 kPa was applied to structures with a bending stiffness range from 2.00×10^7 kNm^2/m to 4.39×10^9 kNm^2/m. The soil profile consisted of London Clay.

The results of a parametric study involving 50 nonlinear plane strain FE analyses show that the structure's weight has a small influence on the deflection ratio DR_{hog} for hogging and DR_{sag} for sagging.

It has been shown that in general the deflection ratio increases with increasing weight. This effect is, however, small compared to the decrease of deflection ratio with increasing stiffness. Since the latter effect is dominating, the graph plotting modification factor against relative stiffness shows little change when the self weight is included into the analysis. The results therefore still lie beneath the upperbound curves proposed by Potts and Addenbrooke (1997).

This study only includes the influence of the structure's weight on deflection ratio. The behaviour of the horizontal strain is now being investigated in order to get an overall picture of the influence of the weight.

ACKNOWLEDGEMENT

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

REFERENCES

Boscardin, M.D. and Cording, E.J. (1989): Building response to excavation induced settlement, *Journal of Geotechnical Engineering*, ASCE, Vol. 115, No. 1, 1-21.

Burland, J.B. and Wroth, C.P. (1974): Settlement of buildings and associated damage, *BGS conference 'Settlement of Structures'*, Cambridge, 611-651.

Jardine, R.J., Potts, D.M., Fourie, A.B. and Burland, J.B. (1986): Studies of the influence of non linear stress-strain characteristics in soil-structure interaction, *Geotechnique*, Vol. 36, No. 3, 377-396.

O'Reilly, M.P. and New, B.M. (1982): Settlements above tunnels in the United Kingdom - their magnitude and prediction, *Tunnelling '82*, The Inst. of Mining and Metallurgy, 173-181.

Potts, D.M. and Addenbrooke, T.I. (1997): A structure's influence on tunnelling-induced ground movements, *Proc. Inst. of Civil Engineers, Geotechnical Engineering*, Vol. 125, 109-125.