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Three dimensional analysis of building settlement caused by shaft construction

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ABSTRACT: An analysis of a case history of settlement damage to a masonry building, due to the construction of a nearby shaft, is presented. Three-dimensional finite element analysis is used, in which the non-linear behaviour of both the soil (a heavily overconsolidated clay) and the building is taken into account. Procedures are used to capture the construction process as realistically as possible. Comparisons are made between a variety of analyses (including separate analyses of the building and the soil, and a fully coupled analysis of the whole system), as well as with observations at the site. The principal comparisons are in terms of observed surface settlements and of the categories of cracking damage suffered by the masonry building. It is concluded that the analysis technique can model adequately the interaction between the building and the ground, but that details of settlement and damage patterns depend critically on assumptions about the structure and stiffness of the building and its foundation.

1 INTRODUCTION

Increasing numbers of tunnels are being constructed in urban areas for transportation and utility purposes. It is inevitable that structures of historic and economic importance will be affected. Precautions such as compensation grouting have been applied to reduce the impact on nearby structures, but are expensive and may not always be effective. Current semi-empirical methods of structural damage prediction are often conservative because they neglect the effect of ground/structure interaction in reducing the severity of settlements. There is a need for accurate prediction of the effects of tunnelling which fully considers the interaction.

Any numerical model addressing this issue must take into account a number of aspects. The behaviour of the soil, especially at small strains, is fundamental. Key aspects of the tunnelling process (advance of the heading, volume loss and the lining process), the ground/structure interaction and the behaviour of the structure itself should all be represented. The analysis must be in three dimensions to model the geometrical complexity of a real site and the progress of an advancing tunnel heading. Ground/structure interaction may be modelled by coupling the structure to the ground from the start of the analysis, rather than applying calculated ground displacements in a separate analysis of the structure. Important structures in urban areas are often constructed from masonry, a highly non-linear material, and the stiffness and self-weight of the building should be modelled realistically. Including all these components in the modelling procedure leads to large and complex analyses that will, however, be justified by the risk of damage due to tunnelling in certain key cases.

Research has been in progress at Oxford University since 1993 on the development of a 3-D finite element modelling procedure, including all of the above features. The detailed description of the approach has been the subject of past publications. A kinematic yield hardening constitutive model is used to describe over-consolidated clay (Chow, 1994, Houlsby, 1999). A macroscopic elastic no-tension model is used for the masonry (Liu, 1997). An incremental excavation technique in which soil elements are removed from the mesh, tunnel liner elements are installed and a shrinkage technique is used to model volume loss is described by Augarde (1997). Tie elements are used to couple degrees of freedom between two- dimensional building facades and the soil (Liu, 1997). General conclusions from the project are presented by Augarde *et al.* (1998) and Houlsby *et al.* (1999).

As part of this project, comparisons are being made between this procedure and case histories involving detailed monitoring of ground and

structure movements. The aim is to confirm the practicability and accuracy of the approach.

A case history is presented and analysed here of an excavation in London. It concerns the construction of a shaft and tunnel adjacent to a historic masonry church.

2 SITE DESCRIPTION

Howard Humphreys Consulting Engineers designed and supervised the construction of a tunnel in 1991/92 for a new electricity supply into the West End of London. The project required construction of a 2.5m tunnel, 12m to 25m below ground. Shafts were required along the route for access during construction. One such shaft was in Maddox Street, which is bounded on the North side by a terrace of four storey brick Georgian houses, and on the South side along its entire length by an 18th century stone-clad brick church. The 4m diameter 15m deep shaft was sunk 5m from the North East corner of the church (Fig. 1). No protective measures such as compensation grouting were used, but extensive monitoring of ground and structure movements and observations of damage were carried out.

Since the church was the most important building, it was modelled in the analysis. As the terrace on the North side and the church are on opposite sides of the settlement trough, it seemed reasonable to assume that there is little interaction between the two. The largest movements were predicted during shaft, rather than tunnel, construction, and so that was chosen for analysis.

A number of assumptions were required in the representation of the church. The first was whether it was valid to model the church as a rectangular box of four facades. The church structure consists mainly of masonry walls. There are columns within the church but these support only the roof and balcony, which both being of wooden construction have little stiffness or self weight compared to the masonry walls themselves, which are 1m thick.

There are windows at regular intervals around the church. All these openings could be modelled, but that would lead to rather complex meshes. The main reason for modelling discrete openings is to model crack growth from the corners. In regions of small movement this may be unnecessary, but it is still useful to capture the reduction of stiffness and self-weight due to the opening as it may have implications for the global structure response.

This reduction may be modelled in two ways. In a fully "smeared" approach, an average stiffness for the whole façade is used. A "semi-smeared"

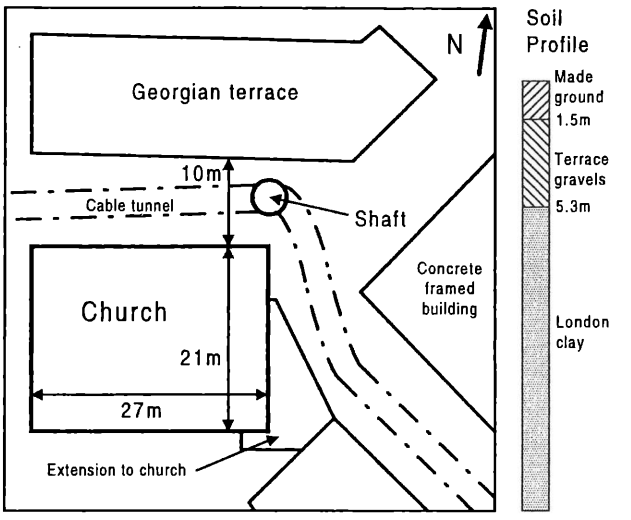


Figure 1. Schematic site location plan

approach recognises that openings aligned vertically may affect the global behaviour by acting as vertical columns of reduced stiffness. This latter approach is similar to that of Simpson (1994) except that his "strata" of reduced stiffness represented rows of openings horizontally rather than columns. This may not model so well the important formation of cracking from the top of the façade, propagating downwards in a hogging region.

In this project, a combination of discrete openings, "smeared" and "semi-smeared" was used. The openings closest to the shaft were modelled in detail, the next as columns of reduced stiffness. The two more distant facades were modelled using a fully smeared stiffness. The masonry was modelled as an elastic, no tension, material with a Young's modulus of 20kN/mm^2.

A borehole at the centre of the shaft showed 1.5m of pavement and made ground, followed by 3.8m of terrace gravels and then London clay to depth, a typical profile for that area of London. Since a foundation depth 3m below ground level was assumed, the response would be dominated by the clay, and so the soil was modelled as undrained London clay throughout. The undrained strength was taken as 120kPa at 3m depth, increasing at 6kPa/m, and the rigidity index was taken as 500.

3 INTERPRETATION OF THE ANALYSIS

Results from four analyses are presented and discussed, as follows:

1. A 3-D non-linear analysis of the shaft construction and the surrounding soil, *i.e.* a "greenfield" calculation without the building.

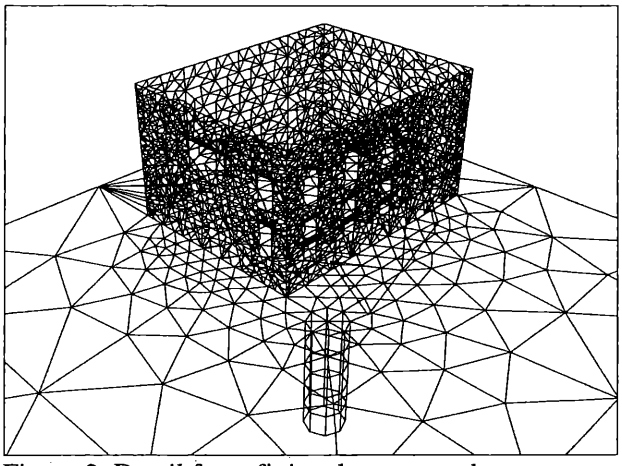


Figure 2. Detail from finite element mesh.

2. Analysis of the building alone, prescribing the displacements calculated from the above "greenfield" analysis.
3. Analysis of the building alone, prescribing the observed field displacements.
4. A full 3-D analysis including shaft, soil and building. A view of part of the finite element mesh for this analysis is given in Figure 2.

The results of the analyses are compared with the observed data. This comparison leads to a reassessment of the methods, and a fifth analysis is then introduced.

The important output from Analysis 1 is the "greenfield" soil surface settlement profile due to the construction of the shaft, particularly around the building footprint. It is a disadvantage that at this site the full settlement contours from the field are not available, due to the number of buildings in the vicinity of the shaft. The buildings other than the church were not modelled in the analysis.

Analysis 4 is of most interest. The soil settlements, especially in the vicinity of the building, would be expected to be different from Analysis 1. In addition, damage to the building is predicted, and comparison of this damage with site observations is a fundamental aim of this project.

The additional analyses are made to assist in gauging whether or not it is beneficial to undertake the full analysis (Analysis 4), or whether a less sophisticated analysis can be shown to give as good results. For example Analyses 1 and 2 model the shaft and soil first, and apply the resulting displacements to the building. Interestingly this approach gives little overall benefit in computing time, but it does have the advantage that, by dividing the problem into two parts, the memory requirements are smaller.

Analysis 3 was carried out for the purposes of verification of the modelling technique for the building. The applicability of the no-tension material model was assessed. Assumptions about the building weight and stiffness, and the way that openings are treated were tested. The application of the site-observed displacements to the building model did indeed give a similar damage distribution to that observed on site (which in this case involved no significant damage, apart from some possible cracking in the basement walls superimposed on pre-existent cracking). This supports the modelling approach for the building, and provides justification for the full model of soil and building.

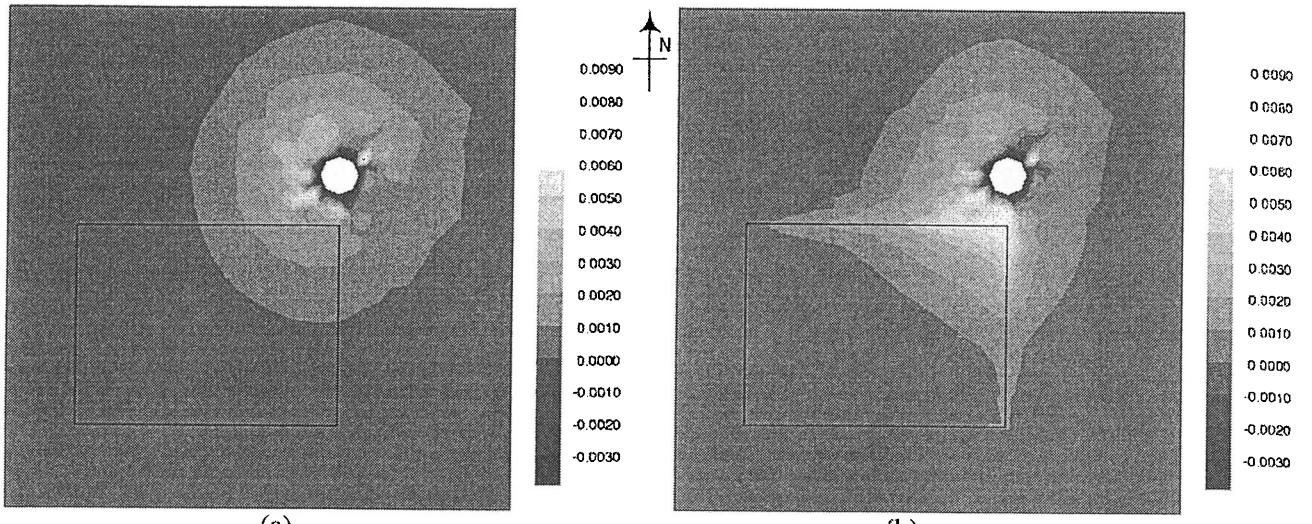
The settlements in the vicinity of the building for the cases with and without the building present are shown in Figures 3(a) and 3(b). It can be seen that, as expected, the presence of the building modifies the shape of the settlement bowl, with the weight of the building appearing to drive settlements further. For example, the settlement at the North East corner of the building nearest the shaft increases from 3mm to 9mm with the building present.

The surface settlements are seen to vary in a somewhat random fashion close to the shaft, with a band of about 1m width around the shaft where heave of up to 4mm occurs. It is considered that these fluctuations are mainly due to numerical problems occurring when applying the ground loss to the shaft, and that they do not extend out far enough to affect the building significantly.

More detail on the settlement profile affecting the building may be obtained by plotting settlement with distance along the facades, and this is presented in Figures 4(a) and 4(b), for the East and North facades, which are those closest to the shaft. The South and West facades did not settle significantly, or experience any measurable damage in any of the analyses or in the field.

The profiles for the observed site data indicate that the building tilted almost as a rigid body towards the North, with a fairly constant settlement of 6–8mm along the whole of the North facade. Some of this settlement may have resulted from the construction of the tunnel, which was not modelled in the analysis, but may estimated as up to 3mm of the settlement according to current design approaches.

The results from the full analysis (Analysis 4) show the bases of both facades following half-troughs similar to a Gaussian shape, but with a somewhat pronounced point of maximum hogging. The maximum settlement of 9mm calculated at the North-East corner is just 2mm greater than that observed. The calculated "greenfield" settlements without the building also follow Gaussian-type half-



(a) (b)

Figure 3. Contours of surface settlements in metres (a) from Analysis 1 and (b) from Analysis 4.

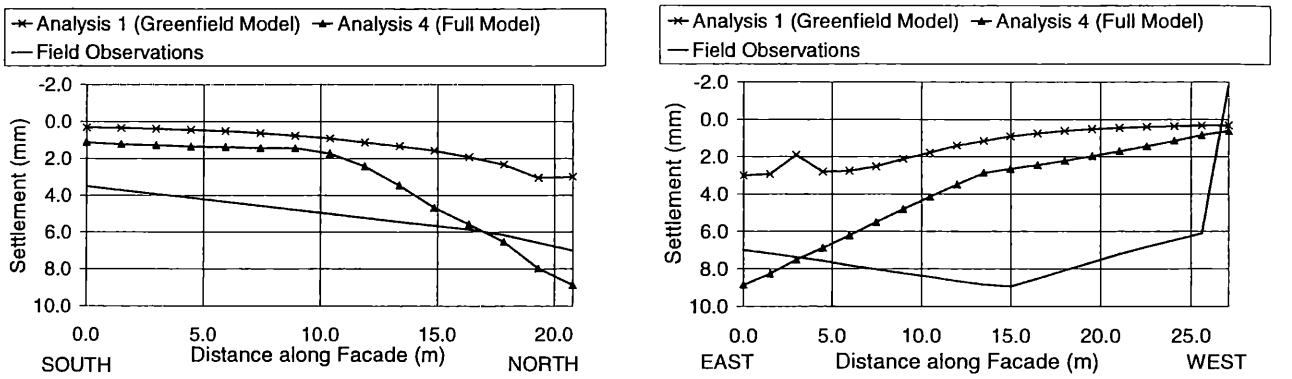


Figure 4. Observed and calculated settlements, (a) East facade, (b) North facade.

troughs, a bit smoother in general but with a reduced maximum settlement of only 3mm at the corner, and with signs of localised irregular behaviour (probably due to mesh discretisation) near the corner as noted on the surface settlement plots.

Both analyses used a value of ground loss due to the excavation of the shaft of 2%, which corresponds to the physical dimension of the overbreak due to the cutting edge on the caisson used to sink the shaft. There may have been other effects, for example heave of the soil into the base of the shaft, which could increase the ground loss above 2%. As already mentioned, it was not possible to measure the settlements over the entire area affected by the shaft and hence calculate the real ground loss. A figure of 2%, however, is not unreasonable.

Plots of damage category for the North and East facades of the building for Analyses 2 and 4 are presented in Figures 5 (for Analysis 4) and Figure 6 (for Analysis 2). The damage category is based on the value of cracking strain calculated in the no-tension masonry material model, in accordance with Boscardin & Cording (1989), using the classification of visible damage in masonry first presented by Burland, Broms & De Mello (1977). The facade has been divided into regions and the damage category based on the average cracking strain over all the Gauss points within the region. This may result in some loss of information on some localised cracking behaviour, but makes it easier in the first instance to compare the results of different analyses quantitatively (Liu, 1997). Some judgement is required on the division of the facades into regions. These were chosen to highlight regions adjacent to openings and areas of different stiffness. Reduced region sizes were used where more detail was required, *e.g.* in the North East corner of the building.

The distribution of the damage, in particular the position of the "slight" damage, is quite distinct for each run, and these will be discussed in turn. "Slight" damage implies typical crack widths of 1mm to 5mm. No specific new cracking of this magnitude was observed on site. The structure was, however, known to be cracked, due possibly to earlier nearby tunnel construction. This cracking particularly affected the East facade and the basement level. Thus the predictions cannot be

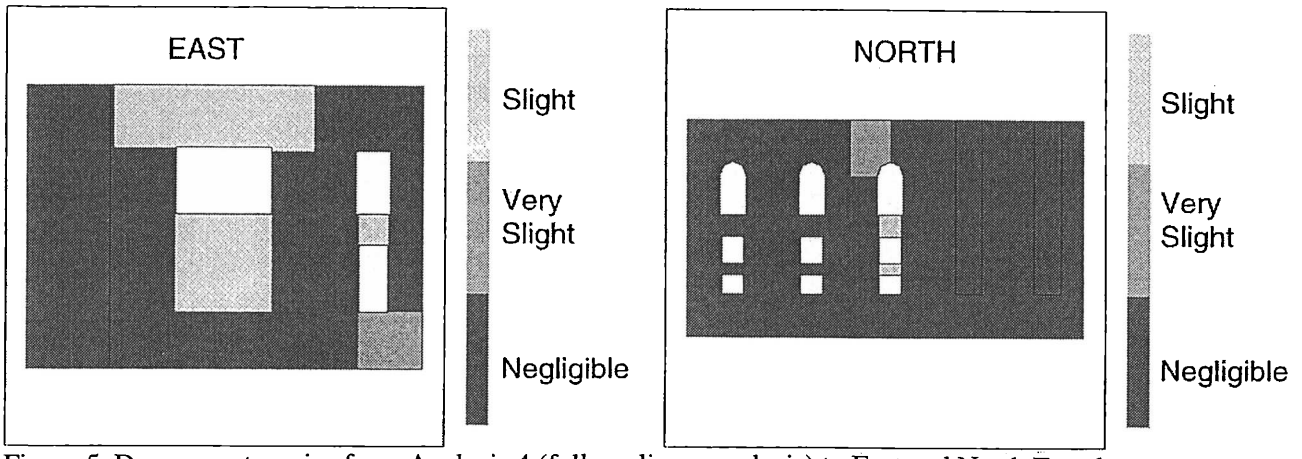


Figure 5. Damage categories from Analysis 4 (full nonlinear analysis) to East and North Facades.

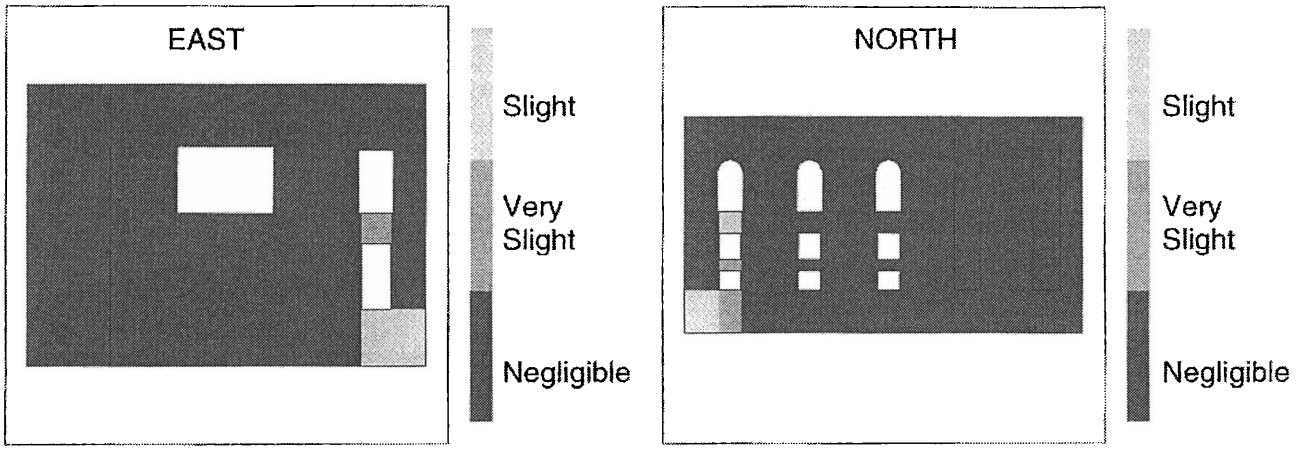


Figure 6. Damage categories from Analysis 2 (application of greenfield settlements to building) to East and North Facades.

regarded as definitely proved accurate, but are entirely consistent with the observations in suggesting "slight" damage. They certainly do not under-predict the real damage.

Figure 5 shows the results for the full non-linear analysis. This predicts the largest amount of damage on the East facade, with "slight" damage around the larger opening, and further damage associated with the North East corner. The North facade experienced damage concentrated in one column of openings. It can be seen by comparison with Figures 4(a) and 4(b) that for both facades these areas of damage near openings occur in the hogging part of the settlement profile, which was quite pronounced for both facades. It appears that some interaction occurs. The hogging region of the settlement bowl lines up with an opening or openings; this causes damage of that part of the facade, thus reducing its stiffness, thereby magnifying the hogging profile, producing a slight "hinge" and saving the remainder of the facade from damage. The position of such hinges would be hard to predict without a 3-D analysis.

The results for Analysis 2 (Figures 6(a) and 6(b)) show damage in the "very slight" and "slight" categories concentrated at the corners of both facades nearest the shaft. This is consistent with the "greenfield" settlements applied, which were fairly small, and slightly irregular near the corner.

Analysis 3 (in which the observed settlements are imposed on the building) is not presented graphically here, but shows no damage greater than "negligible" on the East facade, and on the North facade "slight" damage aligned with the second column of openings. This low damage is consistent with the building tilting as a rigid body towards the North, with almost straight settlement profiles along both facades. The fact that "slight" damage was still registered between openings shows the important effect of the presence of openings on the behaviour of masonry facades, causing stress concentrations and sections of reduced shear strength and stiffness.

The fact that the building tilted almost as a rigid body is thought to be due to the foundation contributing more to the stiffness of the whole system than had been taken into account in the analyses. To test this hypothesis a further calculation, Analysis 5, was made in which a stiff foundation was added. The settlement contours from

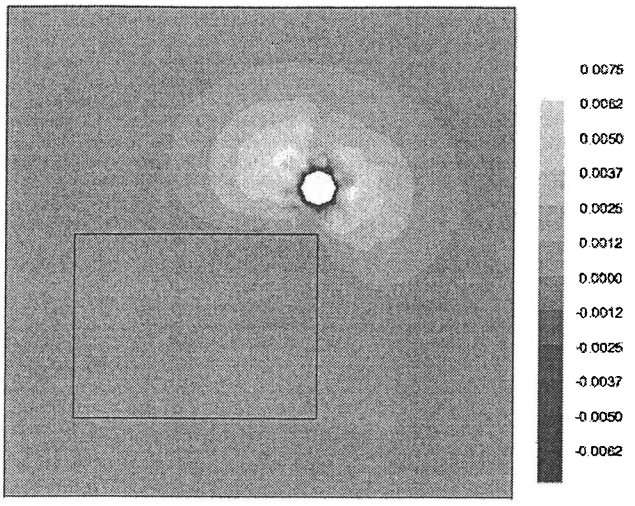


Figure 7. Settlements (m) from Analysis 5.

this analysis are shown in Figure 7. Comparison with Figure 3(b) reveals how important the modelling of the foundation stiffness may be to the accurate prediction of settlement of buildings and of the associated damage.

4 CONCLUSIONS

The results obtained from Maddox Street appear to be reasonable in terms of the damage category of "slight" predicted for the building, which is consistent with the observed result, perhaps a little conservative, but not overly so. There is, however, scope for refinement of the assumptions made in the modelling process, and improvements can be made in future analyses.

The maximum calculated building settlement was similar to that observed on site, although the distribution around the building was different. The real building behaved more as a rigid body, indicating that its global stiffness was higher than in the model. This may be due to tensile strength in the masonry and, more importantly, a basement structure consisting of a vaulted roof supported on columns. Because of its complexity, this was not modelled, except approximately in Analysis 5.

Interaction phenomena between the building and the ground were observed. Cracking of the building in hogging regions reduces the effective stiffness, thus causing the cracking to localise. The importance of the building weight in increasing the settlements compared to the "greenfield" case was seen, as was noted by Liu (1997) in 2-D analyses. The situation would, however, have been quite different if the building had spanned across the whole settlement trough, so enabling arching action to take place. Thus the importance of the particular geometry of the problem is obvious.

The importance of representing openings to model the facade behaviour was shown. Openings were either modelled as holes or as vertical regions of reduced stiffness. The former is appropriate where detail is needed, but the latter seems to be a satisfactory approach for less critical zones.

5 ACKNOWLEDGEMENTS

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