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Analysis of Ground activities and quantity loss due to an advancing tunnel direction

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Abstract— This paper made use of large-scale non-linear finite element analysis. The demand on computing resources was high, stimulating many enhancements to the software, the most important of which was parallelization of the analysis program for use on the Oxford Supercomputer. To obtain optimum results, larger model sizes are required. The computing resources to enable this should become more commonly available within the next few years, enabling the modeling techniques to be used routinely. The overall conclusion of the project is that the modeling procedures are suitable for application to the detailed assessment of the response of buildings to tunnelling. Particular features of the procedures are that the building is modeled together with the ground and a representation of the tunnel excavation, and in three dimensions. It has been confirmed that all these features are necessary to model the building response, which may include a combination of shear deformation, arching and bending behaviour. Further lessons have been learned concerning the importance of the self-weight of the building in determining overall settlements, how to model openings such as doors and windows in façades, and whether it is necessary to model the building foundation. It has not proved possible, through lack of time, to model the advance of tunnels beneath buildings within this thesis. This, however, is observed to be an important effect in the field, particularly in causing damage to internal walls. It is recommended that further research be carried out in this area.

Index Terms— modeling, building foundation, Supercomputer, tunnel excavation, computing.

I. INTRODUCTION

This project planned to use numerical modeling to investigate the response of buildings to tunnelling. It aimed to explore similar effects to those considered in a stage three assessment as numerical model with this capability would be verified against case history information. In doing so, further understanding of the important parameters for a detailed evaluation would be sought. The aim at the start was not necessarily to produce a tool to carry out a stage three assessment, such as a software package or a set of design charts, although these could be a by-product of the work, or the product of work to follow. Rather, the intention was to investigate the feasibility of the option of using numerical modeling for detailed evaluation and identify the resources that were likely to be required, both in human and computing terms. It is recognized that there are other approaches distinct from numerical modeling that could be taken to explore this area, for example gathering case history data for an empirical study, small-scale laboratory testing or centrifuge modeling. However, the aim of this project was to exploit specific advantages of three-dimensional non-linear numerical analysis for this purpose. These were the ability to model any geometry without having to simplify to two dimensions, representation of the advance of a tunnel heading, and the ability to vary the parameters of the constitutive models of soil and masonry to suit those of case history examples, and to carry out parametric studies. The basic elements of the numerical modeling procedure to be used in this project were implemented in the first phase of a research programme at the University of Oxford, commencing in 1993. The approach uses a non-linear finite element method in three dimensions, modeling a building coupled to the ground and the incremental excavation of a tunnel heading. A key aspect of the work at Oxford is to model the building together with the ground and tunnel, in a combined or "fully coupled" analysis. The first phase is described in detail in the theses of Chow (1994), Augarde (1997) and Liu (1997), and summarized in the paper by Burd *et al*. (2000). This project forms part of a second phase of research and development, which looks more towards the industrial application of the methods developed in the first phase. The planned activities for this project were as follows: Firstly, case history data of building responses to tunnel excavation would be obtained. Approximately three to four well-documented sites, involving tunnelling in soft ground under or near masonry buildings but without any intervention measures such as compensation grouting, were sought. At the outset, the Mansion House case history, the subject of three published papers in 1994, was considered a good example. Three-dimensional finite element models of the sites would then be assembled and analyzed. The results would be compared with the field data and reasons for any differences would be explored. Experience gained from each analysis would be applied to the next, any useful enhancements to the analysis software made, and the performance of the existing software and hardware examined. A base of experience of carrying out three-dimensional analyses, and overcoming practical difficulties in the process, would be obtained.

The incorporation of protective measures that may be used on site into the analyses, such as compensation grouting, was deemed to be beyond the scope of this project. A parallel research project at Oxford was given the brief of exploring this area. Although case history data from any site could be considered, there was a preference for sites on over-consolidated clays similar to London clay, and for masonry structures.

The time allowed for the research was three years commencing in October 1996. The project sponsors were the candidate's employer, Howard Humphreys and Partners Ltd., and the Royal Commission for the Exhibition of 1851. Howard Humphreys, as well as allowing for the candidate to be engaged full time on the project for its duration, were also to have an important role in assisting in the obtaining of case history data, from its own projects both past and present and from outside sources.

Ground movements and volume loss due to an advancing tunnel heading

Ground movements are an inevitable consequence of constructing a tunnel in soft ground. It is not possible to create a void instantaneously and provide an infinitely stiff lining to fill it exactly. In the time taken to excavate, the ground around the tunnel is able to displace inwards as the stress relief is taking place. Thus it will always be necessary to remove a larger volume of ground than the volume of the finished void. This extra volume excavated is termed the 'volume losses. The lining, which is of slightly smaller diameter than the shield, is erected immediately behind it. The annulus between the lining and ground is normally filled with grout one or two rings back. Thus there is a further opportunity for the ground to displace radically onto the lining, until the grout has hardened sufficiently to resist the earth pressures. The sum of the two radial displacements is termed 'radial' ground loss. The face loss and radial loss total to give the overall volume loss, V_L, for the construction of the tunnel, measured in m^3 per meter advance of the tunnel drive.

It is hard to make general assertions about the relative magnitudes of the face and radial components of the volume loss, as there is limited case history data of sufficient detail. Macklin and Field (1999) reported an investigation of movements around an advancing 2.8m diameter tunnel in London clay. They were able to relate changes in lateral earth pressure and surface settlement to the position of the advancing tunnel heading. In that case, about 70% of the surface settlement at a particular section took place after the tail of the shield had passed, during lining installation and grouting.

II. Literature Survey

It is much harder to control the radial loss. One option is to inject slurry into the annulus around the shield. However because the shield is moving forward, and because the erection of the lining must necessarily be carried out at atmospheric pressure, prevention or even control of radial movements is not really practicable. It can be the most significant cause of settlement, especially if grouting of the annulus is not carried out immediately (*e.g*. Quinn, 1990). The best approach is to keep a steady fast rate of advance, and rely on dealing with the problem by treating its effects (surface settlements) rather than at source. The use of compressed air to maintain pressure in the tunnel during the entire excavation and lining cycle has safety implications and is usually too expensive. Prediction of the total amount of volume loss would be useful for tunnel designers, but is difficult because volume loss apparently depends on a number of factors that are not known at the design stage. These include the tunneling machine type, the construction sequence and the effectiveness of the grouting behind the lining, the latter being a 'Workmanship' factor. The designer ideally knows the soil properties and *in situ* stress state. It is also known from observation (*e.g.* McCaul *et al*., 1986) that volume loss does not necessarily increase with stress (or depth).

Macklin (1999), after earlier work by Mair *et al*. (1981) and O'Reilly (1988), combined case history and centrifuge modeling results to propose a prediction method for volume loss in over consolidated clays. His method used the concept of stability number, explained in Section 2.3.3. The data fell between fairly well defined upper and lower bounds. The spread either side of the median design line is likely to be due to a combination of variability in soil behavior and of workmanship factors, as conceptualized by Lee and Rowe (1991) with the inclusion of a workmanship term in the calculation of their 'gap' parameter. These factors cause variability, because their effect depends on the rate at which soil responds, relative to the time taken for the event to occur that triggers the same soil response. Investigation of these rate effects is beyond the immediate scope of this study, but would make an interesting topic for further consideration. The above discussion is relevant in tunneling where the void is excavated and then as tiff segmental lining is erected. Tunnels may also be constructed by a process of excavation followed by application of a sprayed concrete primary lining. Conceptually there will still be both face loss, as before, and radial loss (due to deflection of the primary lining during and after curing). Bowers *et al*. (1996) report on ground movements due to the Heathrow Express Trial Tunnel. The focus of this project has not been on tunnels with sprayed concrete linings, although there is no reason in principle why they could not be modeled.

Short, medium and long term ground movements

Time dependency in the mechanical behavior of soil influences ground movements resulting from tunneling, leading to the classification of short, medium and long-term movements. Each of these regimes of behavior is itself a steady-state response of the ground to changes in internal or external conditions. The true transient behavior – the part of the response in each regime that is dependent on the initial conditions – is beyond the scope of this and other studies of tunneling-induced ground behavior, although it might be an interesting subject for further study.

Short-term ground movements are identified to occur during at most the first four days after excavation in London clay. This is a timescale that is shorter than, or comparable with, the time taken by the advance of the tunnel heading that is the cause of ground movements. Macklin and Field (1999) reported short-term settlements taking place at a section over a period of 24 hours before and after the passage of the shield. There is evidence that the movements start and stop almost instantaneously with advance of the tunnel heading. The response of the ground is at constant volume to the suddenly imposed new stress regime, which is essentially one of unloading at the tunnel boundary. Rowe and Lee (1989) proposed that it was the extension modulus in clays that was particularly relevant to tunneling problems. The response may be entirely elastic or include irrecoverable strains. Apart from very locally to the tunnel, strains are small (say less than 0.1%) and not enough to cause failure or alter the structure of the soil significantly. Thus the constitutive model for the soil can be assumed unchanged during this phase.

Medium and long-term settlements are thought to be the result of creep, ageing and consolidation (Mitchell *et al*., 1997), *i.e.* alterations in the properties of the soil at constant load. The timescale over which they occur depends on the ground conditions, ranging from weeks or months for sands and soft clays two years for stiff clays. The relative magnitude of short and long-term movements depends on many factors and it is hard to generalize. Case histories suggest that for a typical site on stiff clay, around 60% of the total settlement occurs in the short term (Simons and Som, 1970; Morton and Au, 1975). Tazewell and Selby (1989) observed long-term settlements up to2.5 times the short term, but also that the long-term trough widths tended to be wider. This meant that the curvature of the trough, the factor most likely to cause damage to structures, was similar to that in the short term. In addition, surface structures are more able to accommodate long-term settlements by creep and stress redistribution. Thus it is the short-term movements that remain the chief issue of concern for engineers.

The prediction of ground movements due to tunneling

A more detailed understanding of the mechanisms by which ground movements occur at tunnel excavations could be beneficial in predicting the volume loss, deducing the mechanisms of interaction with surface structures, and designing countermeasures against settlement. Empirical methods based on case history data, analytical methods (upper and lower bound and closed-form), laboratory 1*g* model testing, centrifuge modeling and numerical analysis have all been employed. For the potential value of numerical analysis to be realized, a sufficiently accurate constitutive model for the soil in the appropriate stress/strain range is required. Any modeling technique, either laboratory or numerical, must represent the tunneling process to an acceptable degree of accuracy.

Empirical 'Greenfield' settlement troughs

In 1969, Ralph Peck described settlement data from over twenty case histories available to him at that time, and was able to deduce that the short-term transverse settlement trough in the 'Greenfield' could be approximated by a normal distribution or Gaussian curve. The value of the trough width parameter *i*, the distance from the axis to the point of inflexion of the trough determines the maximum settlement for a given volume loss. Peck noticed that soils of different classes, for example cohesion less or cohesive, gave distinct ratios of trough width parameter to tunnel depth.

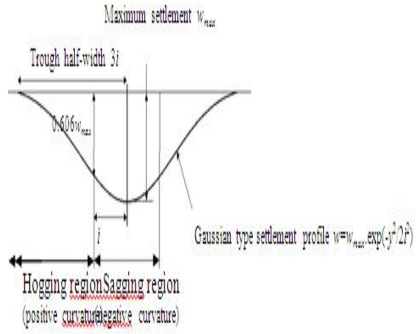

Figure 1: The Gaussian model of transverse tunnelling-induced settlements

The fit to the available data was found to be better for cohesive soils. The data used covered a wide range of tunnel axis depths, from 3.4m to over 34m. It thus appeared justified to take *K* as a constant value, independent of both tunnel depth and diameter. Later work by other researchers has confirmed that for cohesive soils *K* is usually in the range 0.4 to 0.5, and for cohesion less soils in the range 0.25 to 0.35.Attewell and Woodman (1982) extended this model to derive a settlement trough in the longitudinal direction using a cumulative Gaussian distribution. It is assumed that exactly half the total settlement has occurred at the position of the heading and that the longitudinal trough parameter is equal to the trough width parameter. The latter assumption is somewhat conservative, as observed longitudinal troughs show a flatter distribution.

Mair *et al*. (1993) extended the tools available to the engineer for empirical assessment based on the Gaussian model by using case history data to derive formulae for subsurface settlements due to tunneling in clays. A method for prediction of horizontal ground surface movements (and hence strains)was proposed by O'Reilly and New (1982). There was acknowledged to be relatively Littlefield data for corroboration, but centrifuge testing (*e.g.* Mair, 1979) suggested that the

Vectors of ground movements above a tunnel in stiff clay generally converged on a point somewhere between the tunnel axis and the tunnel invert. O'Reilly and New approximated the 'sink' thus formed to be at the tunnel axis.

Mechanisms of short-term settlement response

Davis *et al*. (1980) presented lower and upper bound analytical solutions for the collapse load of a shallow tunnel with support pressure in a cohesive soil. They considered the cases of a plane strain unlined circular tunnel (radial ground movements), a plane strain heading (face movements) and a circular tunnel heading (the full three-dimensional case).The difference between the lower and upper bound collapse loads was greatest (with almost a factor of two between them) for the three-dimensional case, indicating the difficulty in applying analytical solutions in three dimensions. They plotted their analytical solutions in terms of N_{TC} against the cover to diameter ratio (*C*/*D*). Data from centrifuge Modeling (Mair, 1979) confirmed the solutions for a plane strain unlined circular heading, with best agreement for *C*/*D* less than 3.

Laboratory and centrifuge testing

Laboratory 1*g* testing can perform a useful role to identify the main influences in a problem such as tunneling. Kim *et al*. (1996) report tests on the interaction between closely spaced tunnels, finding that significant bending moments could be induced in a tunnel lining due to excavation of another tunnel within two diameters.

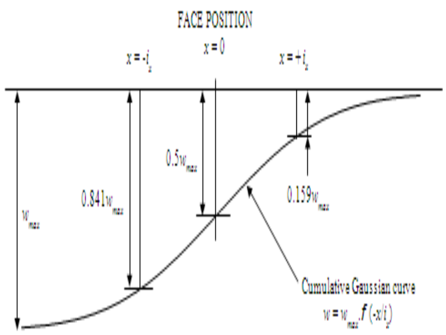

Figure 2: Empirical longitudinal settlement trough based on Gaussian model

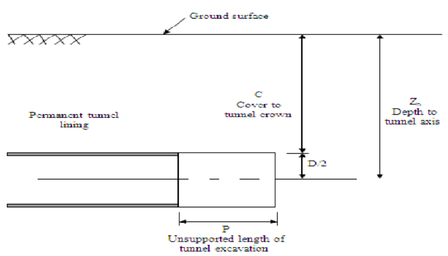

Figure 3: Geometric parameters of a tunnel heading for calculation of stability number at collapse

De Moor and Taylor (1989) carried out model tests using a 250mm diameter tri axial test apparatus with The tunnel represented as a hollow open-ended brass cylinder aligned in the axial direction. Nakai *et al.* (2000) report tests on tunneling in sand, where the finished tunnel is modeled by a solid cylinder and the volume loss due to the advance of the tunnel by removal of 4mm thick rods arranged around the annulus of the cylinder.

Centrifuge modeling is used to reproduce the *in situ* stress state, and hence soil properties, more accurately. Mair (1979) used the centrifuge to model the collapse of a shallow unlined tunnel in clay. Recent work has used the centrifuge to investigate the influence of soil nailing on settlements above tunnels (Kuwano *et al*., 2000), spile reinforcement of the face (Calvello and Taylor, 2000) and settlements in two-layer soil sites (Grant and Taylor, 1996). Grant and Taylor (2000) used centrifuge modeling in an attempt to verify the analytical solutions of Mair and Taylor (1993), with reasonable agreement. Areas of disagreement were related to the finite depth of the tunnel in the centrifuge, compared to the assumption of an ax symmetric stress state in the analysis.

Researchers conducting model tests in the laboratory or centrifuge are inevitably limited by cost and time as to the number of experiments that may be carried out, and thus the amount and quality of data obtained. Some centrifuge experiments require a long time for sample preparation, and it is difficult to model realistically the tunneling process and the advance of a tunnel heading in a centrifuge.

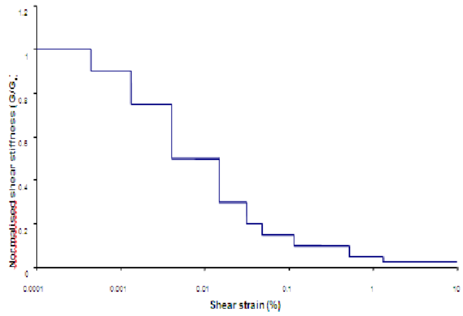

Figure 4: Variation of tangent shear stiffness with shear strain for nine-surface kinematic hardening model

However, previous researchers have made simplifying assumptions about the nature of the combined problem to be able to continue applying methods such as empirical observation, centrifuge modeling and numerical modeling in similar ways as for assessment of the 'Greenfield' case. The main such simplifying assumption has been to consider the problem of a tunnel passing under a building as two-dimensional. Secondary assumptions made by various researchers are that:

1. The surface structure does not interact with the tunneling-induced ground movements but rather deforms according to the movements that would be expected in the' Greenfield' case.
2. The possibility of the structure becoming damaged, altering its stiffness and response to the ground is not allowed for.
3. As an alternative to assumption 1, interaction between the structure and tunneling-induced ground movements is allowed for, but the resulting deformation is limited to being expressed as a

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modification of the movements that would be expected in the' Greenfield' case. This does not allow for the presence of the structure causing a fundamentally different mechanism of behavior.

III. EXPERIMENTAL APPARATUS AND MATERIALS REQUIRED

In order to progress the understanding of building interaction with tunneling-induced settlements, and address many of the shortcomings of current design methods for this problem, a programme of research has been undertaken in the Civil Engineering Research Group at Oxford University since 1993. The first phase, up to the commencement of this project, concentrated on developing the required tools in three-dimensional finite element analysis of buildings and tunnels. As well as working in 3-D, the main aims were to achieve realistic representations of a building, ground and tunnel excavation in one combined model, including appropriate constitutive models for stiff clay and masonry, and numerical procedures for modeling excavation, tunnel lining installation and volume loss. In the first phase, a number of 2-D analyses of a building and ground were undertaken and reported on by Liu (1997), as a precursor to a small number of full 3-D demonstration analyses (Burd *et al.*, 2000).

Phase one two-dimensional analyses

Liu (1997) carried out finite element analyses applying the Gaussian settlement trough to a model of a building, represented in 2-D as either a plain wall or a typical façade with openings. The constitutive models used for the building were linear elastic or elastic, no-tension, the latter having been implemented by Liu into the finite element program OXFEM .Cracking is represented by altering the stiffness properties locally by reducing the tangent stiffness to near zero in a principal direction once the principal stress becomes tensile, without modeling individual cracks. Cracking may occur in either one or both principal directions. The cracks may also re- close, restoring full elastic properties, when one or both of the cracking strains becomes zero, in which case the strain parallel to the crack direction is retained as a non-recoverable residual strain. The average strain at integration points in a region of the mesh after cracking is a measure of the size and spacing of cracking in the structure, which may be correlated to the damage category using the approach of Burland *et al.* (1977).

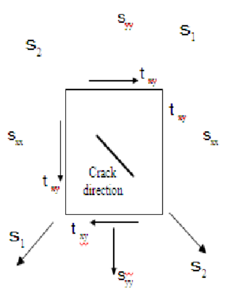

Figure 5: Principal stress and crack directions in masonry constitutive model

This model for masonry has the advantage of being relatively simple to implement, with only six parameters in the input:

1. Young's modulus in compression times wall thickness (*Et*)
2. Poisson's ratio (ν)
3. Residual tensile strength (*c*)
4. Stiffness reduction ratio (ε^{cr}) (rate of reduction of tensile stiffness with strain)
5. Ratio between residual tensile and original Young's modulus (f_1)
6. Ratio between residual tensile and original shear modulus (f_2).

The effect of altering parameters in the kinematic hardening soil model was then explored. In the case of the tunneling problem, it was conceivable that the surface trough width could be narrowed by flattening the 'S' curve, increasing the range of shear strain at which high stiffness is maintained, followed by a more abrupt reduction in stiffness at high shear strain. This might have the effect of reducing the extent of yielding above the tunnel, limiting the strain to a small part of the model that had yielded fully.

IV. EXPERIMENTAL PROGRAME AND RESULTS

This chapter is concerned primarily with the practicalities of carrying out three- dimensional non-linear finite element analysis, the difficulties encountered and the lessons learnt. The process is described as consisting of pre-processing, analysis and post-processing phases. The performance of the hardware used on the project is discussed, in particular the benefit of parallel processing on the Oxford Supercomputer. At the end the principal lessons learnt are summarized. The number of degrees of freedom of the system is the multiple of the number of nodes and the number of displacement freedoms at each node, which may be translational or rotational and which may vary between different regions of a mesh. The number of degrees of freedom defines the rank of the matrix $[K]$ and determines the computing resources required in solving the matrix equation given above. The move from two-dimensional to three-dimensional analysis does not change the basic nature of the calculation, only its size. In general, finite element meshes are stiffer than the theoretical continuum mechanics solution. As the finite element mesh is refined, the solution should approach that of the theory.

In many practical problems the material constitutive behavior is such that the [D] matrix is not independent of strain, giving rise to a nonlinear problem. Most civil engineering materials, such as soil, rock, masonry and concrete exhibit this phenomenon. Materials range from 'weakly' non-linear, in which the variation of stiffness with strain is continuous, to 'strongly' non-linear, in which abrupt changes in stiffness may occur; for example in the cracking behavior of concrete or masonry.

Pre-processing techniques

In the pre-processing stage in finite element analysis, the physical problem is idealized and described to the computer in terms of its geometry, element topology, material properties, external loads and excavation sequence. The first stage in pre-processing is to examine the site under consideration and determine the geometry, ground and structure material properties and excavation sequence for the tunnel. It was found helpful to draw up a set of hand sketches of the geometry of the tunnel and building to form the input to the computer model. Important decisions are required concerning the level of detail at which to model the site, in particular the building, and an advance in understanding of this issue was sought during this project. The design of the model should also cater for the information required from it. I t has highlighted the following as necessary elements for any pre-processing software:

1. Creation of problem geometry in three dimensions. The typical level of complexity achieved in this project was to produce a block of ground with a single straight tunnel aligned along a principal axis of the block. The tunnel itself was usually sub-divided into a number of excavation stages. Each of the façades of a typical building, which usually included openings and other features, was modeled in this study as a separate entity, although the building could conceivably be modeled as one.

2. Mesh generation. Meshes are created on the required surfaces and volumes. For efficient use of computing resources during the analysis stage, unstructured meshes are desirable which cannot be meshed by hand and which require a more sophisticated mesh generator than for regular meshes. It is highly desirable to have control over the local element size at any location in the model, to overcome problems in fitting the mesh to the geometry and obtaining solution convergence, and to keep the overall mesh size within limits.

3. Application of boundary conditions. Boundary conditions may be applied to single nodes, lines of nodes along an edge or to a whole surface. In this study, only restraints of translational degrees of freedom were required.

4. Partitioning of the mesh and the facility to specify different element types or material properties in different parts of the mesh.

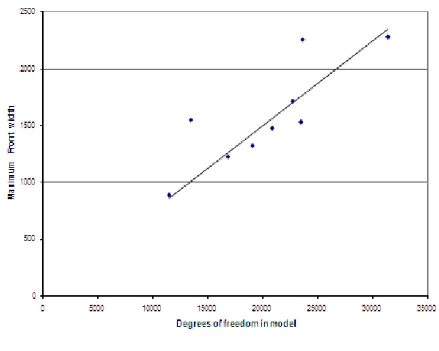

Figure 6: Increase of maximum front width with model size

A probable explanation for the settlement observations lies in the tunneling method and the mechanisms by which the volume loss is occurring. The primary lining is actually cast against the ground ahead of the face, so there is no equivalent of the annular void around a shield, or tail void behind as in conventional bored tunneling. Thus the remaining possible sources of apparent radial volume loss are (i) over break in cutting the pre-vault slot that is not filled with sprayed concrete, and (ii) deflection of the primary lining after excavation. Significant over break was known to have occurred, particularly in the structure less chalk at the start of the drive. However, compaction grouting was used to control movement of the ground. There is no evidence that, even when over break did occur, the whole slot was not completely filled with sprayed concrete. Monitoring of in-tunnel deflections of the primary lining indicated crown settlements of about 12mm (Morgan, 1999), and convergence of up to 8mm. It would be expected that this movement be transmitted to the surface as a form of radial ground loss.

The ground conditions at the face will also influence the volume loss mechanisms. Much of the drive was entirely within fairly competent structured chalk. This would give as table bore with little over break, face loss or heave. Therefore, in the deeper sections, volume loss was likely to be caused exclusively by crown deflection, giving the relatively narrow troughs observed. At the start of the drive and at the cottages, where the cover was low and the ground conditions poorer, the other mechanisms discussed above may have contributed to the volume loss.

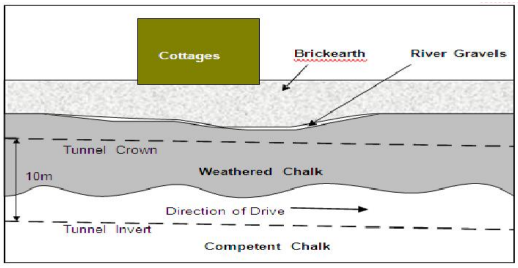

Figure 7: Cross-section showing ground conditions

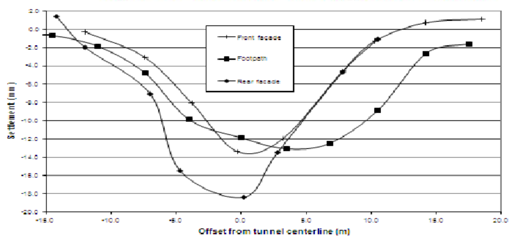

Figure 8: Final settlement troughs

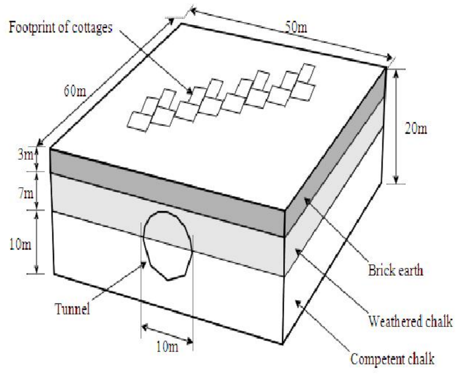

Figure 9: Model of ground

V. CONCLUSION

In any numerical modeling procedure for prediction of tunneling effects on structures, it is always important to adequately reproduce the 'greenfield' settlement trough without the building present. Some previous numerical work has had difficulty in this area, but this study achieved satisfactory results that agreed with the Gaussian prediction provided the tunnel was relatively deep, *i.e*. with a cover to diameter (*C*/*D*) ratio greater than 3. Evidence was obtained that indicated that for shallow tunnels (C/D < 3), the Gaussian model predicts a trough width that is too narrow, and hence over-predicts curvatures and therefore building damage. The models generally over-predicted the damage by one category compared to that observed in the field, for example predicting "Slight" instead of "Very slight" damage. The distribution of damage also tended to be more localized in area than observed in the field. It is thought that softening the tensile stress-strain response of the elastic, no-tension masonry constitutive model, and introducing a small but realistic tensile strength for the masonry may mitigate these two effects. The representation of columns of smaller windows in façades by vertical regions of reduced stiffness in the model was found to be a successful approach, provided the openings are not in a region of significant shear deformation or where arching is important to building behavior. In these cases, the representation of discrete openings is recommended.

VI. FUTURE WORK

Comparison of field data and modeling results on this project has shown that it is not possible to predict the development of a particular crack in a building façade by means of a numerical model. The field data showed that individual cracks do not behave in a way that is consistent with the boundary conditions applied to the building as a whole. Further research is recommended into the reasons for this phenomenon.

REFERENCES

[1] CIRIA (1994). *Foundations in chalk*. Project Report No. 11.

[2] Mair, R.J., Taylor, R.N. and Bracegirdle, A. (1993). Subsurface settlement profiles above tunnels in clays. *Géotechnique*, 43 (2), 315-320.

[3] Macklin, S.R. and Field, G.R. (1999). The response of London clay to full-face TBM tunneling at West Ham, London. *In: Proc. Int. Conf. On Urban Ground Engineering*, Hong Kong, 11-12 November 1998. London: Thomas Telford.

[4] Kwando, J., Takahashi, A., Honda, T. and Miki, K. (2000). Centrifuge investigation on deformations around tunnels in nailed clay. *In: Proc. Int. Sympos. on Geotechnical Aspects of Underground Construction in Soft Ground*, Tokyo, 21-23 July 1999.Rotterdam: Balkema, 245-250.

[5] Houssay, G.T. (1999) A model for the variable stiffness of untrained clay. *In: Proc. Int.Symposium on Pre-failure Deformation Characteristics of Geo materials*, Torino, September. Rotterdam: Balkema.

[6] Finno, R.J. and Clough, G.W. (1985). Evaluation of soil response to EPB shields tunneling. *Journal of Geotechnical Engineering, ASCE*, 111 (2), 155-173.

[7] Jeffery, S. (1993). *The Mansion House*. London: Phillimore & Co. Ltd.

[8] Rowe, R.K., Lo, K.J. and Kack, G.J. (1983). A method of estimating surface settlement above tunnels constructed in soft clay. *Canadian Geotechnical Journal*, 20 (1), 11-22.

[9] Sloan, S.W. and Randolph, M. (1983). Automatic element reordering for finite element analysis with frontal solution schemes. *Int. J. of Numerical Methods in Engineering*,19, 1153-1181.

[10] Freshman, W.W., Hellings, J.E. and Snowden, C. (1994). Protection of the Mansion House against damage caused by ground movements due to the Docklands Light Railway Extension. *Proceedings ICE Geotechnical Engineering*, 107 (2), 65-76.

[11] Gray, D.H.; Mitchell, J.K. (1967). "Fundamental Aspects of Electro-osmosis in Soils."Journal of the Soil Mechanics and Foundations Division, Proceedings of the American Society of Civil Engineers **93**(SM6): 209-236.

[12] Lageman, R.; Pool, W.; Seffinga, G. (1989). Electro reclamation - Theory and Practice. Chemistry and Industry 18th September 1989, pp.585-590.

[13] Lee, H-H.; Yang, J-W. (2000). "A new method to control electrolytes pH by circulation system in electro kinetic soil remediation." Journal of Hazardous Materials **B77**: 227-240.