SOME EXPERIENCES OF MODELLING TUNNELLING IN SOFT GROUND USING THREE-DIMENSIONAL FINITE ELEMENTS

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ABSTRACT: A three-dimensional finite element model has been developed at Oxford University to study the effects of subsidence from soft ground tunnelling on adjacent surface structures. Simulation of excavation and the ground loss associated with tunnelling are incorporated in the model. Surface buildings are also included, as groups of interconnected two-dimensional façades composed of an elastic no tension material, to model masonry. This paper describes the development, implementation and performance of procedures to model the tunnelling processes. A description is also given of the methods used to generate the finite element meshes and to post-process the data.

1 INTRODUCTION

The construction of a tunnel in soft ground usually leads to subsidence of the ground surface. The size and shape of the settlement trough at a *greenfield* site may be estimated by a well-tested semi-empirical approach, described in many references (e.g. Mair *et al.*, 1996). The presence of a surface structure usually changes the settlement profile, due to the interaction between the ground and the building. If differential settlements are significant they may damage the building. It is important, therefore, that reliable methods for settlement and damage prediction are available if urban tunnelling schemes are to be successfully promoted.

Numerical methods, principally the finite element method, have been applied for some years to this problem, although most studies have been two-dimensional and have not included a realistic model of a building (e.g. Potts & Addenbrooke, 1997). It is thought that realistic modelling of this problem can be achieved only with three-dimensional models. In order to respond to this need, research has been underway at Oxford University since 1993 to develop a three-dimensional numerical model for the prediction of the effects of soft ground tunnelling on surface structures.

A paper presented at the last in this series of conferences described the proposed composition of a three-dimensional finite element model of tunnelling (Burd *et al.,*1994). The model has now been implemented and is described in detail in Augarde (1997) and Liu (1997). Some initial results using the model are given in Burd *et al*. (1998). This paper describes some of the challenges met during its development and subsequent use and discusses some of the issues that face developers of complex numerical models of this type. The paper is concerned particularly with the procedures adopted to model tunnel installation.

2 DESCRIPTION OF THE NUMERICAL MODEL

The numerical model uses finite elements to represent the ground, a tunnel and a surface structure. A typical problem is shown in Figure 1, where a straight circular tunnel of 5m diameter, with its axis at a depth of 10m, is constructed beneath four connected façades of a large masonry building of plan dimensions 10m by 20m. The building is unsymmetrical with respect to the tunnel centreline. The tunnel is assumed to be installed with a ground loss of 2%.

A non-linear, elasto-plastic material formulation is used to model an overconsolidated clay deposit with increasing stiffness and strength with depth. The building is constructed from masonry, modelled as an elastic no tension material. Tunnel linings are linear elastic. Tunnel installation is simulated in the model in discrete stages, each of which consists of the simultaneous removal of soil and activation of shell elements to simulate the liner. Four stages of tunnel installation are used in the analysis shown in Figure 1. Elements that are to be removed in the current stage are ignored when the structure stiffness is formed. Nodal loads are imposed on elements surrounding the excavated block to render the exposed faces free of surface tractions (Brown & Booker, 1985). The calculation of these loads is described in Augarde *et al.* (1995).

An important requirement of a finite element model of tunnelling is the ability to model ground loss. Ground loss arises in practice from two sources; radial movement of the soil around the tunnel liner (tail loss) and inward movement of the soil at the tunnel heading (face loss). The amount of ground loss that occurs in practice is determined mainly by the installation method and the quality of the construction procedures. In this model, the excavated face of the tunnel is unsupported, thus permitting movement to represent face loss. Tail loss is simulated by uniform shrinkage of the lining elements in a plane normal to the tunnelling centreline. This occurs at the same time as excavation and lining activation.

While large masonry buildings are three-dimensional structures, their most significant feature, with respect to settlement damage, is the in-plane response of the main façades. The façades are represented by meshes of six-noded triangular plane stress elements, each

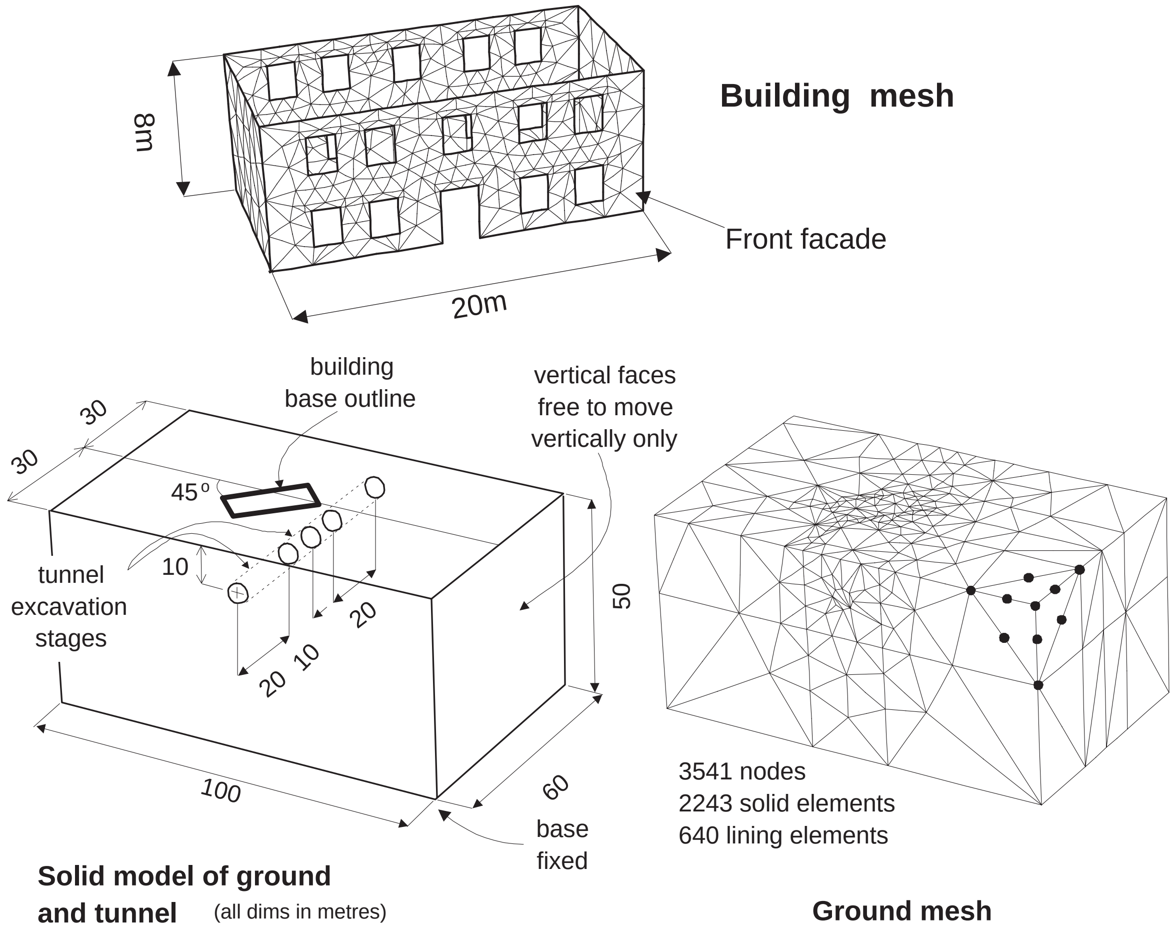


Fig. 1; Example problem

façade having its own local co-ordinate system. The façade meshes are joined to each other, and to the surface of the ground mesh, using a novel system of tie elements that implement the kinematic constraints linking each local two-dimensional co-ordinate system with the three-dimensional global system (Liu, 1997).

The various components of the model described above have been implemented in OXFEM, which is a finite element program written in Fortran 90 and developed at Oxford University. Non-linearity is accommodated in an incremental scheme, the Modified Euler method. Loads are applied in steps, followed by an equilibrium check to determine the out-of-balance force at each nodal degree-of-freedom. This out-of-balance force is then corrected in the next step. Incremental techniques such as this require the user to specify the numbers of steps, a decision that requires some experience of the non-linearities involved.

3 NUMERICAL MODEL OF TUNNEL INSTALLATION

A numerical model of tunnelling requires consideration of the structural behaviour of the liner, and the ground loss that invariably occurs in practice during tunnel installation. A variety of modelling procedures have been developed for 2D analysis (e.g. Rowe, 1983; Potts & Addenbrooke, 1997); no established procedures are, however, available for 3D analysis. In the numerical model described in this paper, the liner is modelled using shell elements, and the effect of one component of ground loss (tail loss) is included by numerical shrinking of the liner elements. These procedures are discussed below.

3.1 The choice of lining element formulation

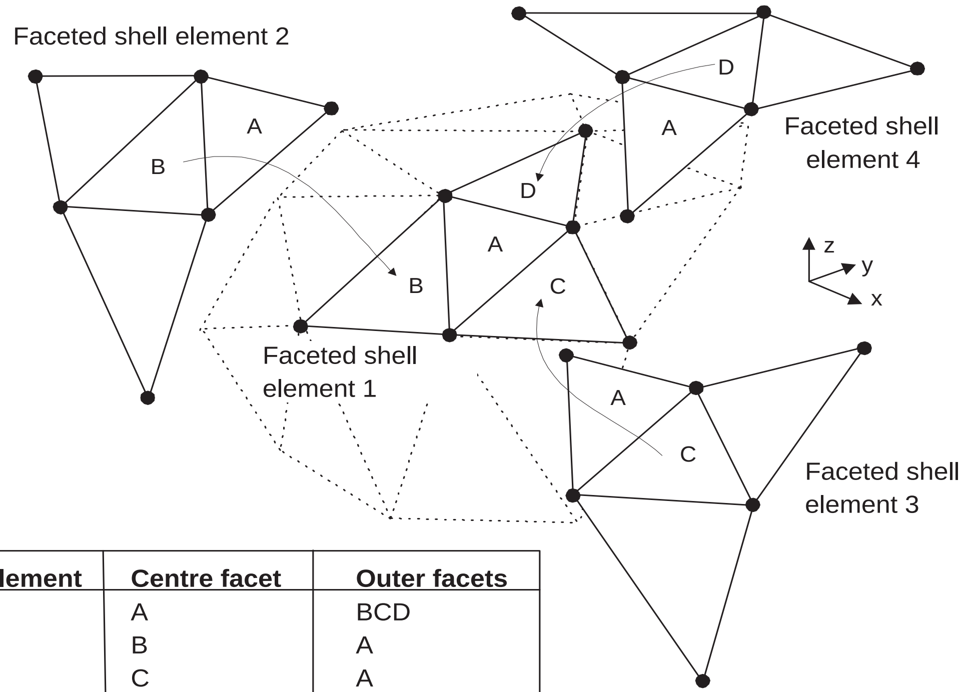
The choices of element formulations for the ground and building are relatively straightforward. The choice of a suitable shell element for the tunnel lining is complicated by the fact that a large number of different approaches are proposed in the literature. Little work appears to have been published, however, on the behaviour of shell elements when connected to continuum elements. It is possible, for example, that the use of these incompatible elements may lead to numerical problems.

In determining a suitable element a choice has to be made from the two main classes of shell element:

- true curved elements based on classical shell theory or derived from the degeneration of a solid continuum element,
- faceted elements, where bending stiffness is attributed to a plate element and membrane stiffness to a plane stress, continuum element.

Many formulations in the first category have been proposed. Yang *et al.* (1990), for example, reviewed over 280 publications relating mainly to curved shell elements. Degenerated solid shell elements appear to be more commonly used than those based directly on shell theory, and are available in various commercial packages. Faceted formulations are usually less complicated both to code and to operate, since membrane and bending effects are uncoupled. This has the advantage that separate formulations for bending and membrane actions can be adopted (Hughes *et al*., 1995).

The element used in this project was a faceted shell with displacement, but no rotational, degrees of freedom. The formulation was developed by Phaal & Calladine (1992) and is novel in that bending stiffness is provided by a six-noded plate element having four flat triangular facets while membrane stiffness is provided by three-noded triangular elements which overlay each facet. Thin plate elements must fulfil the Poisson-Kirchoff requirement (i.e. continuity of displacement gradient) and this is met in conventional plate formulations by the inclusion of rotational degrees of freedom at nodes, in addition to the translational degrees of freedom. In Phaal & Calladine’s shell element, the facets of adjacent plate component elements overlap to achieve this continuity without the need for rotational degrees of freedom (Figure 2). This unconventional feature leads to many difficulties in the inclusion of this element in a standard finite element program.



Element	Centre facet	Outer facets
1	A	BCD
2	B	A
3	C	A
4	D	A

Fig. 2; Overlapping shell elements

This element was chosen because it appeared to offer reasonable behaviour and accuracy without the need for rotational degrees of freedom at the element nodes. The lack of rotational degrees of freedom was thought to have three advantages:

- The number of global degrees of freedom in the mesh is less than that required for conventional shell formulations,
- all nodes in the model all have the same number of degrees of freedom. This makes the element more convenient to implement.
- There is no need to consider the in-plane rotational degree of freedom (the drilling degree-of-freedom).

The Phaal & Calladine (1992) formulation is, at first sight, highly complex. The formulation is presented in a general form, and further work is needed to cast it in a form that may be implemented in a finite element program. The original formulation was described in the context of the analysis of linear shell problems. The model described in this paper is non-linear, however, and this required some minor further development of the shell element formulation.

The linear elastic material model for the tunnel lining requires three properties to be specified: Young's modulus, E, Poisson's ratio, ν, and thickness, t. These parameters are used to derive the flexural stiffness of the plate component of the shell, D where

$$D = \frac{Et^3}{12\left(1-\nu^2\right)} \tag{1}$$

and the membrane stiffness component as Et (per unit length of lining). The elements performed reasonably well when used to model tunnel linings. There are, however some unresolved difficulties using the elements in combination with the non-linear elasto-plastic material model for the ground. It is occasionally found that the analysis fails to converge with the out-of-balance force at the equilibrium check (see Section 2) increasing with each step. The problem may be avoided by increasing the flexural stiffness, D, but this leads to a tunnel liner that is substantially stiffer than would be expected in practice. While the need to impose unrealistic properties on the liner is unfortunate, since it is difficult then to interpret the stress resultants, it does not unduly affect the surface response of the model, which is of primary interest in this study.

3.2 Modelling ground loss in 3D

Procedures for modelling ground loss in 2D finite element analysis are well established. Potts & Addenbrooke (1997), for example, use a technique in which the unloading caused by soil removal is carried out in a series of small increments. At the end of each increment the amount of ground loss is recorded; when the ground loss reaches the specified value then the calculation is terminated. A more complex procedure is described by Rowe (1983). In this procedure separate meshes of beam elements to model the lining and continuum elements to model the soil are used. The soil mesh contains a zone of elements that are removed to simulate excavation and the lining elements are activated to simulate the installation of the lining. The mesh of lining elements is separated from the surrounding ground by a specified gap. After the soil has been excavated and the liner elements activated, the ground closes onto the lining mesh as the soil around the tunnel unloads. Numerical procedures are used to model the contact between the soil and liner after the gap between the two meshes closes.

The above methods would be complex to implement in a 3D analysis; this is particularly the case for the gap parameter approach of Rowe (1983). A third approach was therefore selected in which the elements within the tunnel are removed (to model excavation) and, simultaneously, lining elements are activated to model the liner. At the end of this procedure the lining elements are subjected to a uniform hoop shrinkage to develop the required amount of ground loss. The hoop shrinkage is achieved by the application of a suitable set of radial forces within the tunnel liner. The process leads to fictitious stresses within the liner, but the liner is elastic and so this does not affect the way in which the ground and the liner interact. An important feature of this finite element model is that the tunnel is installed in separate stages. During any stage of tunnel installation (except the first one) it is important to ensure that the applied numerical shrinkage does not cause additional shrinkage in the liner installed during the previous stage. This was achieved by constraining to zero the shrinkage deformation applied at the tunnel heading (except for the final

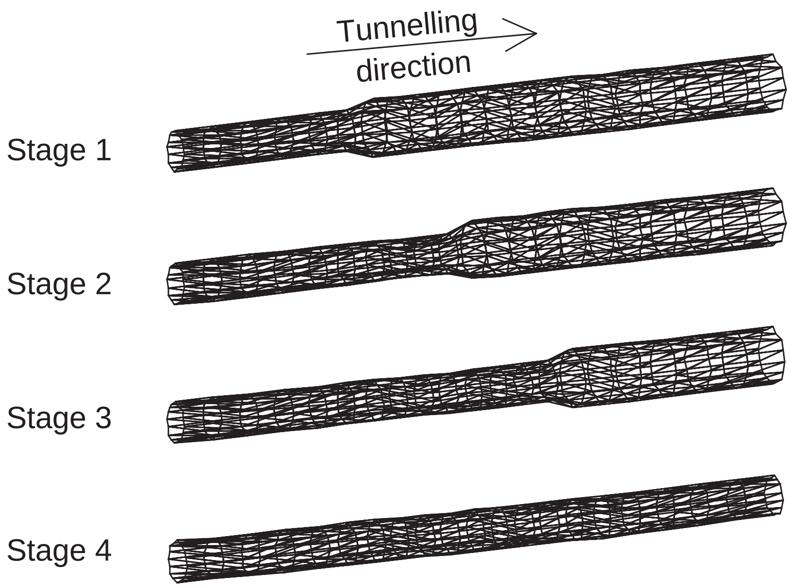


Fig. 3; Lining shrinkage for each tunnelling stage

tunnelling stage). The nodal forces associated with these constraints were then removed during the shrinkage phase of the next tunnelling stage. This was a somewhat artificial, but necessary, device to achieve satisfactory modelling of volume loss for incremental tunnel construction in a 3D model.

Figure 3 shows the lining elements at each stage of the analysis associated with the mesh in Figure 1. The shrinkage at each stage, which is displayed to an exaggerated scale, follows that specified by the user.

4 PERFORMING ANALYSES WITH THE MODEL

4.1 Preparation of input data and interpretation of output

The meshes used with the numerical model are unstructured, to permit localised refinement around the tunnel and below the building. This feature precludes hand generation, and an automated method must be used. Unstructured mesh generation has been the subject of considerable research in other areas, notably computational fluid dynamics, and guidance is available from the literature in this area. It is recommended, however, that a commercial package is used, since the effort involved in writing a robust generator for three-dimensional unstructured meshes is considerable. The package used in this research is I-DEAS, primarily a solid modelling package but with excellent mesh generation tools.

It is vital that the output from the analysis is checked carefully. This is a complex task, however, because of the large amount of computed data. Apart from checking that the nodal forces are in equilibrium, which is an integral part of the solution phase of the analysis (Section 2), it is important to check that the tunnel installation procedures produce the specified amount of ground loss. This finite element model is based on the assumption of undrained soil behaviour, and therefore that the soil is incompressible. Although the soil model adopted in the analysis ensures incompressible plastic deformation, it is not possible to specify perfect incompressibility in the elastic region. (In these calculations a Poisson's ratio of 0.49 was adopted). It is possible that this loss of incompressibility may lead to the volume of the surface settlement trough being less than the total volume of ground loss generated by tunnel installation. To check that volumetric deformation of the soil did not reduce the volume of the surface settlement trough by an appreciable amount, careful comparisons were made between these two volumes. The ground loss achieved in the tunnel is computed from the tunnel liner displacements. The procedure is based on the computation of the volume of a mesh of tetrahedra within the tunnel formed by 3D triangulation. The total volume of the settlement trough at the ground surface was computed by numerical integration of the surface settlements. It was found that the ground loss could be controlled to within 0.1%, and that the volume of the surface trough correlated very closely with the ground loss.

The final stage of an analysis is the post-processing of output data into a useful format. The large volume of data precludes any hand manipulation and the options are either in-house or commercially available software. In this research, use is made of both approaches with I-DEAS and an in-house program, 2CAN, used for generation of contours of surface settlements. 2CAN is also used to process and display the cracking of building façades. This is a feature not available in I-DEAS, and was one of the specific post-processing tools to be written for this project. Figure 4 shows the predicted crack pattern for each tunnel stage for the front façade of the building in the analysis of Figure 1. The intensity and inclination of cracks in the masonry model can be obtained from the OXFEM output. These data are displayed as collections of lines, at the given inclination, centred on the element integration point. The number of lines indicates the magnitude of the crack strain. (Crack strain is closely associated with principal strain normal to the crack and provides a convenient measure of the intensity of cracking, Liu, [1997]). This display gives an immediate indication of the damaged areas of the building façades.

Figure 4 indicates that the location of the most severe damage to the front façade changes as the tunnel approaches and passes beneath the building. At stage 1, the tunnel heading is closest to the large door opening in the façade, and transverse settlements cause cracking adjacent to and above this region. There is little effect in the façade directly ahead of the tunnel at this stage. The greatest damage is predicted at the end of stage 2, when the tunnel heading is situated between the front and side façades. At this point, the building is subjected to the combined effects of differential settlements along the tunnel axis and in a transverse direction. Much of the cracking seen at stage 1 has closed up by stage 2 due to the changing settlement profile along the base of the façade. Stages 3 and 4 are similar since the tunnel heading has progressed beyond the building. The differential settlements along the tunnel axis have reduced, leaving slight damage from transverse differential settlements.

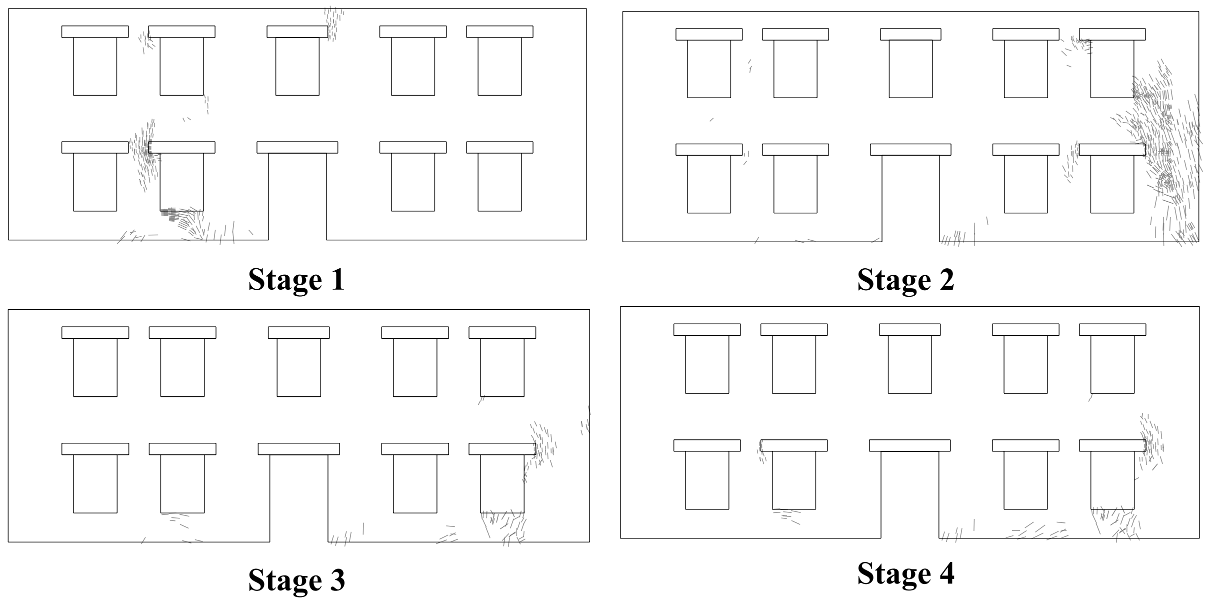


Fig. 4; Predicted crack patterns in the front façade of the building analysed using the model shown in Figure 1

5 ANALYSIS COSTS

Typical hardware used to run analyses with the model described in this paper is currently a Sun Microsystems UltraSparc 2 machine with processors running at 200MHz and 256Mb of RAM. Runs of the analysis shown in Figure 1 with this hardware are completed in approximately five days. During solution, virtual memory use rises to approximately 250Mb, which can be accommodated entirely within the RAM, provided other processes are small or prevented from running via a queue system. While this level of hardware configuration is of a high specification, it is not excessively expensive, considering the value of the information it is able to provide. Equally, while runtimes are long they are not excessively so, considering the complexity of the model.

6 FUTURE DEVELOPMENTS

Further work is underway to increase the scope of the model and to improve its efficiency. In particular, the modelling of compensation grouting, using interface finite elements, is under development. It is also intended to develop an effective stress model for soil in order to study the effect of consolidation settlements. The behaviour of the shell elements in the model has prompted an investigation of other methods of modelling volume loss. It appears possible to use thin continuum elements with high stiffness for the lining, which should remove any difficulties associated with element non-compatibility.

A parallel project began in 1996 to compare predictions from the model with field data from tunnelling schemes in London. This, and further studies, should serve to validate the model and its components and indicate areas for improvement. Work has recently begun on porting the code to run on OSCAR, the Oxford Supercomputer where significant reductions in solution times are anticipated.

7 ACKNOWLEDGEMENTS

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