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A multi-stage finite element analysis of a complex tunnel section in London Clay

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Abstract

This paper summarises selected numerical modelling analysis results related to the stability investigations of proposed tunnel sections within the Liverpool Street Station development project known as Crossrail. The Crossrail project is a proposal for a new main line rail tunnel link across London.

The sections analysed require a multi-stage cumulative displacement approach with delayed support installation. The excavation support consists of multi-stages of temporary shotcrete with a central pillar introduced late in the sequence.

The input parameters and assumptions to achieve the analysis are described as well as the results and their interpretations.

Displacements and stresses are described in stages and the loads and bending moments within the shotcrete temporary support are presented.

Introduction

The analysis described in this paper concerns the design of a tunnel section using numerical modelling. The Crossrail Project for which the analysis has been conducted is a proposed rail link between the counties to the East and West of London. The proposed route runs from Reading to Paddington along the Thames valley over existing lines. New tunnels are planned to run under Central London, serving subterranean stations at Paddington, Bond Street, Farringdon and Liverpool Street / Moorgate.

In the Bethnal Green area the tunnels end and Crossrail continues over existing suburban tracks via Stratford, and Romford to Shenfield.

The analysis of the Liverpool Street Station/Moorgate Station tunnel is described in the paper. This section is situated predominantly in London Clay with the invert of some tunnels just set in the upper part of the underlying Woolwich and Reading Beds. There is 33.5m of overburden to the platform level with a few meters of gravel present towards the surface.

Technical Specification of Model

To model a tunnel, a validated technique and a reliable and representative set of input parameters is required. An understanding of the model results and their consequence concerning support behaviour is also essential.

The numerical model utilised for this study was a Finite Element model developed for tunnel simulation at the Department of Mineral Resources Engineering at the University of Nottingham. Its full specification is given in Boughrarou and Reddish (1995). In brief it has the following features.

1- Stress is not modelled by external forces to the structure but is introduced into the elements as they are formed. This method of stressing has proved to be both quicker and more realistic than some of the alternatives.

2- Because the elements are created in a stressed state the restraints at the external boundary of the problem can be more realistically described. Essentially, the boundaries of the model are chosen to be sufficiently remote to not influence the problem, and are then locked.

3- The model was developed to simulate high deformation complex non-linear problems and the isotropic elasto-plastic analysis being conducted is well within its capabilities. Anisotropy and visco-plasticity are available if required.

4- The method of stressing the model makes it possible for stress and failure output from one model stage to be used as the starting point for further excavation. This cumulative stressing approach is highly suited to multistage excavations where it is an important feature.

5- The stress loading in the model is easily specified and can be both uniform or a function of depth in both the X and Y planes.

6- output graphics from the program can be written to customers specification allowing more rapid assessment of results.

Geotechnical Specifications

Geotechnical assessments were undertaken and the following list of properties specified for the model.

Poisson's Ratio , $\nu = 0.45$
Bulk Unit Weight, $\gamma = 20$ kN/m^3
Elastic Modulus,Eu = 24 + 1.9z MN/m^2
where, z = depth in metres below 105m LULTD.
Depth to Tunnel axis, z = 33.5m
Undrained Shear Strength, Cu = Su = 90 + 7.0z kN/m^2
where z = depth in metres below 105 LULTD
Undrained Angle of Shearing Resistance, $\phi_u = 0$

The shear strength and elastic modulus were specified in terms of depth of burial to take into account the influence of consolidation on these properties.

The initial stress conditions for the section considered were controlled by a number of factors as follows.

1) A surcharge load at the surface of 75kN/m^2 simulating the presence of structures
2) Vertical stress generated by the weight of overburden clay
3) The coefficient of earth pressure Ko=1.5. A figure of 0.8 was used in sections where single stage analysis was required.

Support specifications

For purposes of modelling, stiffness were required for the following support elements.

Short term Shotcrete Modulus E = 5GN/m^2
Long term Shotcrete Modulus E = 15GN/m^2
Composite Steel/Concrete Column Modulus E = 205GN/m^2
Composite Steel/Concrete Column Poisson's Ratio $\nu = 0.3$
Reinforced Concrete Beams Modulus E = 28.4GN/m^2
Reinforced Concrete Beams Poisson's Ratio $\nu = 0.2$
The geometry of the excavation and its supporting components is shown in figure 1

Extraction sequence details

The extraction sequence is shown in figure 2. The extraction sequence is quite complex and involves insertion and re-excavation of support elements as well as lower stresses in the earlier extraction stages. Final stresses were calculated for the end of each stage and used as the starting point for the next stage. The final stage result therefore required 7 finite element runs in sequence. The stages were specified as follows.

Stage 1
The in-situ stresses in the crown of zone 1 are reduced to 50%, then the crown of zone 1 is excavated and a 300mm thick shotcrete lining is constructed, assuming a short term modulus for the shotcrete.

Stage2
The elastic modulus of London Clay in the area of the bench and invert of zone 1 is reduced by 50%, then the bench and invert of zone 1 are excavated and a 300mm shotcrete lining is constructed, assuming a long term modulus for the shotcrete.

Stage 3
A reinforced concrete beam along the invert of zone 1 to support columns spaced at 6.966m centres comprising steel sections embedded in reinforced concrete. A reinforced concrete beam to be supported by the columns is constructed at the crown of zone 1.A long term modulus for the shotcrete is specified.

Stage 4
The elastic modulus of London Clay in the area of the crown of zone 2 is reduced by 50%, then the crown of zone 2 is excavated. A shotcrete lining is constructed assuming a short term modulus for the shotcrete. For the lining in zone 1 a long-term modulus is assumed.

Stage 5
The elastic modulus of London Clay in the area of the bench and invert of zone 2 is reduced by 50%, then the bench and invert of zone 2 is excavated and

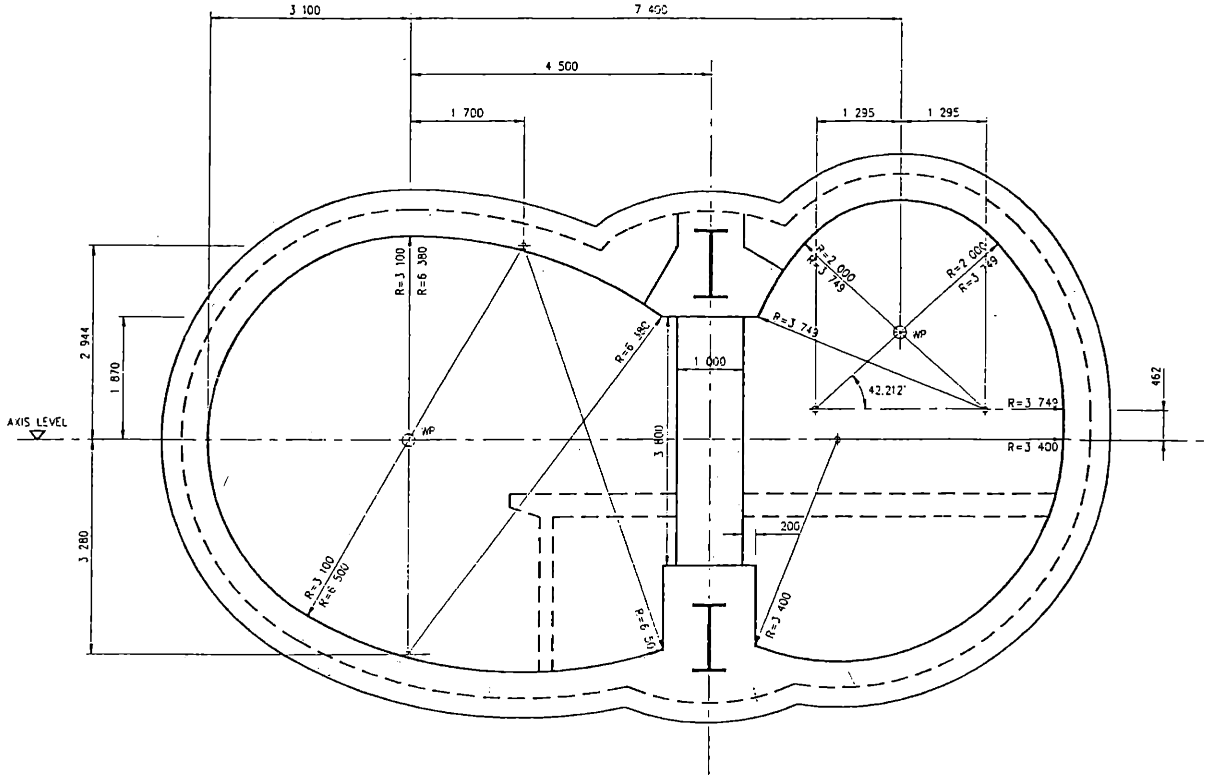


Figure 1. Geometrical details

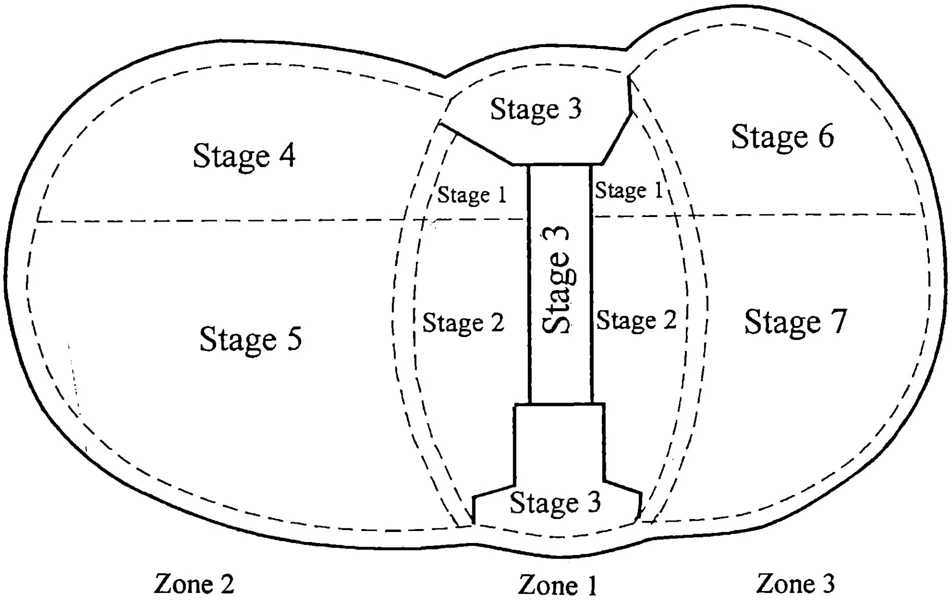


Figure 2. Construction stages

a shotcrete lining is constructed assuming a short term modulus. A short term modulus for the shotcrete in zones 1 and 2 is assumed.

Stage 6

The elastic modulus of London Clay in the area of the crown of zone 3 is reduced by 50%, then the crown of zone 3 is excavated and a shotcrete lining is constructed, assuming a short term modulus. A long term modulus for the shotcrete lining in zones 1 and 2 is assuming.

Stage 7

The elastic modulus of London Clay in the area of the bench and invert of zone 3 is reduced by 50%, then the bench and invert of zone 3 is excavated and a shotcrete lining is constructed. A long term modulus for the shotcrete lining in zones 1, 2 and 3 is assumed.

Results

Due to space constraints results for three key stages have been presented. Figures 3a, b, and c show the displacement, axial thrust and bending moments in the lining at the end of stage 3. Figures 4a, b, and c show the displacement, axial thrust and bending moments in the lining at the end of stage 5.
Figures 5a, b, and c show the displacement, axial thrust and bending moments in the lining at the end of the final stage 7. Figure 6 shows the ground displacement for the final stage 7. The results have been presented in a form to allow the key loading mechanisms in the lining to be quickly assessed by the structural engineers involved with the project.
The results show a number of interesting features. Initially high bending moments are associated with the stage 3 excavation which was a taller than wide in a predominantly horizontal stress field. A good example of a bad shape for the stress field. By the final stage 7 when the excavation is much wider than tall the bending moments are much reduced even though the excavation is three times as large. A good shape for the stress field.

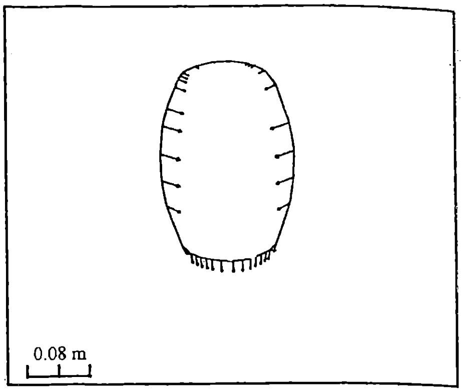
Interpretation of results

As a general principle the shotcrete lining was considered as a thin membrane and the selection of a good geometric shape was all important.
A good shape reduces considerably bending moments and avoids the need to consider shear forces.
The factors of safety employed were:
Self weight, structural finishes-safety factor is 1.4
Geostatic loads, hydrostatic loads surcharge and live loads-safety factor is 1.2
The column interaction curve was used as the main design basis. Figure 7 shows the principles of those graphs in terms of the requirement for reinforcement, see Bennett (1962).

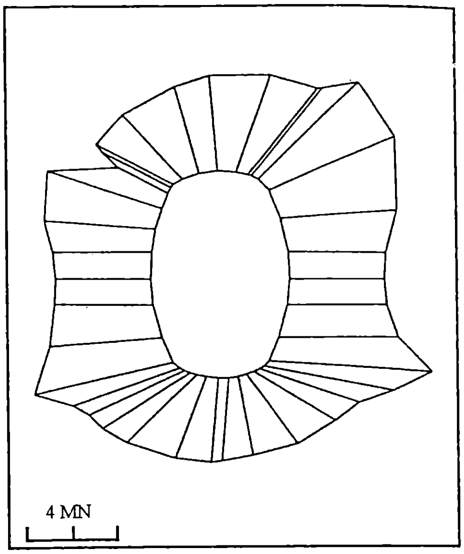
$\alpha = N/bhF_{cu}$
$\beta = M/bh^2F_{cu}$

where,
N is the axial thrust (N),
M is the bending moment (Nm),
b and h are the width and the depth in metres of the section respectively, and

(a) Displacement vectors



(b) Axial thrust



(c)Bending moments

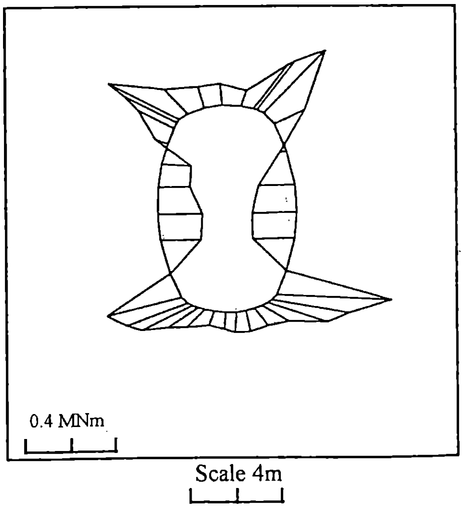
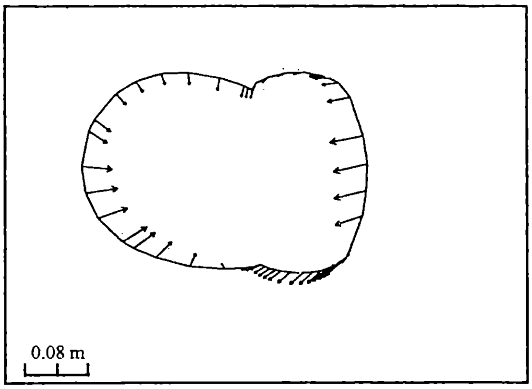
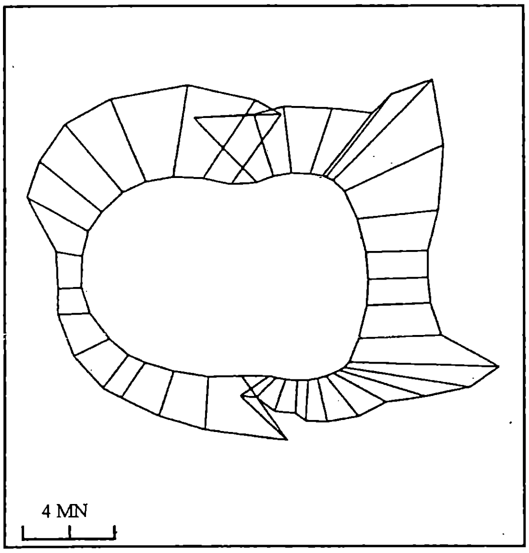


Figure 3. Displacement vectors, axial thrust and bending moments around the shotcrete lining (Stage 3)

(a) Displacement vectors



(b) Axial thrust



(c)Bending moments

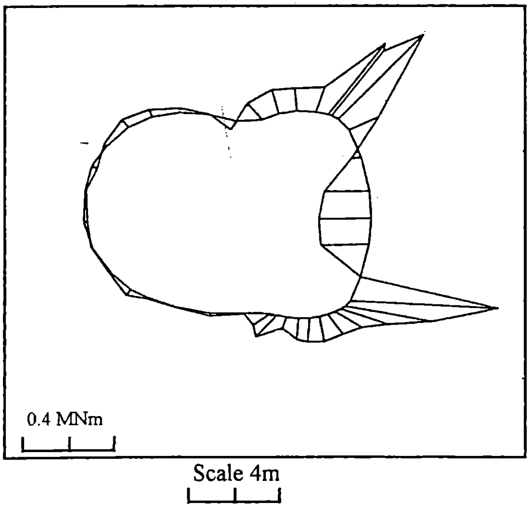
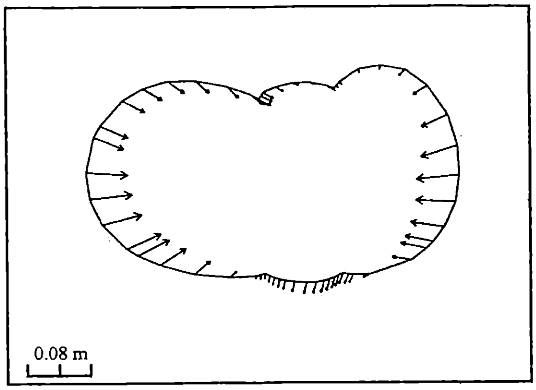
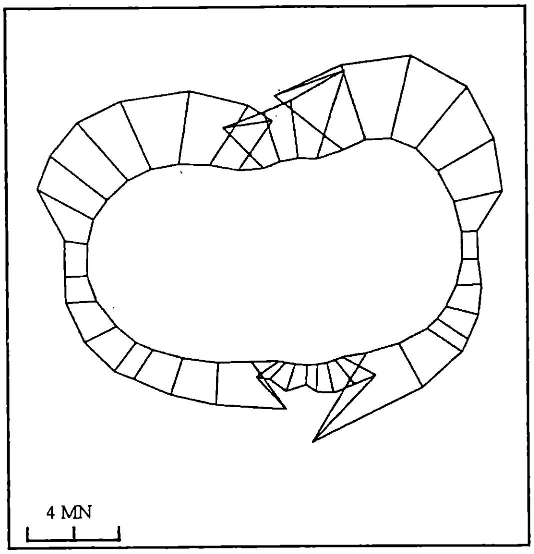


Figure 4. Displacement vectors, axial thrust and bending moments around the shotcrete lining (Stage 5)

(a) Displacement vectors



(b) Axial thrust



(c)Bending moments

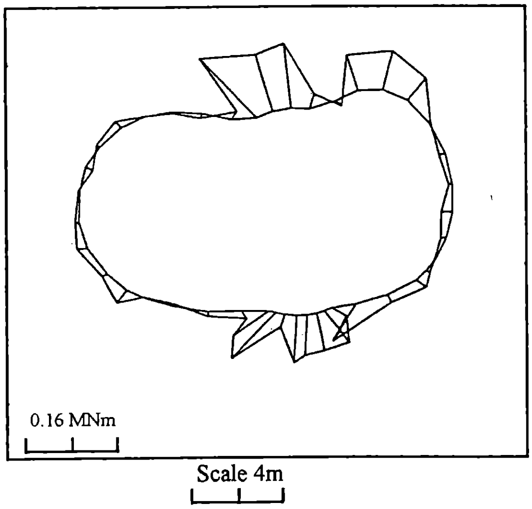


Figure 5. Displacement vectors, axial thrust and bending moments around the shotcrete lining (Stage 7)

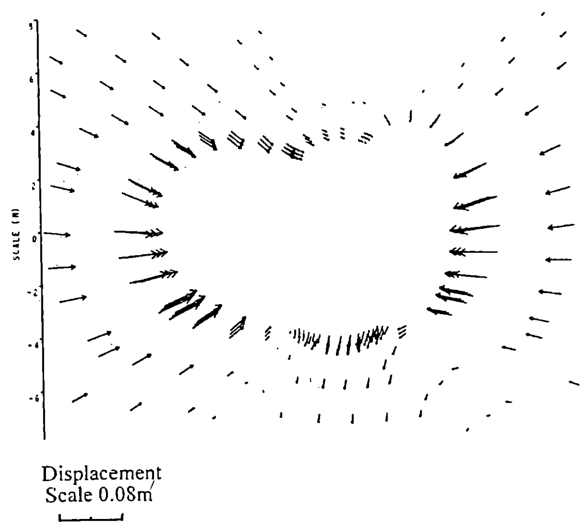


Figure 6. Ground displacement (Stage 7)

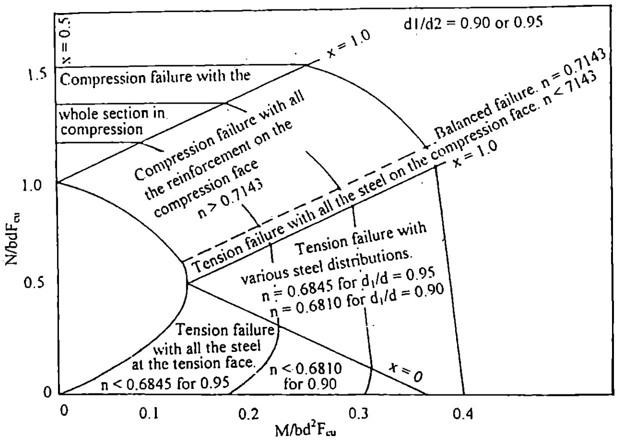


Figure 7. Column interaction curve for reinforced section

F_{cu} is the 28 day characteristic cube strength in Pa.

The reinforcement design has to be limited to 1% each face otherwise it has to be fully restrained by links for example according to Clauses3, 4, 5, 7 & 12 of BS 8110 part1.

Where unfavourable geometry was unavoidable and high shear forces occur, these can be accounted for by checking the critical sections for combined direct and shear stress using the Mohr circle method for the uncracked section where the maximum ultimate tensile stress is $Ft = 0.24\sqrt{F_{cu}}$.

For a cracked section the method of BS 8110 for beams would be appropriate, however the moment would be so large as to place the position in the column interaction curve in the tension failure zone near to or on the M/bh^2F_{cu} axis that a reassessment of the lining geometry will be required.

Conclusions

Numerical modelling offers considerable scope for detailed design of tunnel linings. It is important that the key output parameters from the model are concentrated upon and that the vast array of potential output is avoided particularly in multi-stage excavations such as this. Modern packages are able with care to cope with multi-stage excavation although considerable engineering judgement is required in specifying realistic loading at the various stages.

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