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DESIGN OF SCL TUNNELS IN SOFT GROUND USING EUROCODES

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Abstract. *Numerical methods are increasingly used to predict ultimate limit state stresses in underground structures. Considerable experience has been gained in the UK of design and construction of SCL tunnels in soft ground, particularly under London. The urban setting means that extensive monitoring is used to ensure the safety of existing structures, and therefore a wealth of monitoring data has been gathered and much of it has been published as case studies.*

Verifying that the results of a numerical analysis are reasonable is a necessity, and these case studies of previous experience are commonly used in one of two ways in design. Firstly, the results of a numerical analysis may be directly compared to case studies of previous tunnels constructed in similar material at similar depth and with a similar construction method. This will provide a check that the results of the design analyses are reasonable. Secondly, a back-analysis of a case study may be used to validate the method of analysis itself. As long as the case study is similar to the tunnel being designed, this is a reasonable assumption but there are pitfalls to this method.

If the pitfalls are to be avoided, then the back-analysis needs to be performed in a scientifically rigorous manner. Nevertheless, one feature of both of these semi-empirical approaches to the design of tunnels that cannot be avoided is the emphasis on measuring displacements of the soil and the structure, while ultimate limit state (ULS) design codes such as the Eurocodes [1] are based on the prediction of stresses. Is it really safe to assume that because a numerical model has predicted displacements correctly it has also predicted stresses in the tunnel lining correctly? The paper will explore the importance of this question to the ULS design of SCL tunnels in soft ground. It will also propose solutions, such as the measurement of stresses for design verification.

1 INTRODUCTION

This paper will go through all the stages of the design of sprayed concrete lined (SCL) tunnels in soft ground. It will begin with the design philosophy, and then move onto more detailed aspects of design with particular emphasis on numerical analysis. All examples will involve the use of Eurocodes, and will concentrate on their application in the UK.

2 DESIGN PHILOSOPHY

The level of deformation required to mobilise a ‘ground arch’ in soft ground is very small compared to a tunnel in rock, and for an open-face tunnel there is no benefit to be gained by actively allowing any additional deformation to occur. In fact, there is field evidence from tunnels in London Clay to suggest that closing the invert as close to the face as possible may reduce the loads acting on the tunnel lining, possibly by preventing ‘loosening’ of the ground mass [2, 3, 4].

In addition, the observational method should only be used where there is sufficient time to introduce additional support measures when the monitoring shows that the tunnel is not stable or unacceptably large deformations are occurring. In soft ground, there is too much uncertainty about the amount of time available to respond to instability. Therefore, it is generally considered good practice to fully design the support measures before construction begins [5], and only to adopt minor changes during construction.

Numerical modelling is frequently used for SCL tunnel design, since the complex interaction with the ground, the sequential excavation and the often non-circular geometry are important aspects that cannot be represented explicitly using analytical techniques. 2D numerical modelling is far more common than 3D for routine design [6], although this may change as understanding of the limitations of 2D modelling increases and the ease and availability of 3D modelling increases. Some of the limitations of 2D numerical modelling will be described later in this paper.

3 BUILDING A NUMERICAL MODEL

This section will look at boundary distances, mesh refinement and calibration. Before beginning to build a numerical model of a tunnel, one must decide what outputs are desired. For example, the outputs could be lining stresses, or ground deformations, or both. The outputs will determine the boundary distances and mesh refinement, and also the way in which the model will need to be calibrated.

3.1 Boundary distances

Boundary distances may be ascertained by varying the boundary distance until the sensitivity of the desired output to a further increase in boundary distance is negligible. For example, in London Clay, van der Berg [7] and Thomas [8] established that a lateral boundary distance (distance from the tunnel centreline to the side boundary) of 20R, where R is the radius of the tunnel, was sufficient to make reasonable predictions of surface settlements and 13R was sufficient to make reasonable predictions of ground movements around the tunnel. Jones [3] found that a lateral boundary distance of 9R was sufficient to make reasonable predictions of lining stresses. These values were found for tunnels in London Clay and may vary for different ground conditions.

3.2 Mesh refinement

It is not easy to give any rules of thumb for mesh refinement, since it will depend on the type of elements used in the numerical model and the outputs required. The adequacy of the mesh refinement may be tested by varying the density of the mesh until further increases in density have a negligible effect on the output values. Particular attention should be paid to areas with high stress gradients, especially around tunnel junctions [3].

3.3 Calibration

There are two ways in which a numerical model may be calibrated, by directly comparing the results to case studies of similar tunnels constructed in the past or by calibrating the method of analysis itself by back-analysis of a case study and then using the same method for the new tunnel design.

Ideally, calibration of a numerical model would be focussed on the required output. However, the available case study field data is usually in the form of ground deformations, and often just the surface settlements. Therefore, numerical models are most frequently calibrated to predict the correct surface settlement trough, but are also used to predict tunnel lining stresses and deformations. It does not necessarily follow that because the ground deformations are correctly predicted the lining stresses and deformations will also be correct. One answer to this problem is to make more measurements of stresses in and on SCL tunnel linings, for instance by using radial and tangential pressure cells.

3.4 2D numerical models

For straight tunnels, 2D numerical models are often used to make predictions of stresses and deformations. Because the construction sequence cannot be modelled explicitly, 2D models require assumptions to be made about the stiffness and strength of the sprayed concrete at the various construction stages. There is also the '3D effect', i.e. relaxation of the ground ahead of the face and front-to-back arching of the ground.

The '3D effect' may be taken account of in various ways, the two most common methods being the relaxation approach and the hypothetical modulus of elasticity (HME) approach.

The relaxation approach will allow the stresses in the ground at the boundary of the excavated area to be reduced to a predetermined value by applying an internal pressure, and this relaxation value is usually expressed as a percentage of the existing stress prior to excavation. The value of relaxation to be used will need to be found by calibration, usually to case studies. The problem with using case studies is that the ground properties, geometry and construction sequence need to be similar to the tunnel being designed. If the new tunnel varies significantly from the case studies then the level of confidence in the results of the numerical modelling will be reduced. A parametric study during design applying a convergence-confinement approach [9] could be used, followed by application of the observational method during construction. However, in soft ground the support requirements must be known with a high degree of confidence before construction begins, as discussed in Section 2 above.

Calibration of a relaxation approach may also be achieved by comparison to a 3D numerical model. This may seem like an odd strategy, but for example a 2D model may be used to model the interaction between parallel tunnels where the individual tunnels have been modelled in 3D already.

The HME approach accounts for the '3D effect' by reducing the modulus of elasticity of the sprayed concrete lining [10]. This is acceptable if only ground deformations are required but the HME approach will not predict bending stresses in the tunnel lining accurately since the lining stiffness is being manipulated beyond realistic limits and the lining is being wishedin-place. The calibration must be done carefully, since the model is both calibrated to give expected values of ground deformations while at the same time it is being used to predict ground deformations. There is a danger that this kind of modelling may in the wrong hands give whichever answer it is told to give.

The same pitfall applies to the relaxation method if it is being used to predict ground deformations when the model has been calibrated to give the 'correct' ground deformations. If the modelling method has been calibrated by back-analysing a case study, then care must be taken where the construction sequence or progress rates vary in the new situation, since the relaxation factors may no longer be appropriate. If the model is also being used to predict lining stresses, one must again be aware that because the predicted ground deformations are reasonable then it does not necessarily follow that the lining stresses are predicted accurately.

The inherent weakness of 2D modelling of tunnels is that it is at best semi-empirical and inductive and at worst wholly a matter of judgement with just the illusion of a deductive scientific approach.

3.5 3D numerical models

3D numerical models allow the '3D effect' and time effects to be modelled explicitly. Time effects may include modelling of groundwater flows in soils that cannot be considered either drained or undrained during the timescale of construction, and ageing of the sprayed concrete causing the stiffness and strength to increase with time.

An example of how an advancing tunnel may be modelled in 3D is shown in Figure 1 below. This is a typical 6m diameter SCL tunnel with a top heading - bench/invert excavation sequence. The advance rate is 1.5m/day, working 24 hours per day. The invert is closed within 4m of the face. The time period taken for each advance is shown in brackets and the assumed age of the sprayed concrete lining is shown on the rings in red. Each modelling stage is given a reference number which is shown to the left of each diagram in black. Unshaded boxes represent unlined excavation and the shaded boxes represent a sprayed concrete lining.

At stage 21a, the time taken to excavate the top heading of ring 21 and install the lining in the top heading of ring 20 is 8 hours (shown in brackets). The elastic modulus of the sprayed concrete increases with age, and a relationship must be assumed to calculate this. If no data is available on the sprayed concrete to be used, then there are relationships based on test data in the literature, for example by Chang & Stille [11]. The age to be used in the model to calculate the stiffness is assumed to be 4 hours at this stage, since the load increment in the model is in reality coming onto the lining gradually over the period of the advance. An element of judgement is required, but using an ageing sprayed concrete model is much better than assuming it has the full 28 day strength and stiffness from the beginning.

In order to check the capacity of the lining at early age, the moments and axial forces may be extracted from the model and plotted on a moment-axial force interaction diagram as used for column design in Eurocode 2. The compressive and tensile strength of the sprayed concrete increases with age, and so an assumption has to be made about the age of the sprayed concrete at the time it is loaded. At stage 21a, for example, it would seem reasonable to take the strength at an age of 8 hours, since although the load from the ground comes onto the lining gradually, the lining forces calculated in the 3D model will be the full load at the end of the time period of the advance.

Figure 1 shows clearly that the assumptions made in a 2D model regarding the age and hence the stiffness and strength of the sprayed concrete lining at each excavation stage are largely subjective. During the top heading excavation of a 2D section, the age of the sprayed concrete varies between 4 and 52 hours before the next stage, which is the invert excavation (Stage 20&21b).

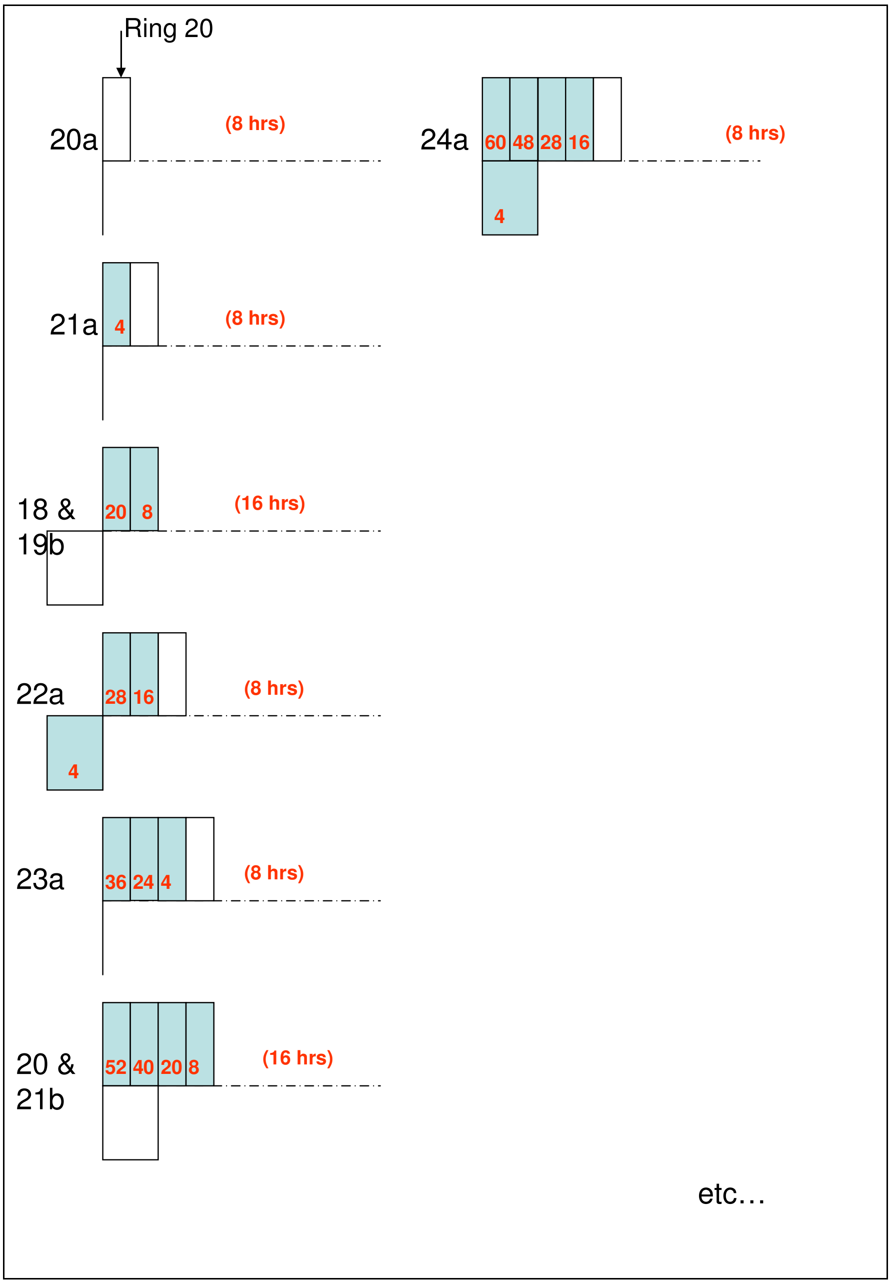

Figure 1: Schematic long section of an advancing tunnel model

4 EUROCODES

In this section, the application of the Eurocodes to ultimate limit state (ULS) design is considered. Firstly, the design approaches of Eurocode 7 [1] will be considered with special reference to tunnel design. Then the applicability of the concrete creep coefficients in Eurocode 2 [12] to the numerical modelling of sprayed concrete will be examined. Finally, the use of factors on moments and forces in Eurocode 2 will be discussed.

4.1 Eurocode 7 design approaches

In the UK, only Design Approach 1 of Eurocode 7 may be used. In Design Approach 1 there are two combinations, one where the characteristic ground parameters are used and the resulting actions are factored (DA1.1), and one where the ground strength is factored and the resulting actions are unfactored (DA1.2).

Reducing the ground strength in a 2D model does not make much sense because the ground deformation is controlled either by the fixed value of relaxation or the HME values. Reducing the ground strength is also unlikely to result in instability of the ground in a 2D model because there is no unsupported face and even the boundary of the excavation is supported. For the lining stresses, when the relaxation method is used, the ground pressure acting on the lining will initially be exactly the same regardless of the ground strength, since it is defined by the value of the relaxation factor. As the lining takes on load and deforms, the ground strength may play a small role in the ground-structure interaction and this may result in different values of bending moments, but will have a negligible effect on the tangential forces in the lining.

In a 3D model, the relaxation of the ground ahead of the face and the face stability can be modelled explicitly. The effect of reducing the ground strength was investigated in a 3D model of an SCL shaft construction and also in a 3D model of an SCL pilot tunnel and enlargement construction. In both cases, reducing the ground strength resulted in smaller tangential forces in the tunnel lining. Figure 2 shows the pilot tunnel lining tangential forces. In this case, applying a factor to the results from the unfactored undrained shear strength model will clearly result in the highest tangential forces.

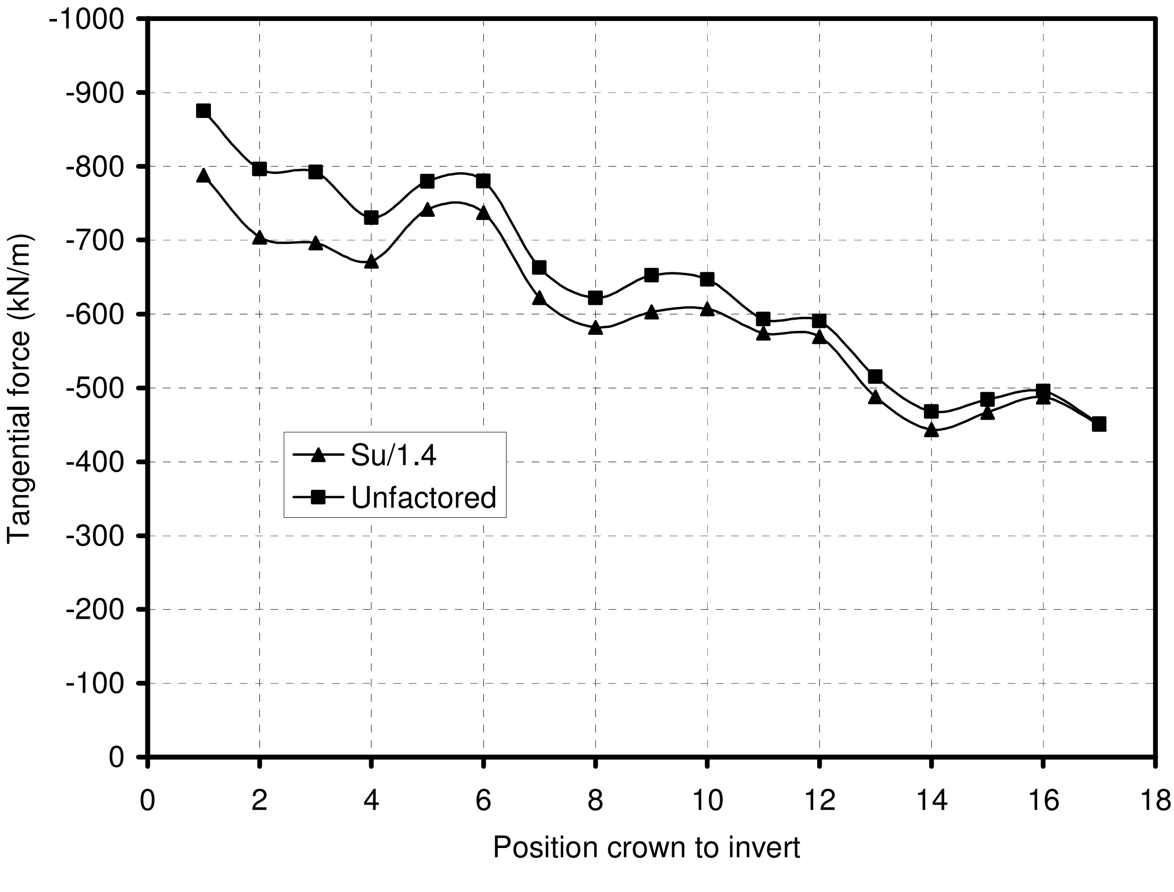

Figure 2: Tangential force for characteristic (unfactored) undrained shear strength S_u and S_u /1.4 (DA1.2) in the pilot tunnel 3D numerical model.

For the shaft construction model, the bending moments were zero. For the tunnel construction, due to the effect of gravity, k_0 and the top heading – bench/invert construction sequence, the bending moments were significant. Figure 3 shows the pilot tunnel bending moments. The worst case bending moments were for the DA1.1 design approach (marked “unfactored*1.35” in Figure 3).

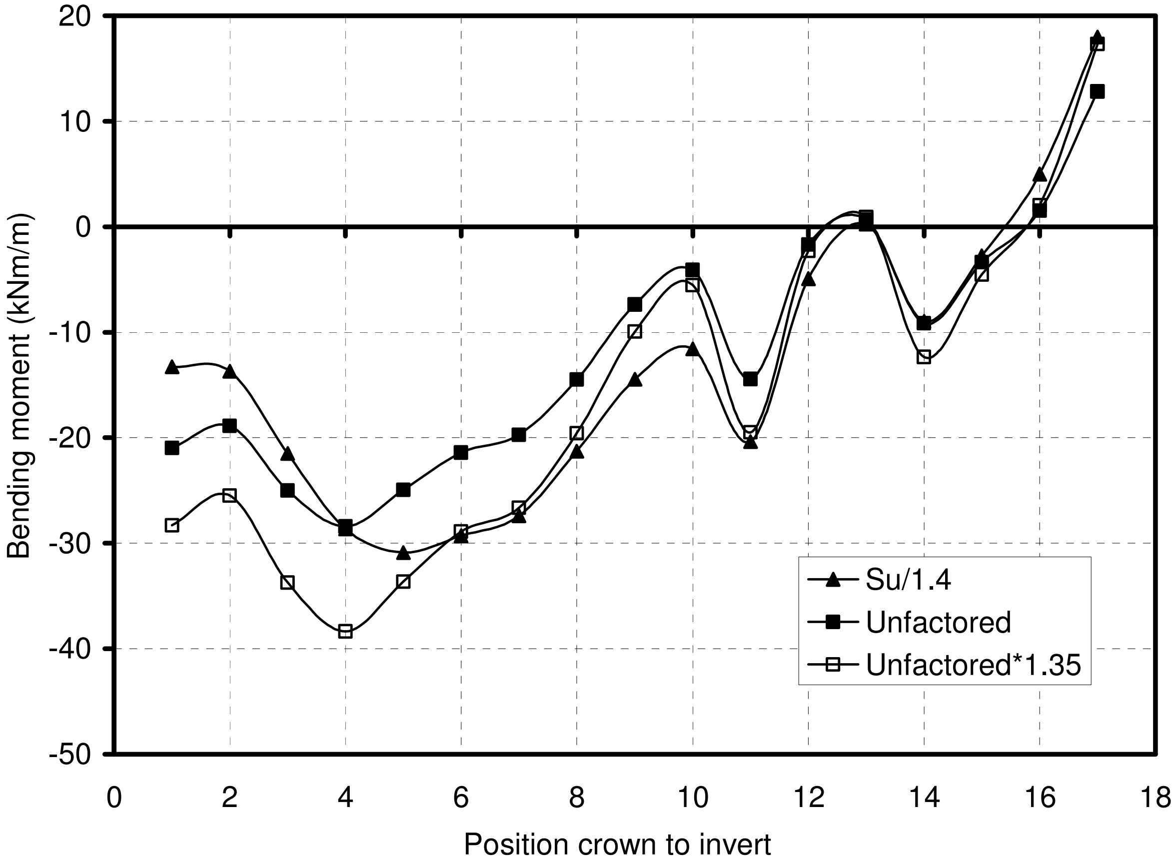

Figure 3: Bending moment in the pilot tunnel 3D numerical model for characteristic (unfactored) undrained shear strength S_u , unfactored S_u multiplied by the factor on actions of 1.35 (DA1.1), and S_u /1.4 (DA1.2).

Figure 4 shows the bending moments in the enlargement tunnel lining. In this case, the worst case bending moments are for the DA1.2 approach. Note that the tangential forces were lower for this case, and tangential force is usually beneficial in the design of relatively shallow tunnels.

Therefore, it may be necessary to model DA1.2 to verify the worst case ULS lining forces. On some projects it may only be necessary to check this for a representative section of tunnel if the geometry and excavation sequence of the tunnels are similar.

Pound [13] found that hand calculations could be used even for non-circular geometries to check face stability, which would be less time-consuming than 3D numerical modelling. A situation where a 3D model may be desirable to check face stability is where there are mixed strata, for instance in the Lambeth Group soils where there are sand channels and clayey sand or silt layers in between very stiff clay layers.

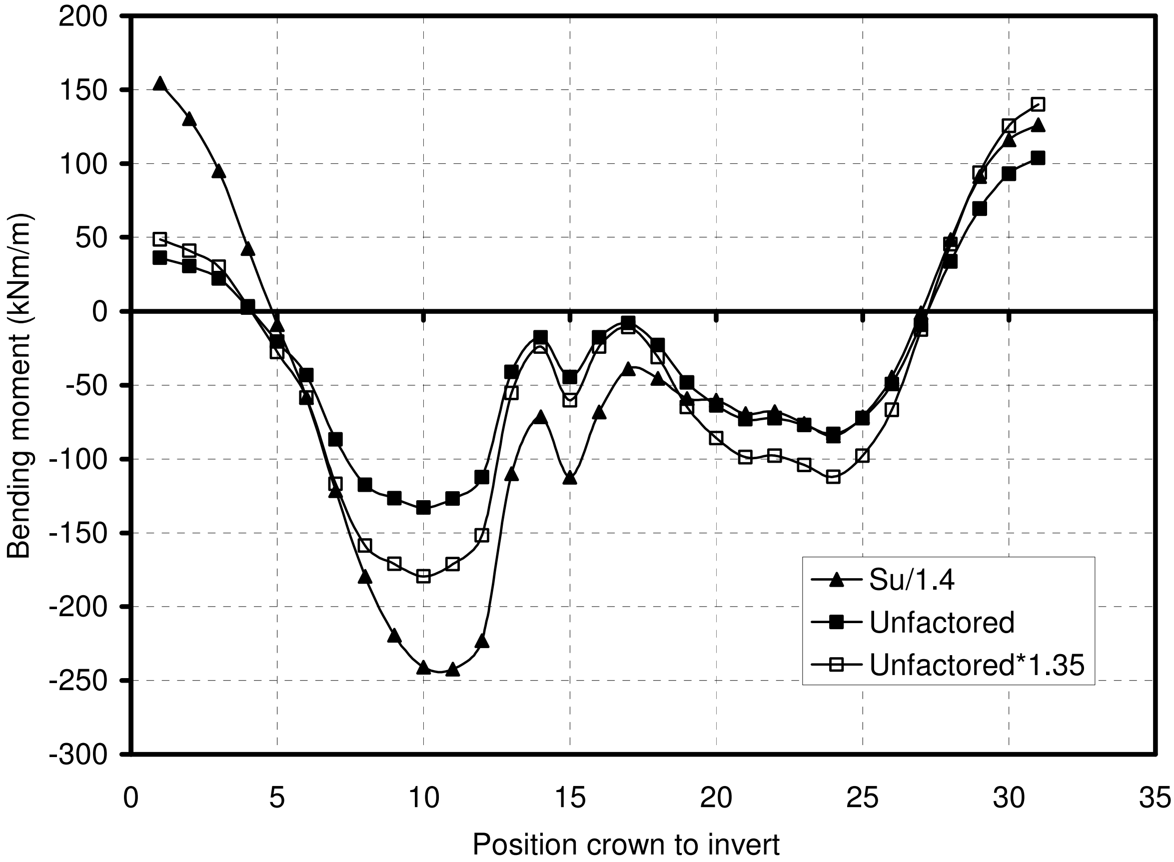

Figure 4: Bending moment in the enlargement tunnel 3D numerical model for characteristic (unfactored) undrained shear strength S_u , unfactored S_u multiplied by the factor on actions of 1.35 (DA1.1), and S_u /1.4 (DA1.2).

4.2 Eurocode 2 creep coefficients

Eurocode 2 allows creep to be calculated. It was decided to verify that these creep coefficients were reasonable for sprayed concrete and to decide on what values to use to reduce the sprayed concrete elastic modulus in the numerical models.

In a numerical model, only the creep during the time period of the current load increment can be assumed. This is because really what is occurring is relaxation – we are hoping that a more flexible lining will result in lower loads. But if the load decreases with time due to creep, it had to at some point sustain that load. Hence during the time period of one excavation stage, over which the load is increasing gradually in reality but is assumed to occur instantaneously in a numerical model, it is reasonable to assume that creep will have time to interact with the application of the load during this time.

Thomas [8] defined an age-dependent generalised Kelvin model for uniaxial creep based on a literature review of creep tests on sprayed concrete samples.

$$\varepsilon_{xx} = \frac{\sigma_{xx}}{9K} + \frac{\sigma_{xx}}{3G} + \frac{\sigma_{xx}}{3G_k}\left(1 - e^{-G_k t/\eta}\right) \tag{1}$$

where t is the age of the sprayed concrete in hours,
σ_{xx} is the uniaxial stress in kPa,
K is the elastic bulk modulus in kPa,

G is the elastic shear modulus,

$$\eta = \frac{1.5 \cdot 10^{11} \cdot e^{\left(\frac{-1.5}{t^{0.6}}\right)}}{2(1+\nu)} kPa.s$$ is the Kelvin viscosity parameter and

$$G_k = \frac{8.0 \cdot 10^{6} \cdot e^{\left(\frac{-1.0}{t^{0.4}}\right)}}{2(1+\nu)} kPa$$ is the Kelvin spring stiffness, and in both these last two equations t is the age in days.

The aim was to compare this relationship based on a literature review of sprayed concrete creep tests to the Eurocode 2 equations.

An SCL thickness of 0.3m was assumed, and for the Eurocode 2 equations cement class N, compressive cylinder strength of 30 MPa and relative humidity of 80% were assumed. Eurocode 2 will provide a creep coefficient for a time period between an age at loading t_0 and the age at the time considered t. An equivalent time relationship based on compressive strength was used to ensure that the accelerated sprayed concrete and the unaccelerated Eurocode 2 concrete were at equivalent stages of hydration. In effect, the values of t and t_0 used in the Eurocode 2 equations were altered by considering the equivalent age the Eurocode 2 concrete would need to be to have the same strength as accelerated sprayed concrete. The strength gain of accelerated sprayed concrete was taken from Chang & Stille [11].

For successive 1 day periods, the creep coefficients using the generalised Kelvin model fitted to the test data and the Eurocode 2 equations are shown in Table 1 below.

t_0	t	$\varphi(t_0, t)$ test data	$\varphi(t_0, t)$ Eurocode 2
0	1	0.795	0.539
1	2	0.406	0.274
2	3	0.329	0.245
3	4	0.295	0.227
4	5	0.276	0.220
5	6	0.262	0.214
6	7	0.253	0.208
7	8	0.246	0.204
8	9	0.240	0.201

Table 1: Comparison of Eurocode 2 and test data creep coefficients.

From Table 1 it can be seen that there is reasonable agreement between the two methods, and the Eurocode 2 creep coefficients could be used for sprayed concrete, providing an equivalent time method is used to account for the faster hydration of accelerated sprayed concrete. The elastic modulus of the sprayed concrete was divided by 1.5 to allow for creep in the numerical models. This accounted for the fact that much of the loading was occurring at early age and also allowed for some ongoing creep from previous load increments.

4.3 Eurocode 2 - factors on bending moments and tangential forces?

For tunnel design in soft ground at relatively shallow depths (< 40 m), tangential forces in the tunnel lining are usually beneficial, due to the shape of the moment-force interaction curve (Figure 5). Therefore, it is not clear that the factors on the actions should be applied to both the axial forces and the bending moments. The phenomena that cause bending moments to occur in SCL tunnel linings, for example anisotropy of the soil, non-circular geometry, interaction with other structures and sequential excavation, are not necessarily related directly to the phenomena that cause tangential forces. Bending moments could be increased without the tangential forces increasing at the same time.

An example of an interaction curve for a 400 mm thick C30 concrete tunnel lining with two layers of 8 mm mesh at 150 mm centres is shown in Figure 5. The plotted points were extracted from a 3D model of an 11 m wide and 9 m high platform tunnel. The combination of a factored bending moment with an unfactored tangential force brings the points closer to the capacity of the tunnel lining.

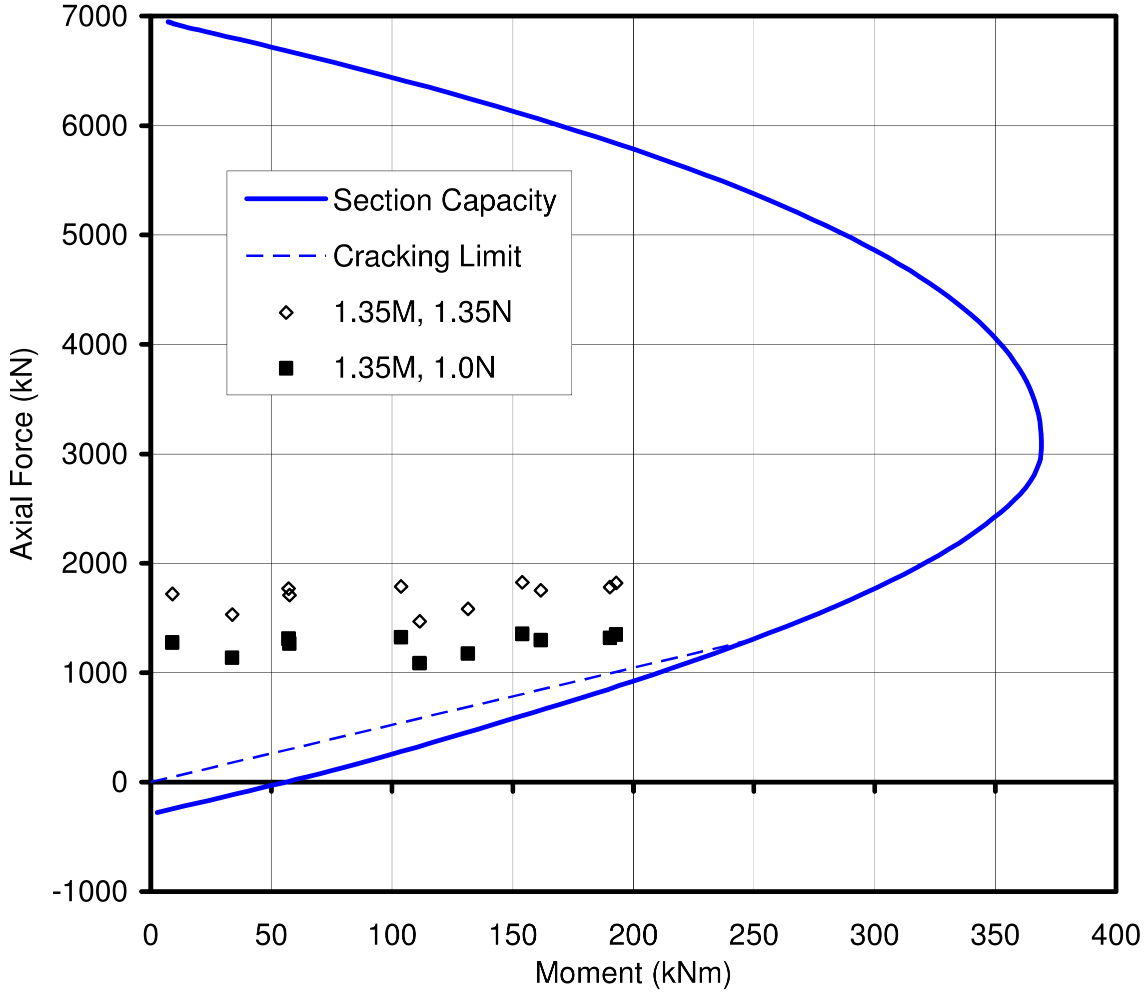

Figure 5: Moment-force interaction according to Eurocode 2

5 CONCLUSIONS

- The level of deformation required to mobilise a 'ground arch' in soft ground is very small compared to a tunnel in rock, and for an open-face tunnel there is no benefit to be gained by actively allowing any additional deformation to occur.
- The use of the observational method is limited in soft ground because there is too much uncertainty about the amount of time available to respond to instability.
- Calibration of a 2D model must be treated with care because the inherent weakness of 2D modelling of tunnels is that it is at best semi-empirical and inductive and at worst wholly a matter of judgement with just the illusion of a deductive scientific approach.

- Combinations 1 and 2 of Design Approach 1 in Eurocode 7 should both be checked to find the worst case. Usually low tangential forces coupled with high bending moments are the worst case in shallow soft ground tunnels. Where bending moments are unimportant, only DA1.1 needs to be modelled.
- The creep coefficients in Eurocode 2 may be used for sprayed concrete, as long as an equivalent time is used to account for the accelerated hydration.
- Bending moments and tangential forces may occur independently and so all combinations of factors on actions should be verified.

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