A Loaded

Benoît Jones, Principal Teaching Fellow & Tunnelling and Underground Space MSc Course Manager, University of Warwick, UK, begins his series of articles reflecting on current tunnelling research, this month looking at tunnel linings

TUNNEL LININGS ARE designed to support the ground. When using Eurocodes, this is done by assuming 'ultimate limit states' – one or more states of failure where the lining will collapse due to shear, bending, punching, crushing or bursting. These ultimate limit states are then compared to the 'resistance', or capacity, of the structure. For the structure to be considered a safe design, the resistance has to be higher than the ultimate limit state.

In order to calculate the ultimate limit states, it is usual to calculate the loads in the tunnel lining using conservative values of geotechnical parameters, and then to apply a partial load factor (e.g. multiply the loads by 1.35 for permanent loads) to allow for unfavourable deviations from the calculated loads, uncertainty in the modelling of the loads and dimensional variation. In Eurocode parlance this is Design Approach 1, Combination 1 (or DA1-1 for short). These conservative values of geotechnical parameters are referred to as 'characteristic values' in Eurocode 7, and are defined as "a cautious estimate of the value affecting the occurrence of the ultimate limit state".

Alternatively, the characteristic values of the geotechnical strength parameters could be reduced by a partial factor (e.g. divide undrained shear strength by 1.4 or divide drained cohesion and the tangent of the angle of friction by 1.25) and in this case the loads would not be factored. This is Design Approach 1, Combination 2 (or DA1-2 for short).

For the structural design of a tunnel lining, DA1-2 usually results in a less onerous situation because the stress in the lining is relatively weakly coupled to soil strength and lining stresses will not usually increase by more than a factor of 1.35 when the strength parameters of the soil are reduced by a partial factor.

The resistance of the lining is calculated based on conservative values of material parameters reduced by a partial material factor, for example for concrete the characteristic strength would be divided by 1.5.

There are other design approaches in Eurocode 7, but only DA1 is allowed in the UK. This combination of using conservative parameters and partial factors means that the likelihood of the ultimate limit state collapse occurring should be very small indeed. And this is as it should be!

Design approach

Design of tunnels may be approached in various ways. The most common in use today are:

- 2D plane strain analytical solutions
- 2D plane strain numerical modelling
- 3D numerical modelling

Since 2D methods do not explicitly take account of relaxation of the ground ahead of the face, an analytical or numerical model will always overestimate the loads and underestimate ground movements. Therefore, if we would like a more efficient design, an estimate of the relaxation prior to installation of the lining can be made. This estimate of relaxation may be based on empirical evidence; for instance, by trying to mimic the pattern and magnitude of ground movements observed around tunnels constructed in the past using similar methods in similar geology. This approach is inductive – to a certain degree we are telling the computer the answer we want - and so this should always be applied with caution to new situations.

3D numerical modelling does take construction sequence into account explicitly, and so one might expect a more realistic answer. However, there are still many assumptions made about the ground behaviour, which are usually based on laboratory testing of soil samples and perhaps some in situ tests. In some cases, back-analysis of previous tunnels constructed using similar methods in similar geology is used to 'calibrate' the ground parameters. In this case, the ground parameters obtained may be thought to be realistic, at least for that tunnel at that location, but are they 'cautious estimates'?

Also, adjusting ground parameters with the aim of approximating ground movements from case histories is difficult when there are several parameters to consider, which will include the value of K_0 (the ratio of horizontal to vertical effective stress), at least 2 pre-yield stiffness parameters, parameters to define stiffness anisotropy if included, and the parameters that define the failure criterion. There are also myriad constitutive models (equations that govern the stress-strain behaviour of the ground) that may be used.

Even if the designer manages to choose a coherent and realistic set of ground parameters that can approximate the settlements measured on a previous project, there is still considerable uncertainty.

The most obvious objection is that there may be several combinations of ground parameters and several different modelling methods that can approximate the settlements reasonably well. For instance, Negro & de Quieroz (2000) reviewed 65 papers on numerical modelling of tunnels where a large number of different modelling methods were used. Of the 55 papers that compared predicted maximum settlement with measured values, 39 of the predictions were within ± 10%. Shirlaw &

question

Wen (2005) pointed out that natural variations in settlements from one array to the next were usually much greater than ± 10%, so very few predictions should be in that range. Thus the excellent 'predictions' in these papers are not evidence of geotechnical science reaching a level of accuracy that will see insurance premiums tumbling, rather, it shows something rather more interesting as we will see.

The three reasons?

Shirlaw & Wen put this down to three reasons. First of all, there is probably a publication bias; people are unlikely to publish bad predictions of ground movements. Secondly, there is the possibility that 'predictions' were made after the event and geotechnical parameters were tweaked to match the field data. Lastly, the 'representative' settlement values used for comparison with the numerical modelling may have been carefully selected.

Treating these predictions not as predictions then, but as back-analyses, Negro & de Quieroz's study could be used to highlight another issue; that a large number of different numerical modelling methods can replicate settlement data observed in the field apparently to the satisfaction of the designers.

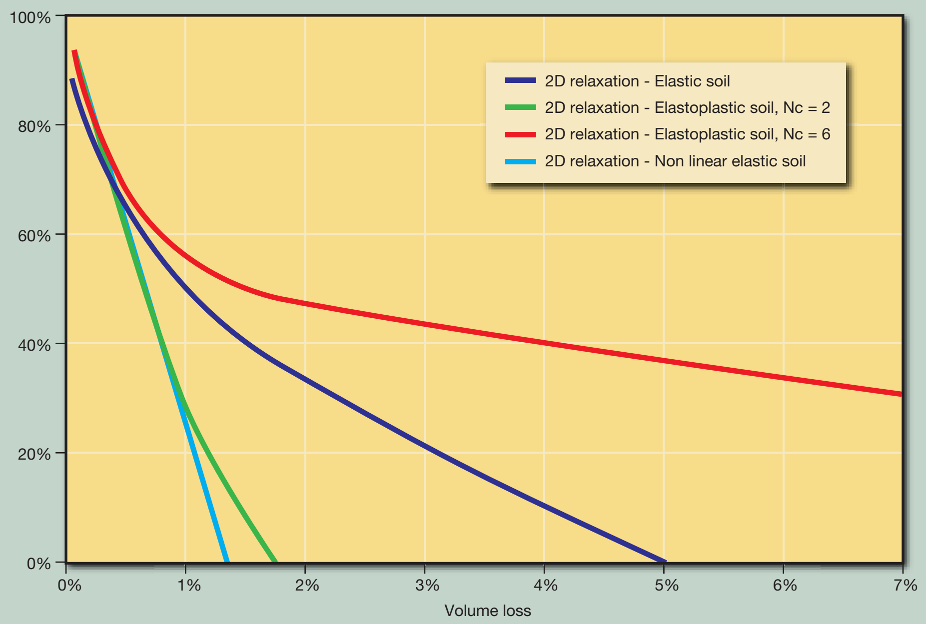
There is also the problem of trying to match not only the maximum settlement over the tunnel centreline, but also the rest of the surface settlement trough. Franzius et al. (2005) found that in London Clay, using the most realistic soil model they could achieve using state-of-the-art laboratory test methods and modelling the advancing tunnel in 3D, it was still not possible to replicate the shape of the settlement trough. This finding has been replicated by others using similarly sophisticated models. For example, Jones et al. (2008) also found the predicted settlement trough to be significantly wider and shallower than measured values. This indicates that there is something about the ground mass behaviour around a tunnel that is not being picked up in in situ or laboratory soil tests and not being included in numerical models, or that there is something inherently wrong with the way that we are testing the soil to obtain parameters.

Another objection to this design methodology is that we are calibrating the model to give a cautious, but realistic value of maximum surface settlement or volume loss, but we are primarily using it to determine the stresses in the tunnel lining. The problem with this is that different soil models will result in very different stresses in the tunnel lining for the same value of maximum settlement or volume loss. So if a large number of different modelling methods can replicate ground movements apparently to the satisfaction of the designers, they will most likely be coming up with very different stresses for use in the structural design of the lining. Negro & de Quieroz (2000) noted that differences between calculated lining stresses and measured lining stresses are frequently attributed to unrepresentative or erroneous field measurements rather than inadequacy of the model. Which is convenient, but not necessarily true.

A very simple example comparing an elastic tunnel lining in linear elastic soil to the same tunnel in nonlinear elastic or linear

"When tunnels are designed there still remains considerable uncertainty surrounding how much load is applied by the ground to the tunnel lining, even when sophisticated numerical models are used with soil parameters based on state-of-the-art laboratory tests and back-analysis of case histories"

Figure 1: The effect of choice of soil model on lining load



elastic-perfectly plastic soil is shown in Figure 1.

The tunnel axis is at 22.5m depth and the soil assumed undrained and homogeneous with a unit weight of 20kN/m^3 and a Young's modulus of 80MPa. The 'full overburden pressure' is therefore 450kPa.

There are two elastoplastic models shown on the figure: one where the undrained shear strength is set to 225kPa, such that the stability ratio $N_c = 2$ and the ground should be very stable, and one where the undrained shear strength is set to 75kPa, such that the stability ratio $N_c = 6$ and the ground will fail if unsupported. These two values of stability ratio probably bracket most situations in clay, since at one extreme the tunnel is at the limit of stability (considering that a safety margin is required) and at the other extreme there is very little plasticity and the soil is behaving almost elastically. There are also two elastic models shown in the figure: one linear and one nonlinear, which was based on small-strain stiffness data from the Heathrow Terminal 5 project (Hight et al., 2007).

If data from case histories indicate that a cautious estimate of short-term volume loss is 1.0%, then the elastic model will result in the lining supporting 24% of the full overburden pressure, the nonlinear elastic model will result in the lining supporting 51%, the elastoplastic model with $N_c = 2$ will result in the lining supporting 28%, and the elastoplastic model with $N_c = 6$ will result in the lining supporting 60%. One can immediately see that small changes in the target value of volume loss or small changes in the soil model will result in substantial changes to the short-term lining load used to design the tunnel lining.

Critical bending moments

So far we have looked at radial ground pressure as though it were applied evenly around the tunnel. However, for designing the thickness of the lining and the amount of reinforcement required, it is usually the bending moments that are critical, not the hoop thrust. In fact, for shallow tunnels the hoop thrust will actually be beneficial to the designer of a concrete tunnel lining, as compressive stress will increase the concrete section's bending capacity.

As an aside, it is probably a good idea to check the tunnel lining for a minimum, unfactored hoop thrust coupled with a factored bending moment, since this may well be the worst case and it can't be reasonably argued that they are co-dependent.

Bending moments are generated in a lining by non-circular shape, eccentricities of load across joints (in a segmental lining), by the construction sequence and, most importantly, by stress anisotropy in the ground. We are not so much concerned with stress anisotropy in the ground prior to construction (defined by coefficient of earth pressure at rest, K_0) as what is the stress regime after construction that imposes unequal stresses on the lining.

Therefore, in a 2D model, a further assumption has to be made about how stress anisotropy is relaxed before the lining is installed. In a 3D model, stress anisotropy in the soil can be allowed to adjust itself as the tunnel is constructed, which would appear to be better, but this then depends on the parameters chosen for stiffness anisotropy.

In conclusion

To summarise the preceding arguments, when tunnels are designed there still remains considerable uncertainty surrounding how much load is applied by the ground to the tunnel lining, even when sophisticated numerical models are used with soil parameters based on state-of-the-art laboratory tests and back-analysis of case histories. To address this uncertainty it would seem sensible to go out and measure loads in tunnel linings much more systematically than we do.

REFERENCES

Franzius, J. N., Potts, D. M. & Burland, J. B. (2005). The influence of soil anisotropy and K0 on ground surface movements resulting from tunnel excavation. Géotechnique 55, No.3, 189-199.

Jones, B. D., Thomas, A. H., Hsu, Y. S. & Hilar, M. (2008). Evaluation of innovative sprayed-concrete-lined tunnelling. Proc. Instn. Civ. Engrs. Geotech. Engrg. 161, GE3, June, 137-149.

Hight, D. W., Gasparre, A., Nishimura, S., Minh, N. A., Jardine, R. J. & Coop, M. R. (2007). Characteristics of the London Clay from the Terminal 5 site at Heathrow Airport. Géotechnique 57, No.1, 3–18.

Negro, A. & de Queiroz, P. I. B. (2000). Prediction and performance: a review of numerical analyses for tunnels. Proc. Int. Symp. Geotechnical Aspects of Underground Construction in Soft Ground (eds. O. Kusakabe, K. Fujita and Y. Miyazaki), Tokyo, Japan, pp.409-418. Rotterdam: Balkema.

Shirlaw, J. N. & Wen, D. (2005). Checking and reviewing the output from numerical analysis. Proc. Underground Singapore 2005, Singapore, pp.241–249.