Numerical modelling for shallow tunnels in weak rock

In designing very shallow tunnels the proximity of the ground surface and the tendency for the material ahead of and above the tunnel face to “cave to surface” have to be taken into account. These factors introduce issues that are not present in the analysis of deep tunnels and approaches such as the convergence-confinement method are not applicable to shallow tunnels. By means of an example, some of the problems of numerical modelling for shallow tunnels are discussed and practical solutions to these problems are presented.
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Introduction

When designing a shallow tunnel in a poor quality rock mass, the designer has to face a number of problems that do not exist or are less significant in deeper tunnels.

1. The proximity of the ground surface usually means that the preferred failure path for the rock mass surrounding the tunnel and ahead of the face is to ‘cave’ to surface. This is a different failure process from the ‘squeezing’ that occurs around a deep tunnel in a weak rock mass and any analysis employed must be capable of accommodating this difference.

2. Because of the different failure process, the conventional ‘rock support interaction’ or ‘convergence-confinement’ tunnel design process cannot be applied to this problem. Traditional approaches for tunnels at shallow depth usually involve the assumption that the ‘rock load’ is calculated on the basis of the dead weight of the rock mass above the tunnel.

3. In weak rocks the stability of shallow tunnels usually involves instability of the face as well as failure in the rock mass surrounding the tunnel. Consequently, a complete analysis of this problem requires the use of a full three-dimensional numerical model. While such models are available, they are not user-friendly for the average tunnel designer and it is therefore necessary to make some rational approximations that can be used in more commonly used two-dimensional analyses.

4. Near surface rock masses are subject to stress relief, weathering and blast damage as a result of nearby excavations. These processes disrupt or destroy the interlocking between rock particles that plays such an important role in determining the overall strength and deformation characteristics of rock masses. Near surface rock masses tend to be more ‘mobile’ than similar rock masses in the confined conditions that exist at greater depth. This greater mobility must be recognised by the designer and allowed for in the selection of input parameters for any analysis.

These problems are examined in the text that follows. A practical tunnel design example, utilising the program Phase2\(^1\), is presented. This does not imply that this program is the only one available or that the author insists upon its use. There are many excellent two- and three-dimensional numerical programs available and the user has to choose the one that is most appropriate.

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Example problem

Problem definition

Consider the problem defined in Figure 1 in which a 12 m span tunnel is to be excavated by top heading and bench methods at a depth of about 15 m below the surface. The parallel highway is to be placed on a cut, the toe of which is about 38 m from the tunnel boundary.

In order to understand the processes involved in excavation of the highway cut and the subsequent tunnel excavation, a simple model is constructed with no support in the tunnel. This model is useful to give a view of the overall magnitude of the problem and to give a first estimate of the forces to be carried by the support systems.

![Figure 1: Model of unsupported tunnel adjacent to a cut slope.](image-url)
The rock mass properties assumed for this analysis, based on Hoek, Carranza-Torres and Corkum (2002)\(^2\), are:

- Geological strength index GSI = 25
- Hoek-Brown constant \(m_i\) = 8
- Intact rock strength \(\sigma_{ci}\) = 3 MPa
- Disturbance factor \(D\) = 0.3
- Material constant \(m_b\) = 0.342
- Material constant \(s\) = 0.0001
- Material constant \(a\) = 0.531
- Deformation Modulus \(E\) = 308 MPa.

In choosing these properties the value of the Disturbance Factor \(D\) = 0.3 to allow for the near surface loosening described earlier.

Note that the Hoek-Brown criterion has been used directly in this analysis. There is no direct theoretical relationship between the Hoek-Brown failure criterion and the Mohr Coulomb criterion and any attempt to estimate the equivalent cohesion and friction values for the highly variable gravitational stress field in this near-surface problem will be inaccurate. On the other hand, the curvilinear Hoek-Brown failure criterion automatically accommodates the changing stress field. If the reader wishes to use the Mohr Coulomb criterion, this should be used directly with the cohesive strength and angle of friction determined from tests or from some method that does not depend upon a correlation with the Hoek-Brown criterion.

**Method of analysis**

The steps in the analysis are as follows:

1. **Stage 1** – The complete model, with a horizontal top surface, is allowed to consolidate with no excavations present. The vertical stress is assumed to be due to gravity and it is calculated as the product of the depth below surface and the unit weight of the rock mass. For want of any better information, the horizontal stresses are assumed to be equal to the vertical stress. In cases of large topographic relief or where large tectonic forces have been active, some modification to the lateral stress assumption may be required. For example, in a tunnel parallel to a steep valley side, the horizontal stress acting on the tunnel will have been relieved due to down-cutting of the valley and it may be appropriate to use a lateral stress of less than the vertical stress. In other cases, for example when the tunnel is in the close proximity of a major fault, an increase in the horizontal stresses may be appropriate. In the example considered here a horizontal to vertical in situ stress ratio of 1.0 is assumed. The

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stresses in the rock mass surrounding the tunnel will be modified by the excavation of the adjacent road cut.

2. Stage 2 - The original ground surface is ‘excavated’ to represent the formation of the original slope. In this case it has been assumed that this original slope was created by erosion and the profile, as illustrated in Figure 1, is relatively gentle.

3. Stage 3 - The cut for the second carriageway is excavated. If the rock mass is very weak and the cut design is inappropriate, slope failures may occur and these will need to be rectified before excavation of the tunnel proceeds. This would normally be the subject of a separate analysis using limit equilibrium slope stability analysis tools such as the program Slide³.

4. Stage 4 - The top heading of the tunnel is excavated. Note that, in the analysis being considered here, no support is installed.

5. Stage 5 - The tunnel bench is excavated.

**Rock mass behaviour for an unsupported tunnel**

The results of this analysis are summarised in Figure 2 that shows displacement vectors in the rock mass surrounding the tunnel. Failure extends to the surface and the direction of the displacement vectors indicates that a ‘caving’ process has developed. Under these circumstances arching of the rock mass above the tunnel cannot be relied upon and the support will have to be designed to carry the dead weight of the overlying rock. The support pressure required to carry the weight of the failed rock above the tunnel is approximately 0.4 MPa and this will have to be provided by means of a passive support system such as steel sets embedded in shotcrete. Rockbolts are not capable of developing this support pressure and it is questionable whether they could be anchored effectively in this type of rock mass.

**Considerations of face stability**

The results summarised in Figure 2 are for a two-dimensional section through a long tunnel. This analysis gives no indication of the stability of the face which, in weak rocks, is of as much or more concern than the stability of the tunnel perimeter. Unfortunately, there are no simple methods available at present that will permit the complex three-dimensional behaviour of the rock mass ahead and around the tunnel face to be investigated. The program Phase2 has an axi-symmetric option that can be used to investigate simple three-dimensional problems. However, this option cannot be used for gravity loaded near-surface excavations and so it is of no value for the example under consideration here.

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³ Available from www.rocscience.com
Figure 2: Results of an analysis in which the excavation of the slope, tunnel top heading and complete tunnel has been simulated. Note that no support has been placed in the tunnel.

a. Displacements in the rock mass after excavation of the cutting are negligible. The cut design is stable for these conditions.

b. Excavation of the top heading of the tunnel, with no installed support, results in ‘caving’ to the surface. The figure shows the displacement vectors (m) in the rock mass overlying the tunnel.

c. Excavation of the bench to form the complete unsupported profile of the tunnel results in additional rock mass failure and collapse of the tunnel.
Full three-dimensional models such as FLAC3D\(^4\) can obviously be used for such investigations but these programs are not readily available to most tunnel designers and their use is best left to experienced analysts. A simplified investigation of the face stability problem can be carried out by considering a two-dimensional cross-section parallel to the tunnel axis. This represents an infinitely long tabular excavation and the results can only be used to obtain an indication of the behaviour of the rock mass surrounding a tunnel. No attempt should be made to utilize the results of such an analysis for detailed design of the excavation sequence and support systems.

Figure 3 shows part of a Phase2 finite element model in which the advancing tunnel is represented by a horizontal slot. The 12 m tunnel is driven with 3 m advances and tractions are applied to represent the installed support. The vertical stress due to 15 m of cover is approximately 0.4 MPa. No support has been installed within 3 m of the face. The support pressure from 3 to 6 m is 0.2 MPa and this is approximately the support that would be provided by a layer of shotcrete. With the addition of steel sets, the support pressure from 6 to 9 m is assumed to be 0.3 MPa. Finally, embedding these sets in shotcrete gives the support pressure of 0.4 MPa at 9 m behind the face.

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4 Available from ITASCA Consulting Group Inc., Thresher Square East, 708 South Third Street, Suite 310, Minneapolis, Minnesota 55415, USA, Fax +1 612 371 4717
Figure 4: Displacements in the rock mass above and ahead of the advancing tunnel face.

Figure 4, while a very crude approximation, indicates that face stability is likely to be a significant problem in this case. Consequently, consideration will have to be given to some form of face support in addition to the support provided for the rock mass.

Face support options

The first question to be addressed in deciding upon the types of support to be used for this tunnel is how to deal with the instability of the face. The available options are:

1. Reducing the area of the face by using multiple drifts and ensuring that each face is stable before the next drift is excavated. A typical arrangement for advancing the tunnel in this manner is illustrated in Figure 5. This method works well and there are many variations on the number, size and sequence of excavation of the multiple drifts. For the sake of brevity details of this approach will not be discussed further in this document.
2. Advancing the tunnel under a forepole umbrella as illustrated in Figure 6. In a 12 m span tunnel of the type being considered here, the method would typically involve installing 12 m long 114 mm diameter grouted pipe forepoles at a spacing of 300 to 600 mm. These forepoles would be installed every 8 m to provide a minimum of 4 m of overlap between successive umbrellas. A first step in this method may involve drilling holes, up to 30 m ahead of the face, for drainage. This is followed by the drilling of the 12 m long holes and installation of the pipe forepoles to form the umbrella arch. In some cases, depending upon the nature of the rock mass being tunnelled through, jet-grouted forepoles are used rather than the grouted pipe forepoles. These jet grouted forepoles can be very effective in low cohesion frictional materials such as weathered granites. Note that the installation of 12 m long grouted pipe or jet grout forepoles requires the use of a special drilling rig. An alternative approach, favoured by many designers, is to use 6 m long 50 mm diameter grouted pipe forepoles which can be installed by means of a conventional jumbo. Obviously the spacing between the forepoles and the advance between successive forepole umbrellas must be smaller in this case but the overall cost is generally lower than for the 12 m long forepoles.
3. In cases of relatively minor face instability it may be sufficient to grout fibreglass dowels into the face. These have the advantage of being lightweight and very strong and they are also easily cut and disposed of as the face advances. However, this system can only be used when the face is sufficiently stable to allow the grouted dowels to be installed. In many cases this system is used to supplement the face support provided by the partial face or forepoling methods described above.

In some situations all of these methods are used and an example of such an application is shown in Figure 10. The photograph illustrates the top heading excavation for an underground station in the Metro do Porto project on Portugal. The side drift on the right hand side of the excavated is being advanced under an umbrella of 12 m long grouted forepoles and the face is stabilized by grouted fibreglass dowels. A temporary invert in placed as close to the advancing face as possible in order to provide a closed structural shell. The rig shown in the left hand side of the photograph is used for installing the jet grouted forepole umbrella. Benching, with the provision of a closed invert, follows and a final concrete lining is placed inside the stabilized excavation.
Analysis of face support

Analysis of the support provided by systems such as forepoles is even more difficult than the analysis of face stability described earlier. A full solution requires the use of a program such as FLAC3D but such programs are seldom used for routine tunnel design. Consequently, it is worth considering whether two-dimensional models such as Phase2 can provide any guidance on this complex issue.

There are no general rules currently available for the support provided by forepoles and, in the absence of such rules, a crude equivalent model is used in this analysis. This assumes that a process of weighted averages can be used to estimate the strength and deformation of the zone of ‘reinforced rock’. For example, the strength is estimated by multiplying the strength of each component (rock, steel and grout) by the cross-sectional area of each component and then dividing the sum of these products by the total area. In this case, the steps in the tunnel roof required to install the forepoles are approximately 0.6 m deep and hence we will consider a rock beam 1 m wide and 0.6 m deep. The
forepoles have an outer diameter of 114 mm and an inner diameter of 100 mm and are spaced at 0.5 m. The quantities involved are as follows:

<table>
<thead>
<tr>
<th>Component</th>
<th>Area</th>
<th>Strength</th>
<th>Product</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rock</td>
<td>0.6 m²</td>
<td>0.2 MPa</td>
<td>0.12</td>
</tr>
<tr>
<td>Forepoles</td>
<td>0.005 m²</td>
<td>200 MPa</td>
<td>1.0</td>
</tr>
<tr>
<td>Grout</td>
<td>0.015 m²</td>
<td>30 MPa</td>
<td>0.45</td>
</tr>
<tr>
<td>Sum</td>
<td>0.62</td>
<td>-</td>
<td>1.57</td>
</tr>
</tbody>
</table>

The resulting rock mass strength for this composite ‘beam’ is $1.57/0.62 = 2.5$ MPa. The equivalent rock mass properties can be estimated by iteration of the Hoek-Brown failure criterion (using the program RocLab\(^5\) as follows:

- Geological Strength Index GSI = 25
- Hoek-Brown constant $m_i$ = 8
- Intact rock strength $\sigma_{ci}$ = 29.5 MPa
- Rock mass strength $\sigma_{cm}$ = 2.5 MPa
- Material constant $m_b$ = 0.549
- Material constant $s$ = 0.0002
- Material constant $a$ = 0.531
- Deformation modulus E = 1288 MPa

Note that the Disturbance Factor D = 1 in this case since the forepole umbrella is assumed to be undamaged.

The same model as that illustrated in Figure 1 is used to investigate the effectiveness of a forepole umbrella, represented by a composite beam as described. The forepoles are installed over the crown of the excavation at the same time as the rock mass in the top heading is ‘softened’ (its deformation modulus is reduced 50%) to represent the fact that the face has already reached this point before the forepoles are installed. Excavation of the top heading follows together with softening of the rock mass forming the bench. Finally the bench is removed to create the complete tunnel profile. The results are shown in Figure 11 which shows that caving to surface (as in Figure 2) has been controlled by the installation of the forepole umbrella.

In this model no invert has been provided for the top heading although the lower sidewalls of the top heading have been enlarged to create “elephant feet” to prevent foundation failure. Figure 11a shows that there is significant and uneven heave of the top heading invert and this suggests that the provision of a temporary invert would be prudent. The closure of the invert after excavation of the bench is also required in order to provide a closed load-bearing structural shell.

\(^5\) This can be downloaded free from www.rocscience.com
a. The top heading has been excavated after installation of the forepole umbrella. Note that significant displacement of the top heading invert has occurred but that caving has been controlled.

b. Complete excavation of the tunnel.

Figure 11: Support provided by the forepole umbrella is very effective in protecting the tunnel and controlling surface subsidence (compare with Figure 2). However, there is uneven “heave” of the top heading invert.
b. The top heading with a temporary invert has been excavated after installation of the forepole umbrella.

b. Complete excavation of the tunnel.

Figure 12: Support provided by the forepole umbrella is supplemented with the provision of a 30 cm thick shotcrete temporary invert for the top heading.
Figure 12 shows the results obtained from the same model but with the provision of a temporary invert for the top heading. Typically, this temporary invert would consist of a 30 cm thick layer of wire mesh or steel fibre reinforced shotcrete. Figure 12a shows that the displacements of the top heading invert have been halved, as compared with those shown in Figure 11a, and the displacements are evenly distributed.

This design has not been optimised in any way and the results have been presented to demonstrate that these simple numerical models make it possible to investigate a number of alternatives in a very realistic manner. The reader should not hesitate to explore such options; there is no penalty for getting it wrong and there is always a great deal to be learned in such explorations.

Since the upper surface of the model represents the ground surface, surface displacements or surface subsidence can also be investigated. This is very important in designing shallow urban tunnels, such as metro tunnels, since overlying buildings and services can be adversely affected by excessive surface displacement or tilt. The procedure followed in such cases is the same as that described in the example given earlier but more attention is paid to modelling the ground surface in order to provide the best possible estimates of surface subsidence.

**Excavation sequence**

Before leaving this example it is worth considering the effects of excavation sequence. Figure 13 illustrates the difference between the rock mass behaviour for the cases where the adjacent cutting is completed before excavation of the tunnel and where the tunnel is excavated first. All other conditions are identical and the scales of the vector plots are the same in both figures.

There is clearly a difference between the two cases with the tunnel being dragged towards the open cut excavation in Figure 13b. Both solutions, although they give different results, are acceptable in this example since the tunnel is stable and no caving to surface has occurred. However, in some marginal cases, the difference in excavation sequence can give rise to serious problems in the tunnel.

In particular, the placement of a concrete lining in the tunnel before excavation of the adjacent cut can give rise to damage due to bending of the tunnel. As a general rule, the adjacent cutting should always be completed before the tunnel is excavated. Where this is not possible, the placement of the final concrete lining should be delayed until the excavation of the cut has been completed. The shotcrete primary lining is relatively flexible and is more able to withstand the rock mass deformations than the stiff final lining.
a. Cutting completed before excavation of tunnel

b. Tunnel excavated before cutting of slope

Figure 13: Effects of excavation sequence on the response of the rock mass surrounding the tunnel.