Numerical Modelling for Shallow Tunnels in Weak Rock

By Evert Hoek – Unpublished Notes

Introduction

When designing a shallow tunnel in a poor quality rock mass, the designer has to face a number of problems that either 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 generally 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 authors are insisting 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.

Example problem

Problem definition

Consider the problem defined in Figure 1. 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

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[1] Available from Rocscience Inc., 31 Balsam Avenue, Toronto, Ontario, Canada M4E 3B5, Fax 1 416 698 0908, Email: software@rocscience.com, Internet: http://www.rocscience.com.

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.
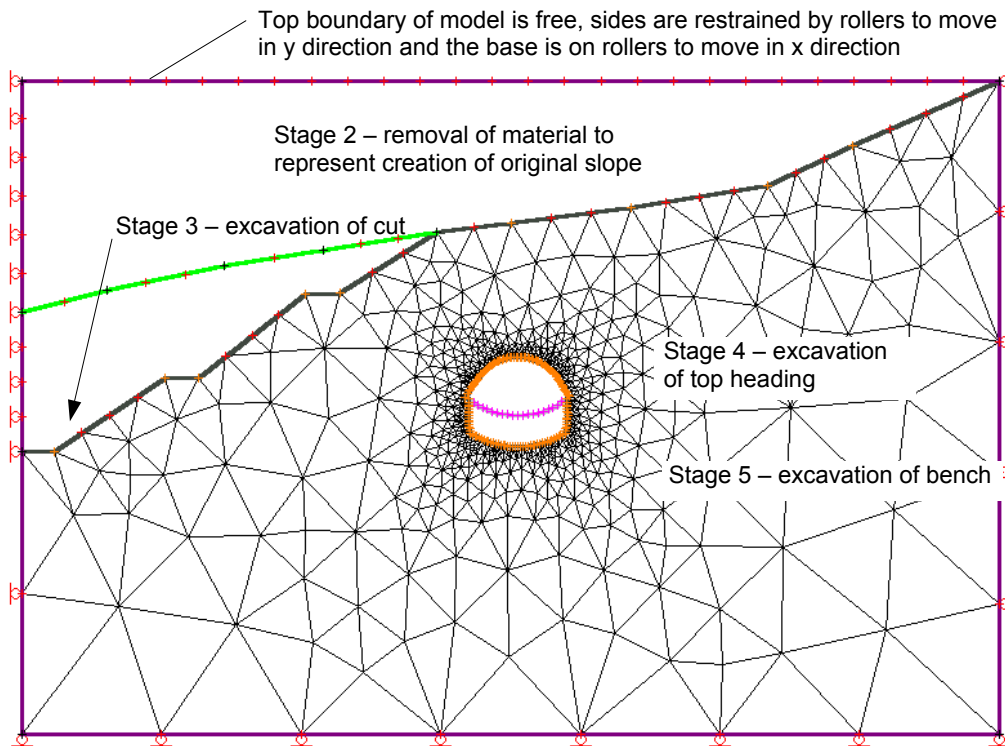

Figure 1: Model of unsupported tunnel adjacent to a cut slope.

The rock mass properties assumed for this analysis, based on Hoek and Brown (1997)[2], are:

> Geological strength index GSI = 20
> Hoek-Brown constant m_i= 8
> Intact rock strength σ_{ci} = 3 MPa
> Friction angle ϕ = 21°
> Cohesive strength c = 0.055 MPa (55 kPa)
> Deformation Modulus E = 308 MPa.

In choosing these properties the designer should use lower bound values of the constants GSI, m_i and σ_{ci} estimated in the field in order to allow for the near surface loosening described earlier.

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[2] Hoek, E., and Brown, E.T. (1997). Practical estimates or rock mass strength. *Int. J. Rock Mech. & Min. Sci. & Geomech. Abstracts*. Vol. 34, No. 8, pp 1165-1186.

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 one half the vertical stress. However, in this case, there would be no reduction in the horizontal stress parallel to the tunnel axis. 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.

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. At this stage, 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.

4. Stage 4 – The top heading of the tunnel is excavated. In this profile illustrated in Figure 1, provision has been made for an 'elephant foot' enlargement at the base of the top heading arch. This is usually required in very weak rock in order to provide a foundation for the arch. 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, which 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 that, 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 tunnels and so it is of no value for the example under consideration here. Full three-dimensional models such as FLAC3D[3] 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 very crude 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 utilise 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 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 4 MPa at 9 m behind the 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.

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[3] Available from ITASCA Consulting Group Inc., Thresher Square East, 708 South Third Street, Suite 310, Minneapolis, Minnesota 55415, USA, Fax + 1 612 371 4717
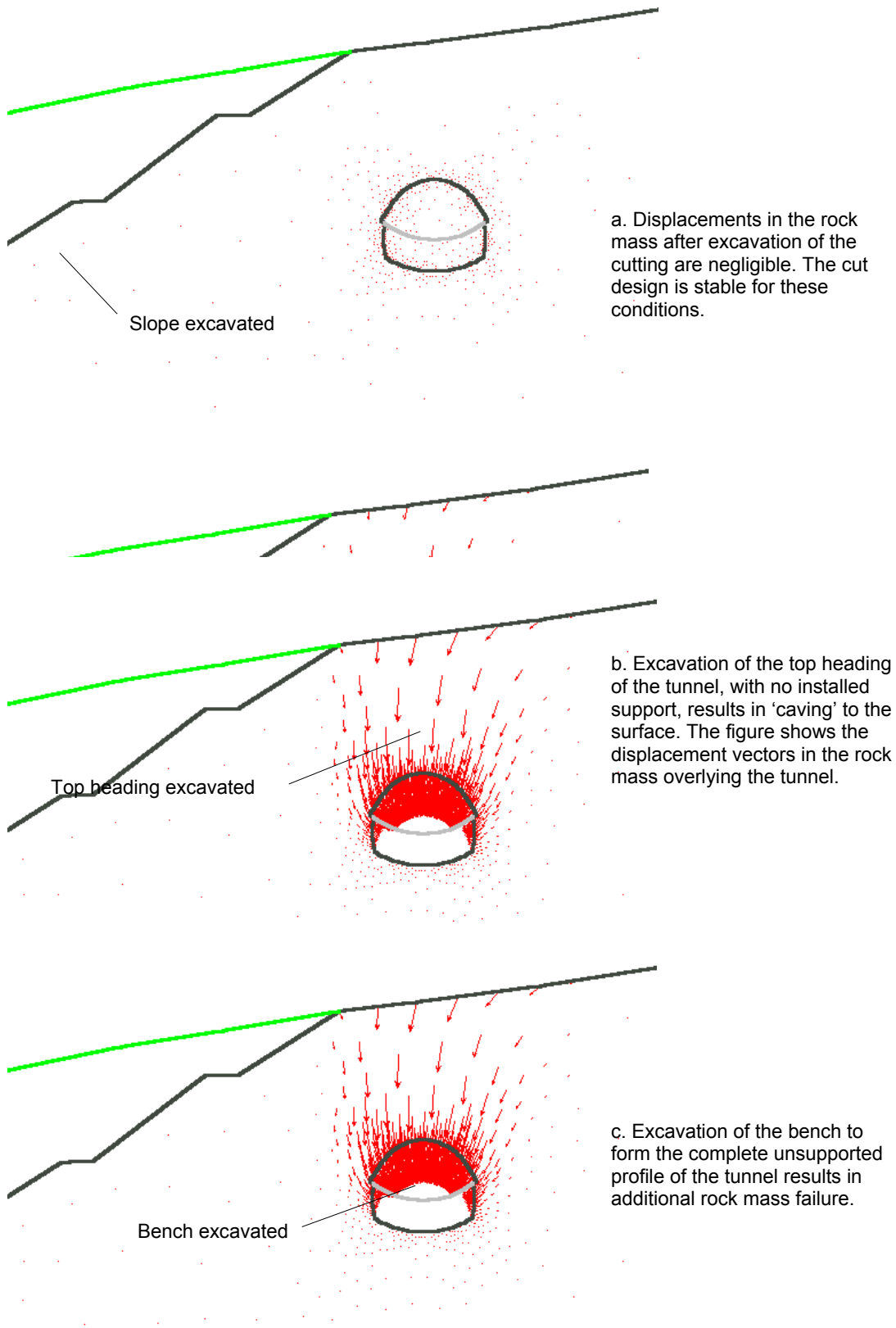
a. Displacements in the rock mass after excavation of the cutting are negligible. The cut design is stable for these conditions.

Slope excavated

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 in the rock mass overlying the tunnel.

Top heading excavated

c. Excavation of the bench to form the complete unsupported profile of the tunnel results in additional rock mass failure.

Bench excavated

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.

Figure 3: Phase2 finite element model of an advancing tunnel face represented by a vertical section along the tunnel axis. No support is installed within 3 m of the face and tractions are applied to the tunnel boundary to represent increasing support pressures behind the face.

Figure 4: Displacements in the rock mass above and ahead of the advancing tunnel face.

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 tends to be slow and expensive and tends to be used for specialised excavations rather than for highway tunnels. It will not be considered further in this article.
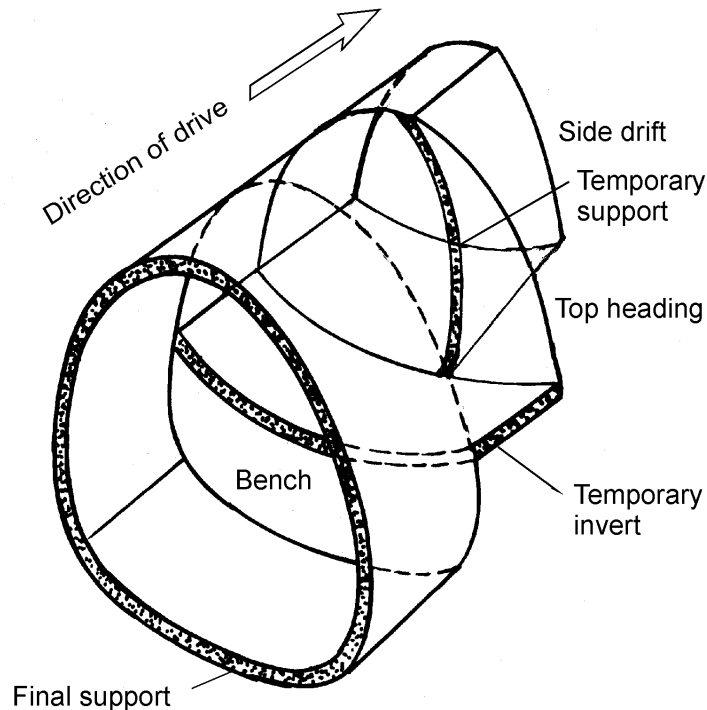

Figure 5: Partial face excavation method using a side drift followed by top heading and bench.

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 columns are used rather than the grouted pipe forepoles. The tunnel is then advanced 8 m before the cycle is repeated to create another protective umbrella.

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.
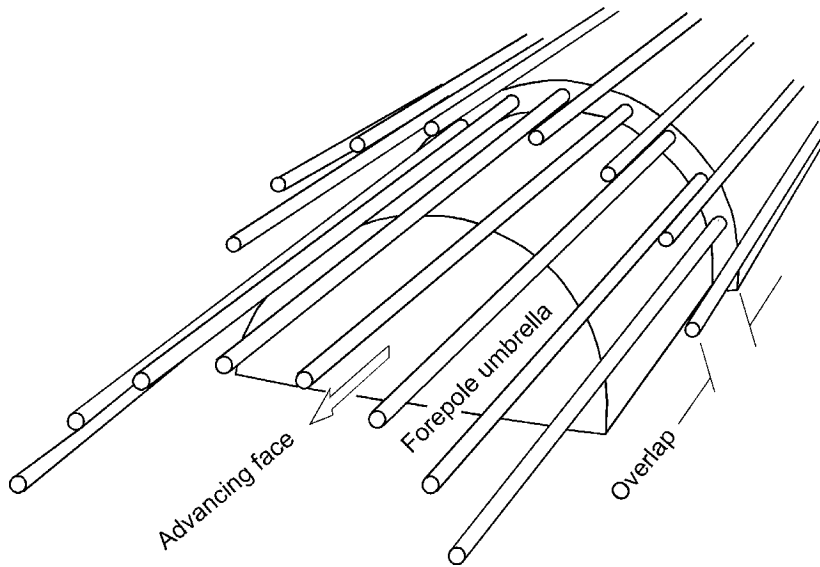

Figure 6: Sketch of tunnelling under the protection of a forepole umbrella.

Analysis of face support

Analysis of the support provided by systems such as forepoles or grouted fibreglass dowels 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.

In order to analyse the support provided by forepoles it is tempting to consider using a two-dimensional model representing a section parallel to the tunnel axis, similar to that illustrated in Figure 3. In such a model it should be possible to model 'forepoles' by inserting elements along the lines of the forepoles. Ideally, 'beam' elements that are capable of simulating bending resistance should be used for this type of analysis. However, it is not at all clear how the properties of these beam elements would be

defined in order to give a realistic simulation of the action of forepoles. Consequently, the reader is advised to forget about this modelling option and to concentrate on the alternative solution proposed below.

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:

Component	Area	Strength	Product
Rock	0.6 m^2	0.16 MPa	0.1
Forepoles	0.005 m^2	200 MPa	1.0
Grout	0.015 m^2	30 MPa	0.45
Sum	0.62		1.55

The resulting rock mass strength for this composite 'beam' is 1.55/0.62 = 2.5 MPa. The equivalent rock mass properties can be estimated from the Hoek-Brown failure criterion as follows:

Geological Strength Index GSI	20
Hoek-Brown constant m_i	8
Intact rock strength σ_{ci}	47 MPa
Rock mass strength σ_{cm}	2.5 MPa
Friction angle ϕ	21 degrees
Cohesive strength c	0.86 MPa
Deformation modulus E	1220 MPa

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.

Figure 7a shows the extent of rock mass failure (denoted by the x and o symbols) as well as the displacement vectors in the rock mass surrounding the tunnel after excavation of the top heading. A significant amount of rock mass failure is evident but the displacements in the rock mass have been controlled to a few centimetres by the

placement of the forepole umbrella. Compare this result with Figure 2b in which the top heading has been excavated without support.

Figure 7b shows that, once the bench is excavated, failure of the forepole umbrella foundations occurs and the arch moves downwards while the lower sidewalls of the tunnel fail inwards. The rock mass caves to surface, as was the case in Figure 2c.

An 'elephant foot' foundation was provided for the arch foundation in the example illustrated in Figure 7. The rock mass forming the lower tunnel sidewalls and the arch foundation is normally strengthened, where required, by jet grouting or the placement of micropiles. These are generally satisfactory procedures but, from a practical construction point of view, they require the use of an additional piece of equipment since the forepoling machine boom is generally too large to drill the downward holes required for jet grouting or micropile placement. An interesting alternative is to use the forepoling machine to install an additional series of horizontal forepoles parallel to the lower tunnel sidewalls[4].

This alternative has been investigated by means of a Phase2 model, using the same geometry and properties as for Figure 1. In this model the 'elephant foot' arch foundation has been eliminated and the forepole umbrella has been extended downward along the lower sidewalls. The results achieved for this design are illustrated in Figure 8 and these should be compared with Figure 7b. It can be seen that the caving to surface has been completely controlled and, although there is still significant failure in the rock mass surrounding the tunnel, the displacements have been controlled to a few centimetres.

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.

Conclusion

Numerical models for use in tunnel design have improved enormously over the past decade and there is little doubt that they will continue to improve. However, even the best models are only approximations of reality and they should never be considered as a replacement for common sense and engineering judgement.

The most effective use of numerical models is as exploration tools in which the influence of different input assumptions and parameters are systematically investigated. Fortunately, most of these models are fast enough that this type of exploration can be carried out very efficiently. Some examples of this type of process are included in this appendix.

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[4] This alternative is based upon a design proposed by Dr Nikos Koronakis of Omikron Kappa Consulting Ltd., Athens.

Top heading excavated

Forepole umbrella

The top heading has been excavated after installation of the forepole umbrella. Some failure of the rock mass surrounding the tunnel has occurred but the displacements have been controlled.

Excavation of the bench results in failure of the lower sidewalls and collapse of the forepole umbrella.
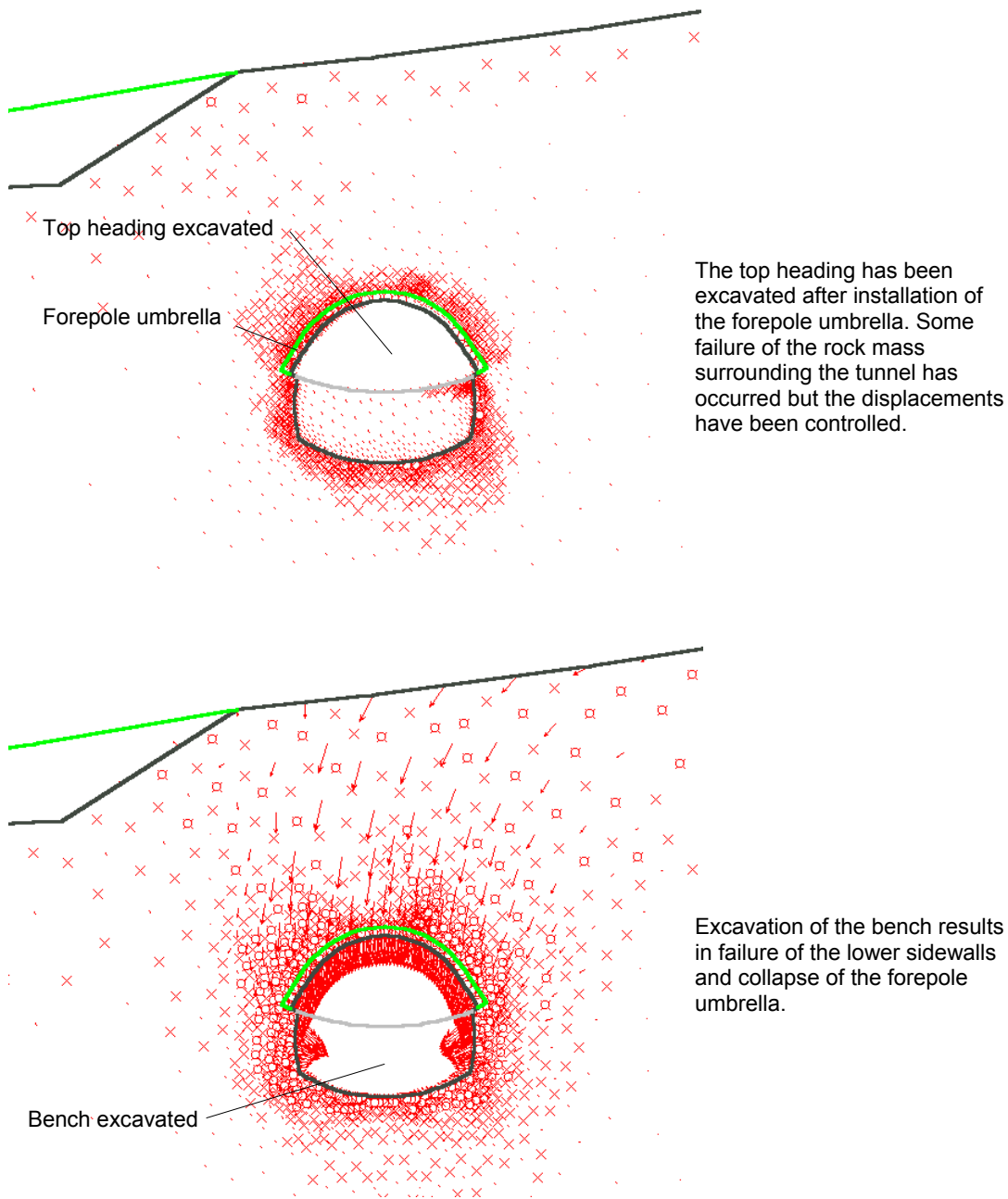
Bench excavated

Figure 7: Support provided by the forepole umbrella is very effective for the top heading excavation but, after excavation of the bench, the lower sidewalls of the tunnel fail and the forepole umbrella collapses with the rock mass caving to surface.
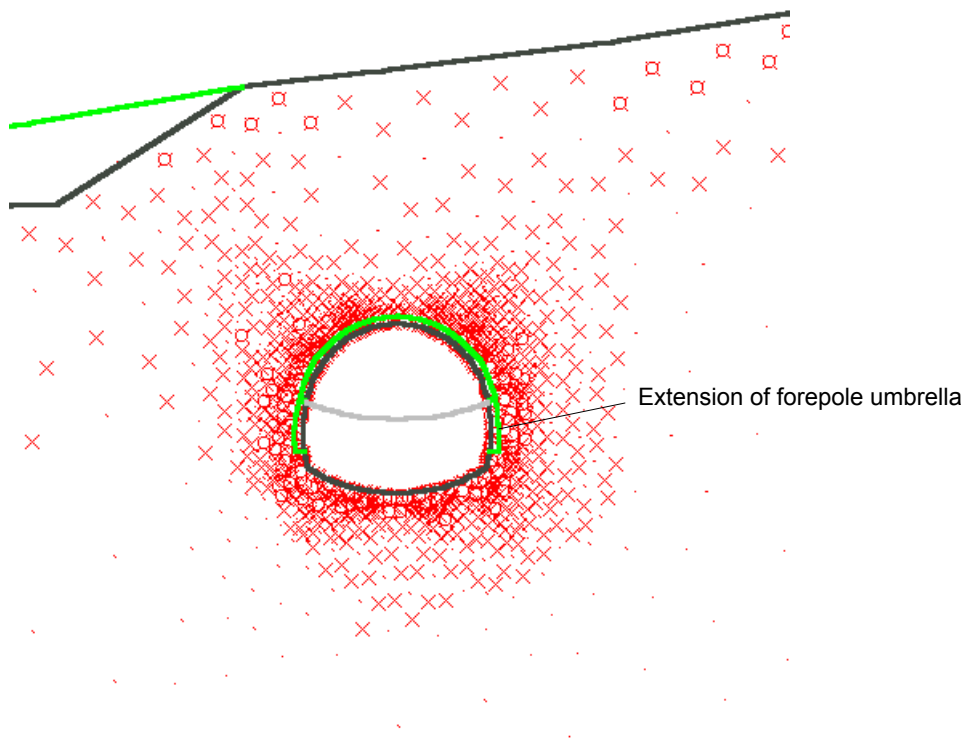
Figure 8: Extending the forepole umbrella down the sidewalls may be a very effective way of strengthening the foundation for the forepole umbrella.

In spite of the limitations of currently available closed form solutions and numerical models, the tunnel designer is faced with the need to produce a design that will stand up to rigorous scrutiny by reviewers and checkers. In some cases these designs have to meet national codes and regulations, many of which were written for steel and concrete structures rather than rock masses. This is a serious problem and there are no simple 'recipe book' answers. The best advice that can be offered is that the designer has to present a design, based upon the best available tools, and that the assumptions upon which the designs are based have to be very clearly stated. The designer must be prepared to defend these assumptions and the final design in both verbal and written discussion and must be prepared to accept that this can be a very frustrating experience.

Since there are no hard and fast rules in tunnel design, the purpose of any discussion or review should be to arrive at a practical solution that is both safe and economical. None of the parties involved in such discussions should be afraid to defend their opinions but all must be prepared to make some compromises in order to arrive at an optimum solution.