Integration of geotechnical and structural design in weak rock tunnels

Discussion by Evert Hoek August 2003

Introduction

Traditionally the design of civil engineering tunnels in weak rock was divided into two separate activities. The site investigation, determination of rock mass characteristics, calculation of tunnel stability and the determination of "rock loads" were carried out by a geotechnical engineer. The design of the final concrete lining was then carried out by a structural engineer on the basis of the loads provided by the geotechnical engineer. In present day weak rock tunnelling the distinction between these two activities has become blurred because of the need for structural design at various stages in the construction process and because this construction process has a direct impact on the final lining design. Hence, there is an increasing tendency to integrate the geotechnical and structural design components into a single package. This process is discussed in the following text.

Weak rock tunnelling

In typical hard rock masses, such as that illustrated in Figure 1, the stability of relatively shallow civil engineering tunnels is controlled by the structural features such as joints, shear zones and faults. In strong rocks the intact rock pieces between these features simply act as rigid blocks and all of the movement and consequent instability problems and controlled by the three-dimensional geometry associated with the intersection of these features and the excavation boundary. Support of the excavations depends upon retaining the interlocking structure of the rock mass and preventing key blocks from being released. Typically, combinations of rockbolts or cables and shotcrete linings are used to ensure that the excavations remain stable. The process for designing excavations in such materials is very well established in geotechnical literature and will not be discussed further in this text.

Weak rock masses are those in which the tectonic history of the rock mass has resulted in the shearing and crushing of the original rock pieces and in a disruption of any interlocking structure which may have existed. A typical example is illustrated in Figure 2. In the extreme case, a weak rock mass which has been further disrupted by transportation, either by natural processes or by man, becomes a soil or a waste rock pile. The strength of these rock masses is generally very low and failures can easily be induced by excavation of either tunnels or slopes. Hoek and Marinos (2000) have discussed the characterization of such rock masses, the estimation of strength and deformation characteristics and the analysis of potential over-stressing (squeezing) problems in weak rock tunnels.

Figure 1: Jointed sandstone rock mass showing well developed structural features.

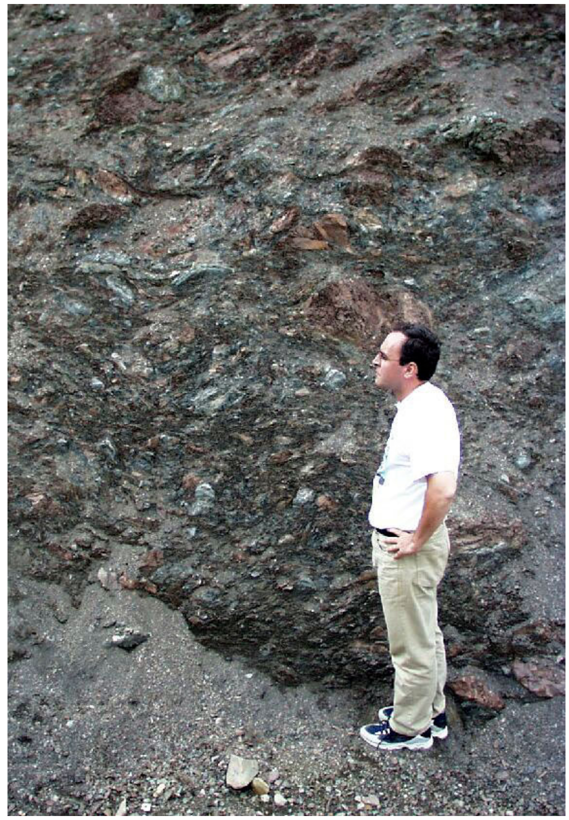


Figure 2: Weak rock mass of tectonically sheared flysch.

Many options exist for tunnelling through weak rock masses and a brief summary is presented in Figure 3. The methods chosen tend to vary with the size of the tunnel and, apart from TBM tunnelling in which a pre-cast concrete final lining can be placed immediately behind the machine; all methods require both temporary lining for support during construction and final lining for the permanent operation of the project. A common feature of all of these methods is the necessity to create a completely closed structural shell, capable of carrying all the imposed loads, at each excavation stage.

A full discussion on the advantages and disadvantages of the various methods exceeds the scope of this paper. The discussion that follows concentrates on the design of the temporary and final linings for a typical highway tunnel excavated and supported three stages. The first stage involves the excavation of a top heading for the complete length of the tunnel, followed by removal of the bench in the second stage and the placement of the final concrete lining in the third stage.

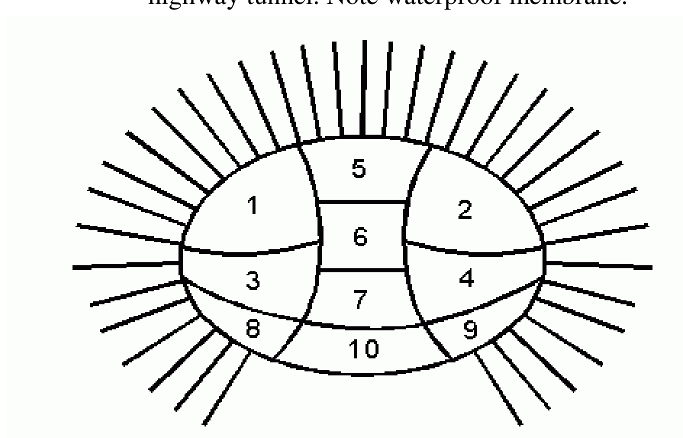
a. Shotcrete lining in a 5.5 m diameter tunnel driven conventionally full face

b. Final pre-cast concrete lining in a 9.5 m diameter tunnel driven by a TBM

c. Top heading and bench excavation in a 12 m span highway tunnel. Note temporary invert in bench.

d. Placement of final concrete lining in a 12 m span highway tunnel. Note waterproof membrane.

e. Temporary reinforced shotcrete lining in a partial face heading



f. Excavation sequence for an 18 m span underground metro station using multiple headings as in e.

Figure 3: Examples of typical temporary and final linings for different size tunnels in weak rock.

Practical example

In order to illustrate the design of temporary and final linings, a typical example of a 12 m span highway tunnel will be considered. This example has been assembled from many such tunnels that have been constructed on the route of the 680 km long Egnatia highway in northern Greece and, while it does not represent any particular tunnel, the details are typical for tunnelling projects in the mountains along this route.

A simplified construction sequence is illustrated in Figure 4 which shows the excavation and temporary support of the top heading, the excavation and temporary support of the bench and the placement of the permanent concrete lining. Note that the concrete lining is placed within a stable tunnel and it is therefore subjected to no loads other than its self weight. Long term loading, for which the final concrete lining must be designed, occurs as a result of the following:

1. Loss of support provided by the rockbolts of other embedded steel elements due to corrosion.
2. Deterioration of the properties of the temporary shotcrete lining. In many cases the applicable design specifications require that any support provided by the rockbolts or temporary shotcrete lining have to be ignored when designing the final lining.
3. Deterioration of the surrounding rock mass properties due to long-term chemical alteration or creep.
4. Build up of water pressure around the tunnel due to long term blockage of the drains. Note that, in cases when the tunnel is located in a particularly sensitive environmental region, long term drainage may be prohibited and the tunnel lining has to be designed for the full external water pressure due to re-establishment of the groundwater table.
5. Live loads due to vehicle impact, fire, explosions in the tunnel or earthquakes. In general earthquake loads are ignored for tunnels with a vertical and lateral cover of more than one diameter but they have to be incorporated into the design of very shallow tunnels or for portals.
6. Shrinkage and thermal stresses induced by the construction process and sequence.

The design process involves calculation of the bending moments and axial thrusts in both the temporary and final linings at each construction stage and checking the maximum values against moment-axial thrust capacity diagrams for appropriate reinforced shotcrete or concrete diagrams. The design is an iterative process in which the thickness of the lining and the amount of reinforcement is adjusted until the maximum moment-axial thrust combinations are acceptable. This process is illustrated in the figures and text that follows.

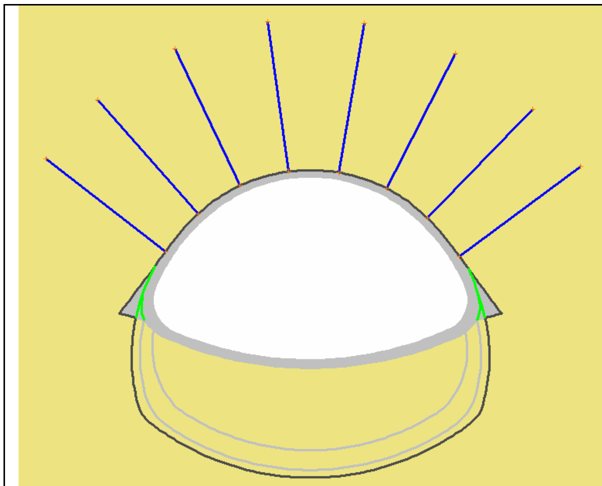
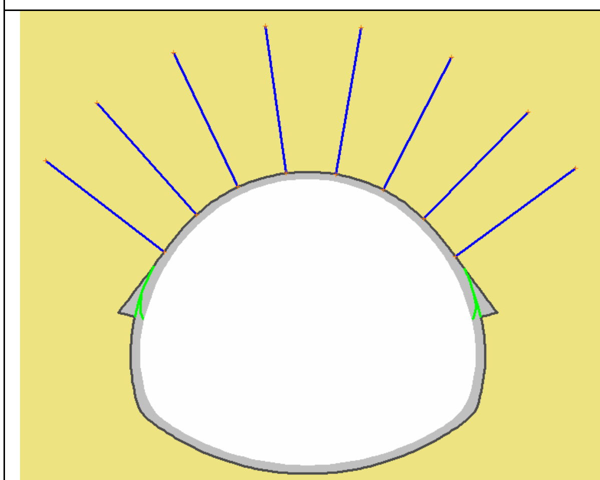
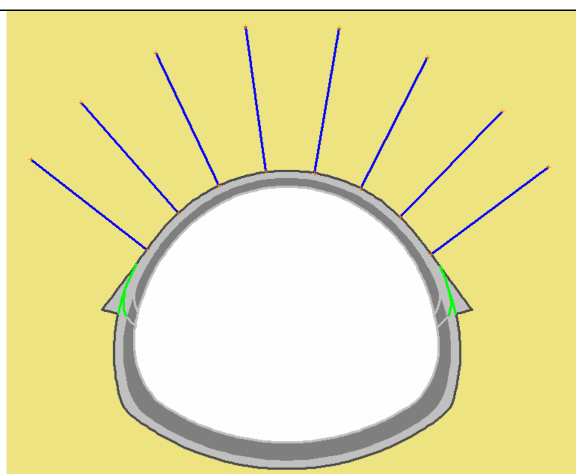
	a. Top heading excavation is advanced with the installation of rockbolts as a close to the face as possible, followed by a temporary shotcrete lining and invert. Note that "elephant feet" are included, if necessary, to provide a foundation for the construction of the top heading support arch.
	b. Bench excavation is advanced with the removal of the temporary invert in the top heading and the placement of a shotcrete lining and invert.
	c. Placement of cast-in-situ concrete final lining within the temporary shotcrete lining.

Figure 4: Sequence of excavation and support installation for a 12 m span highway tunnel

Control of tunnel deformation

In driving a tunnel through a rock mass where the ratio of rock mass strength to in situ stress is low enough that failure develops around the tunnel (Hoek and Marinos, 2000), deformation of the rock mass occurs as shown in Figure 5. The first issue that has to be addressed by the tunnel designer is the stability of the face itself and, for large tunnels, this is usually controlled by the use of grouted fibreglass dowels and/or forepole umbrellas as shown in Figure 6. The design of these systems is beyond the scope of this discussion and details can be found in publications such as that by Hoek (2001) and Lunardi (2000).

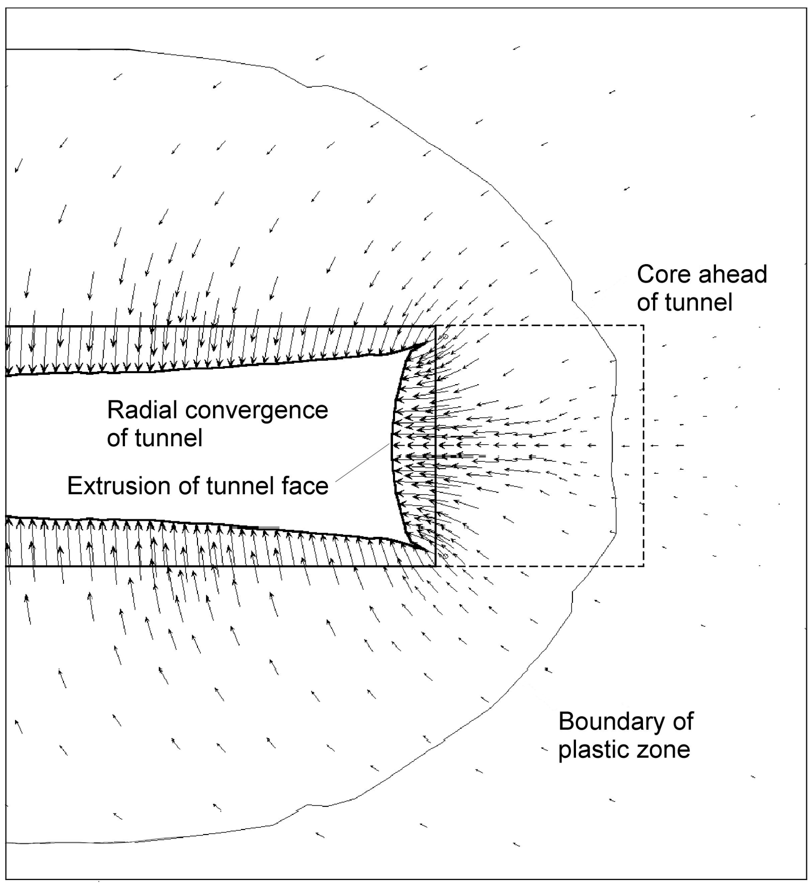
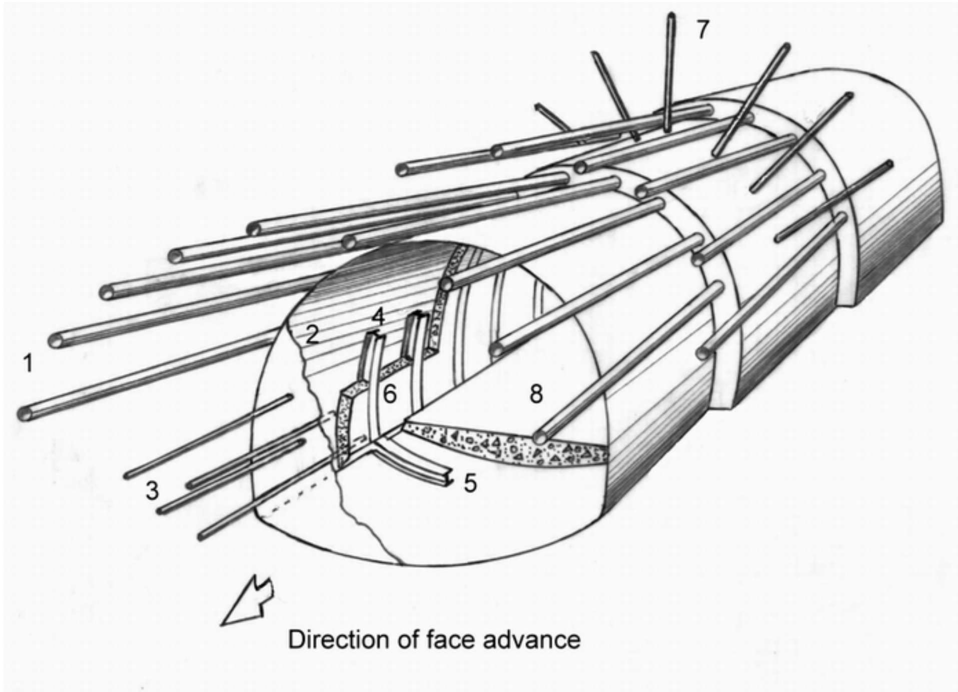


Figure 5: Section through an axi-symmetric finite element model of a tunnel advancing through a highly stressed rock mass. Deformation of the rock mass commences in the core ahead of the tunnel, about one diameter ahead of the face. Convergence of the tunnel is usually complete about 1.5 diameters behind the face.



1 Forepoles – typically 75 or 114 mm diameter pipes, 12 m long installed every 8 m to create a 4 m overlap between successive forepole umbrellas.

2 Shotcrete – applied immediately behind the face and to the face, in cases where face stability is a problem. Typically, this initial coat is 25 to 50 mm thick and it is to prevent deterioration of the exposed rock surfaces.

3 Grouted fibreglass dowels – Installed midway between forepole umbrella installation steps to reinforce the rock immediately ahead of the face. These dowels are usually 6 to 12 m long and are spaced on a 1 m x 1 m grid.

4 Steel sets – installed as close to the face as possible and designed to support the forepole umbrella and the stresses acting on the tunnel.

5 Invert struts – installed to control floor heave and to provide a footing for the steel sets.

6 Shotcrete – typically steel fibre reinforced shotcrete applied as soon as possible to embed the steel sets to improve their lateral stability and also to create a structural lining. This shotcrete may be up to 300 mm thick.

7 Rockbolts as required. In very poor quality ground it may be necessary to use self-drilling rockbolts in which a disposable bit is used and is grouted into place with the bolt.

8 Invert lining – either shotcrete or concrete can be used, depending upon the end use of the tunnel.

Figure 6: Typical support systems used to control the stability of large span tunnels in weak rock masses. Note that the tunnel illustrated is driven full face, ie a single excavation for the complete tunnel profile. After Hoek (2001).

Numerical modelling

The discussion which follows is concerned with the design of the temporary and permanent linings of the tunnel shown in Figure 4, excavated by a top heading and bench process. As can be seen from Figure 5, a critical step in the support process is to install an appropriate temporary lining within the tunnel in order to control the tunnel convergence. If this temporary lining is installed too late or if it has inadequate load carrying capacity then it may not be possible to control the deformations and the tunnel will collapse. On the other hand, if the support is placed too close to the face then it can attract very high loads and heavy support will be required. The choice of the optimum distance behind the face to install the support and the correct load carrying capacity of the temporary support is the most difficult step in tunnelling and is one that demands all of the skill of the tunnel engineer.

Two-dimensional numerical models are frequently used today to assist the tunnel designer in analysing the stresses and deformations in the rock mass surrounding a tunnel and in choosing the correct temporary and permanent support systems. Simulation of the three-dimensional conditions associated with advancing the tunnel, as shown in Figure 5, is usually achieved by progressively "softening" rock mass within the tunnel excavation boundary. The extent of this softening can be judged from a "characteristic curve" such as that shown in Figure 7 in which vertical displacement of a tunnel roof is plotted against the deformation modulus of the inclusion material. This particular curve is for a 12 m span top heading excavation (as shown in Figure 4a) at a depth below surface of 90 m in a poor quality flysch defined by a friction angle $\phi = 17°$, a cohesion $c = 0.11$ MPa and a deformation modulus of 400 MPa. Note that these characteristic curves are unique for every combination of in situ stresses (tunnel depth) and rock mass properties and it is necessary to compute such a curve for each new tunnel design.

For this example it was decided to install the temporary support after a tunnel roof displacement of 0.35m and, as can be seen from Figure 7, this requires that the deformation modulus of the inclusion material be reduced to 40 MPa. This would provide a starting point for an iterative analysis of the interaction between the installed temporary support and the rock mass deformation. Note that, in this case, the influence of the forepoles and grouted fibreglass dowels which would be used to stabilize the face have been ignored in the interests of simplicity.

The next step in the process is the analysis of the behaviour of the tunnel and installed support systems as the tunnel is excavated stage by stage. Only three of the ten stages used will be considered here and these are the steps representing the completion of the top heading, the completion of the temporary support installation in the full tunnel excavation and the final situation in which the concrete lining in the completed tunnel is subjected to long term loading.

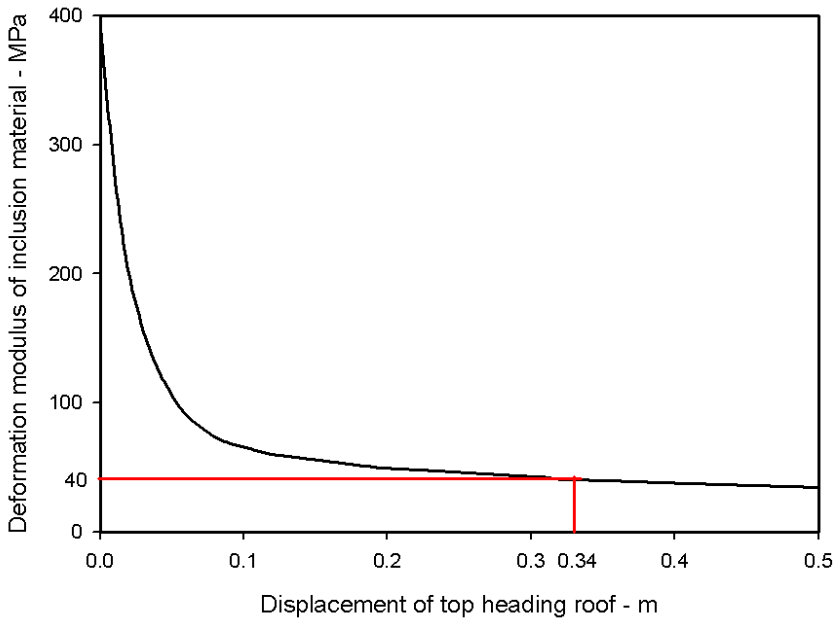


Figure 7: Characteristic curve for a 12 m span tunnel top heading at a depth of 90 m below surface excavated in a rock mass defined by a friction angle $\phi = 17°$, a cohesion c = 0.11 MPa and a deformation modulus of 400 MPa.

Lining capacity calculations

Figure 8 shows the bending moments and axial thrusts induced in the temporary linings for both the top heading and the full excavation profile and also for the long term loading on the final concrete lining. The finite element program Phase2[1] was used and the linings were represented by elastic beam elements with a thickness of 30 cm for the temporary linings and varying from 50 cm to 1 m for the final concrete lining. The long term loading conditions for the final concrete lining involved removal of the rockbolts and a reduction in thickness of the temporary lining and the imposition of an external pressure of 0.5 MPa on the concrete lining. This external pressure simulated long-term build up of water pressure due to clogging of drains and also long term creep of the rock mass.

The method described by Sauer *et al* (1994) has been used to estimate the moment-axial thrust combinations for the temporary lining of both the top heading and the full excavation profile, plotted in Figure 9. The values for the connection between the inverts and the walls have been plotted in different colours so they can be distinguished from the other part of the linings. These connections require special consideration.

[1] Details available from www.rocscience.com.

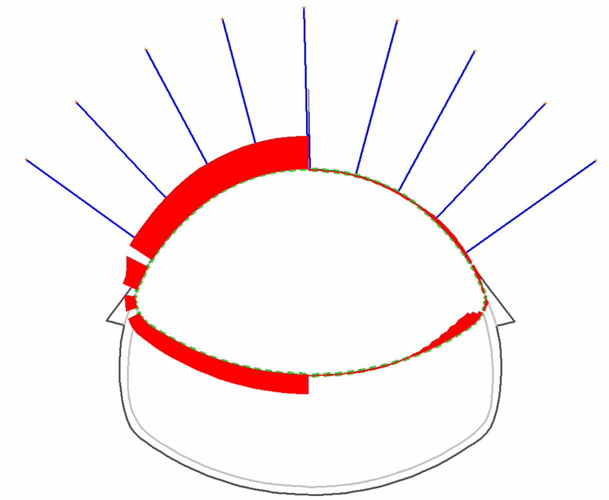
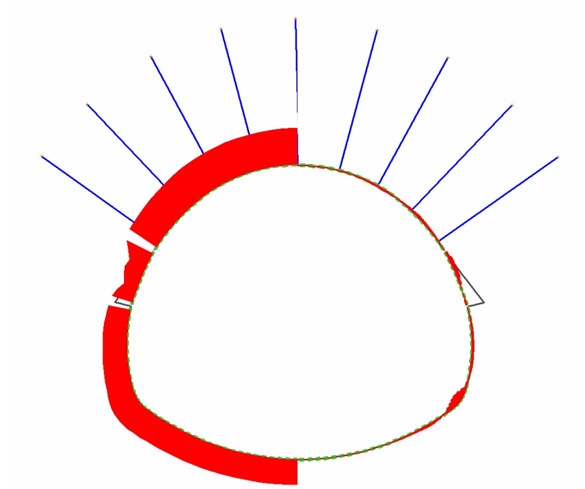
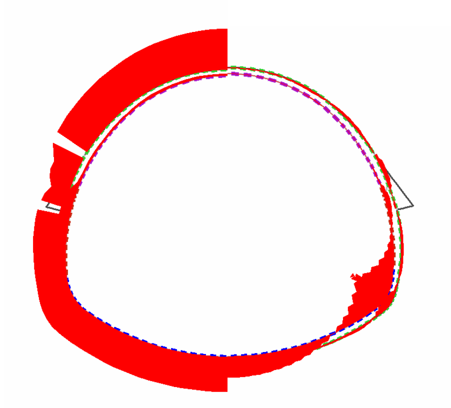
	a. Axial forces (left hand side of figure) and bending moments (right hand side) induced in the lining and temporary invert of the tunnel top heading.
	b. Axial forces and bending moments induced in the temporary lining for the full tunnel excavation profile.
	c. Axial forces and bending moments induced in the final concrete lining after the removal of all temporary support and the application of an external pressure of 0.5 MPa to simulate long term loading.

Figure 8: Axial forces and bending moments induced in linings for different stages.

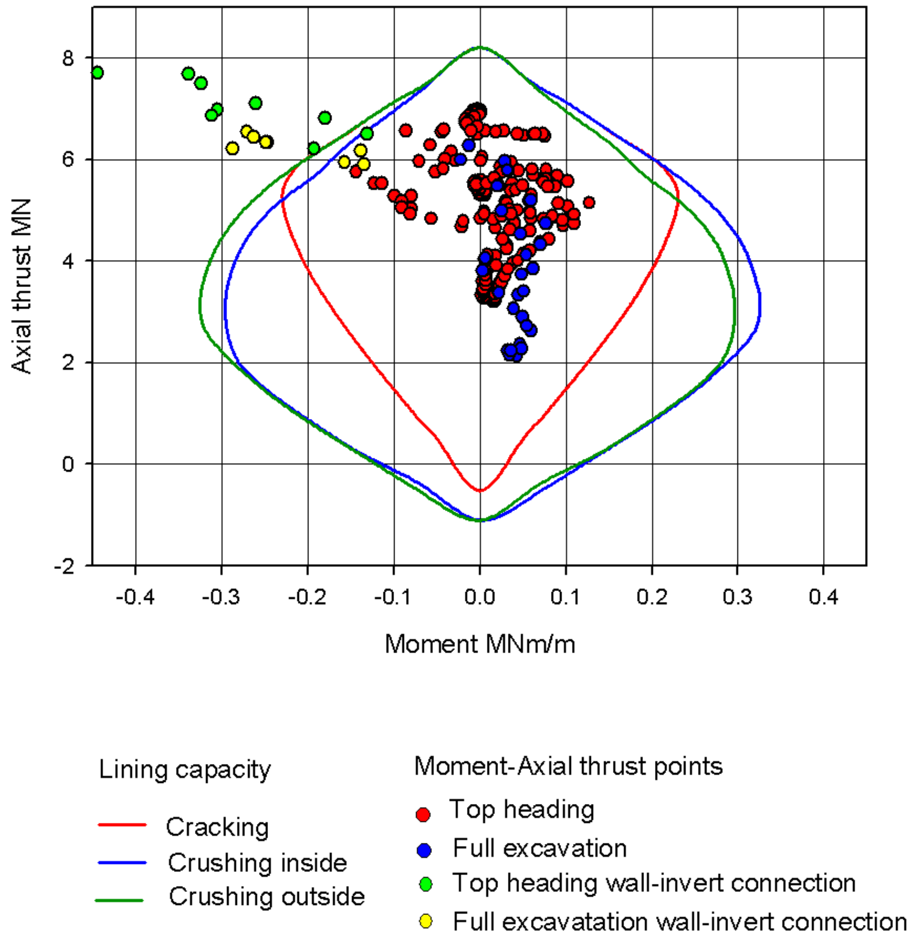


Figure 9: Plot of moment-axial thrust combinations for the temporary linings in both the top heading and the full excavation boundary. The lining capacity curves have been estimated by means of the reinforced concrete design program Response 2000[2].

Temporary lining design

The moment-axial thrust capacity curves plotted in Figure 9 were calculated for a factor of safety of 1.00 on the basis that the linings are temporary and that some cracking and crushing can be tolerated provided that it does not lead to progressive failure. These curves were calculated on the assumption that the temporary lining, including the inverts, consists of 30 cm thick shotcrete (25 MPa uniaxial compressive strength, 5000 MPa deformation modulus) reinforced by 4 bar lattice girders at 1 m spacing as shown in Figure 10.

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[2] This program was developed in the Department of Civil Engineering of the University of Toronto and can be downloaded from www.ecf.utoronto.ca/~bentz/r2k.htm.

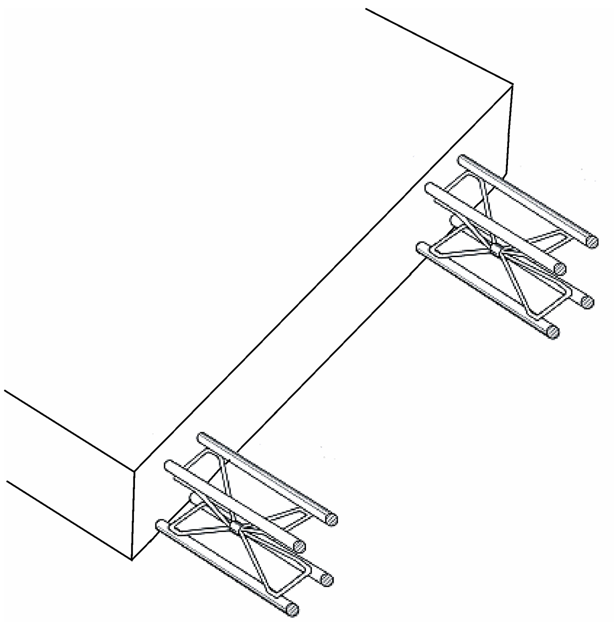

Figure 10: Sketch of a 30 cm thick shotcrete lining reinforced by 4 bar lattice girders at 1 m spacing

Figure 9 shows that the moment-axial thrust combinations for the temporary linings (except for the connections between the walls and the inverts) fall well within the capacity curve defined by cracking of the shotcrete. Consequently, these linings can be considered acceptable. On the other hand, the bending moments in the connections between the inverts and the walls are such that the lattice girder reinforced shotcrete layer does not provide adequate support capacity. This is a well recognized problem in tunnel design and these connections require special designs.

In the example under consideration here the top heading is provided with “elephant feet” in order to ensure that sufficient capacity is available for the footings of the arch (see Figure 4). These elephant feet have to be designed to ensure that there is sufficient reinforcement present to ensure an adequate connection between the walls and the invert. Similarly, additional reinforcement is required in the connections between the walls and the invert of the temporary lining for the full section.

A critical step in the excavation sequence is to ensure that the legs of the lining for the bench excavation are correctly connected to the top heading arch at the time of the removal of the temporary invert of the top heading. The bench excavation is carried out in a step wise manner so that the length of unsupported top heading arch is kept to a minimum. In some cases rockbolts are used to anchor the arch legs while in other cases these arch legs are connected by longitudinal beams so that they remain stable during the bench excavation process.

Final lining design

In contrast to the temporary lining, where some failure of the lining can be tolerated and where a factor of safety of 1.00 can be used in the lining design, a final concrete lining must be designed for a maintenance-free life of 50 or more years. Consequently, all the loads that can occur during the operational life of the tunnel have to be taken into account and an appropriate factor of safety has to be used for the design. According to DIN 1045 the factor of safety that should be applied to unreinforced concrete is 2.1 while more ductile reinforced concrete can be designed with a factor of safety of 1.75. (Sauer *et al*, 1994)

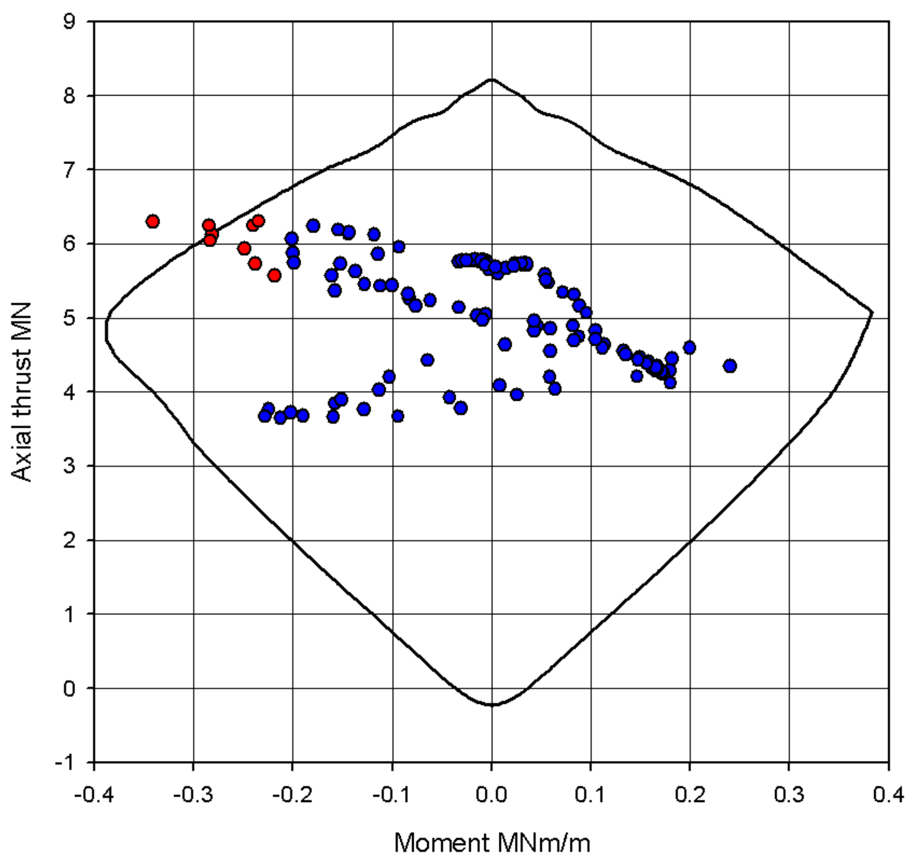

Figure 11: Moment-axial thrust capacity curve (based in the start of cracking) for a 50 cm thick unreinforced concrete lining with a uniaxial compressive strength of 35 MPa. The blue plotted points are for induced moment-axial thrust combinations in the arch, walls and invert of the final concrete lining. The red points are for the connections between the walls and the invert.

The moment-axial thrust capacity diagram for a 50 cm thick lining of 35 MPa unreinforced concrete is plotted in Figure 11. In the same figure, the induced moment-axial thrust combinations shown in Figure 8c are also plotted. These points all fall within the capacity envelope except for the red points which are for the connection between the walls and the invert. However, as shown in Figure 4c, the lining is almost 1 m thick at these connections and hence there us adequate capacity to accommodate the higher moments.

While this analysis indicates that an unreinforced final concrete lining would be adequate for this tunnel, the final choice of the lining would depend upon the owner of the project. In many cases owners are reluctant to use unreinforced concrete linings and insist that nominal reinforcement should be included in the lining design. This is always a matter of intense debate in tunnel design and there is currently no standard for when unreinforced linings can or cannot be used.

Conclusion

This note has been prepared for discussion only and it is not intended for publication until a number of issues have been resolved. These include:

1. The accuracy of the bending moment calculation process. This depends upon the number and type of finite elements used and also on the method and sequence of construction of the boundaries, particularly the connection between the temporary invert and the primary lining in a top-heading and bench operation. Some of the issues of calculation accuracy have been discussed by Wittke (2002) but further work needs to be done on this matter for the programs used in these notes.
2. The Moment-Thrust interaction calculations preformed by *Response 2000*, while ideal for typical reinforced concrete structures, are not yet adequate for tunnel linings in which different reinforcing elements, such as steel sets, are used. This deficiency needs to be rectified and the calculation fully integrated into the finite element programs used for calculating the stresses and deformations in the rock mass surrounding the tunnel.
3. Factors such as thermal and shrinkage stresses, dynamic loading due to vehicle impacts and earthquakes and the effects of tunnel fires need to be incorporated into the final lining calculations.
4. In the case of hydraulic tunnels the internal and external water loads acting on the final lining must be taken into account for the wide range of conditions that can occur along a pressure tunnel.

References

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