Cavern reinforcement and lining design

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Introduction

This note has been prepared in response to a technical support question submitted to RocScience on the issue of reinforcement and shotcrete lining design for an underground cavern in a good quality rock mass at a depth of 880 m below surface. The analysis presented here was carried out using the program Phase2 Version 8.

Rock mass properties

The rock mass is a jointed granodiorite which has been characterised with the following properties:

Uniaxial compressive strength of intact rock	σ_{ci} = 125 MPa
Constant mi for intact rock	m_i = 23
Geological Strength Index of rock mass	GSI = 50
Modulus Reduction factor	MR = 425
Constant m_b for rock mass	m_b = 3.857
Constant s for rock mass	s = 0.0039
Constant a for rock mass	a = 0.506
Deformation modulus E of rock mass	E = 17,000 MPa
Poisson's ratio for rock mass	ν = 0.27
Unit weight of the rock mass	γ = 0.027 MN/m^3
Residual strength of jointed rock mass	
Characterised by a blast damage factor	D = 0.5
Constant m_{br} for residual strength	m_{br} = 2.217
Constant s_r for residual strength	s_r =0.0013
Constant a_r for residual strength	a_r =0.506
Deformation modulus for residual strength	E_r = 7,806 MPa

In situ stresses

Based on in situ stress measurements, the in situ stresses are defined as the gravitational overburden stress = 880 x 0.027 = 23.8 MPa (vertical). The maximum horizontal stress acting normal to the cavern axis is 1.2 times the vertical stress while the intermediate horizontal stress is equal to the vertical stress.

Note that the determination of these rock mass properties and the assessment of the in situ stresses were done by someone else and are beyond the scope of this note.

Setting up the model

The cavern has a span of 16 m and a height to the top of the domed arch of approximately 25 m. The cavern will be mined from an adit in the centre of the arch. This adit will then be slashed out on either side to form the complete arch and this will be followed by benching down in six stages to excavate the body of the cavern.

The first step in setting up the model is to decide on the finite element mesh required to give the most reliable results. Since the primary purpose of this model is to design a thin (typically 0.3 m) shotcrete lining in a relatively large cavern, it is necessary to choose a mesh that will allow vertices to be spaced at approximately half the thickness of the shotcrete lining. Widely spaced vertices will result in a poor distribution of forces in the beam elements which are used to model the shotcrete lining.

Experience has shown that a six-noded triangular element mesh gives good results in these conditions. It has been found that a precise definition of the vertex spacing on the excavation boundary is critical and the steps in setting up this boundary and the finite element mesh are as follows:

1. The arch of the cavern is entered as three arcs as shown in Figure 1. The coordinates defining these three arcs can be determined from a construction using the CAD interface built into the Phase2 modeller. Choose the "Materials" menu for this construction and set the approximate segment length at 0.15 m as shown in the inset option box in Figure 1. Draw two circles of 4 m radius with their centres located at the coordinates - 4, 20.4 and 4, 20.4 as shown. Draw a line through each centre to the centre of a large circle, of chosen radius, defining the roof arc. The intersection of the projections of these lines with the two small haunch circles will define tangent points for the connection of the haunch and roof arcs. Note the coordinates required for the definition of the three arcs from the display when snapping onto a vertex.

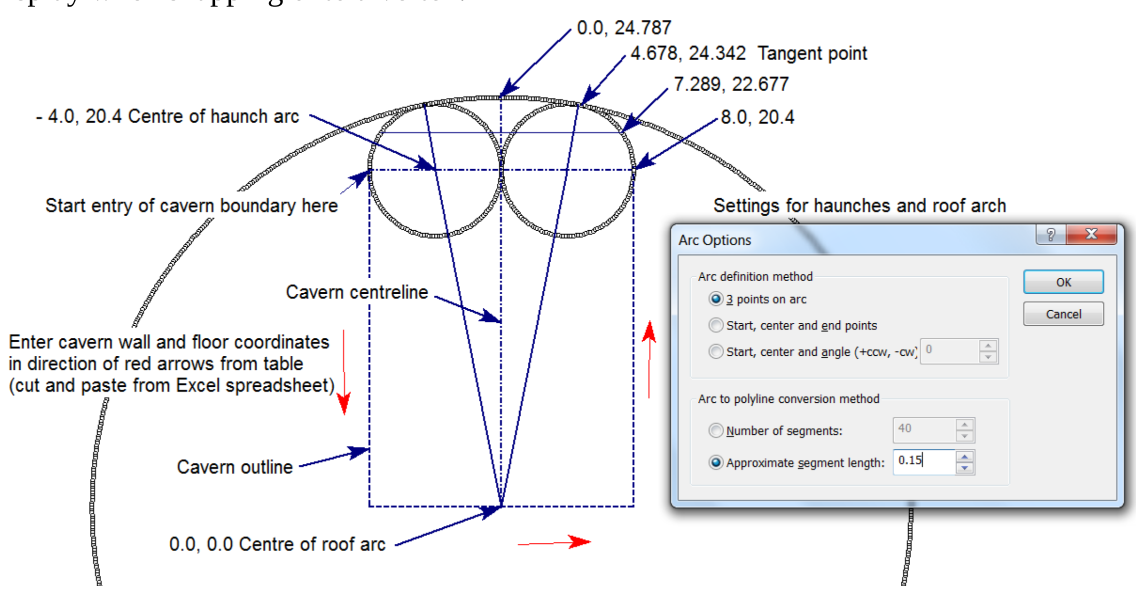


Figure 1: Construction of the cavern profile in order to determine coordinates.

2. Having determined these cavern arch coordinates, entry of the cavern boundary can commence. It has been found that the least frustrating way of doing this is to enter the cavern walls and floor first.

3. A vertex spacing of 0.15 m has been chosen for this analysis. As shown in Figure 1, installing the boundary commences at the connection between the roof arch haunch and the left vertical wall (coordinate -8, 20.4). The complete box forming the walls and floor of the cavern can be entered by importing or cutting and pasting calculated coordinates from an Excel spreadsheet into the table that is displayed when choosing the table option in the "Enter Vertex" menu.

4. Once the walls and floor have been entered and the coordinates 8, 20.4 have been reached, the cavern haunches and roof arch can be installed as a series of three point arcs, using the coordinates defined in step 1 above. Note that the approximate segment length of 0.15 m is used to match the segment length for the walls and floor.

5. Benches and the vertical walls of the centre adit are now entered as stage boundaries in order to allow sequential excavation of the cavern.

6. Before meshing the model, the discretization process should be set using the "Mesh setup" menu shown in Figure 2. Note that the "Improve discretization grading" option has been checked in this menu. This improves the node grading on the stage boundaries as they approach the cavern boundary and it results in an optimum mesh at these connections.

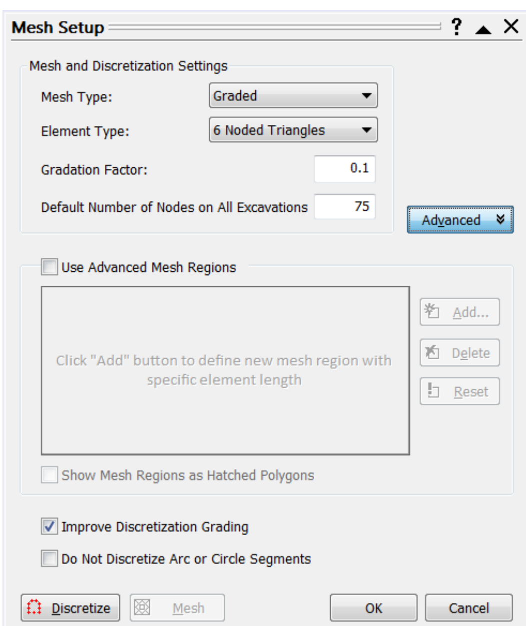


Figure 2: Mesh setup options.

7. The model can now be discretized and meshed and Figure 3 shows the resulting mesh.

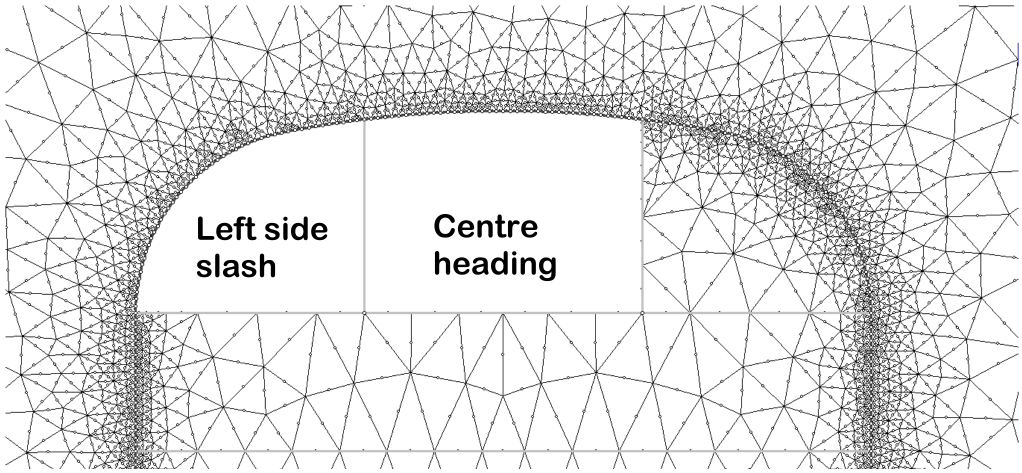


Figure 3: Example of the six noded triangular element mesh created using the steps described above. Note that the centre heading and left side slash have been mined in this example.

Optimization of the roof arch shape

The arch of the cavern is excavated first from a central heading followed by slashing the sides until the full arch has been excavated to the top of the first bench. The performance of the reinforcement and support is very sensitive to the shape of the cavern arch and to the sequence of excavation. Hence, before spending time on the design of reinforcement and support, it is useful to optimize the shape of the arch.

It has been found that the best way to do this is to construct a simple model of the cavern, as illustrated in Figure 4, with three stages. The first stage is the excavation of the full cavern arch to the level of the first bench. This is followed by the installation of a 30 cm thick lining of 40 MPa uniaxial compressive strength shotcrete in the arch. Finally, the remainder of the cavern is excavated in a single stage. This model simulates a rather severe but realistic loading of the shotcrete lining, the performance of which gives a good indication of how close the cavern arch is to an optimum shape. The results of these analyses are plotted in Figure 5.

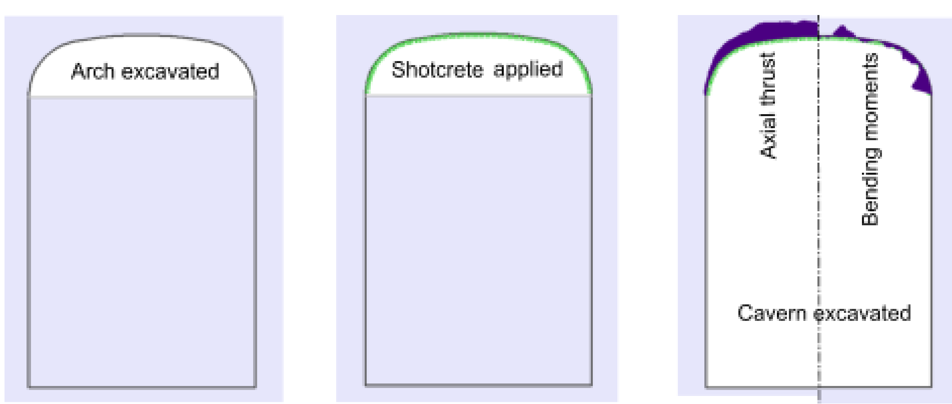
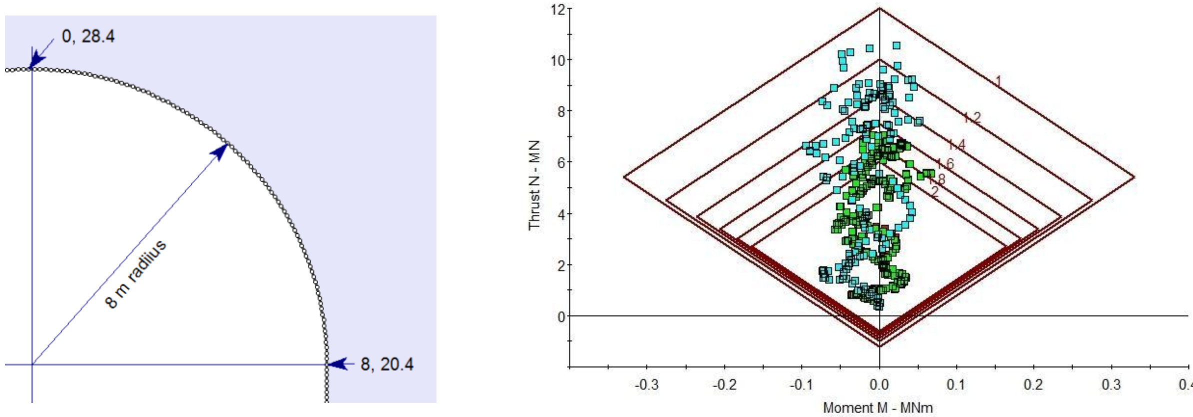
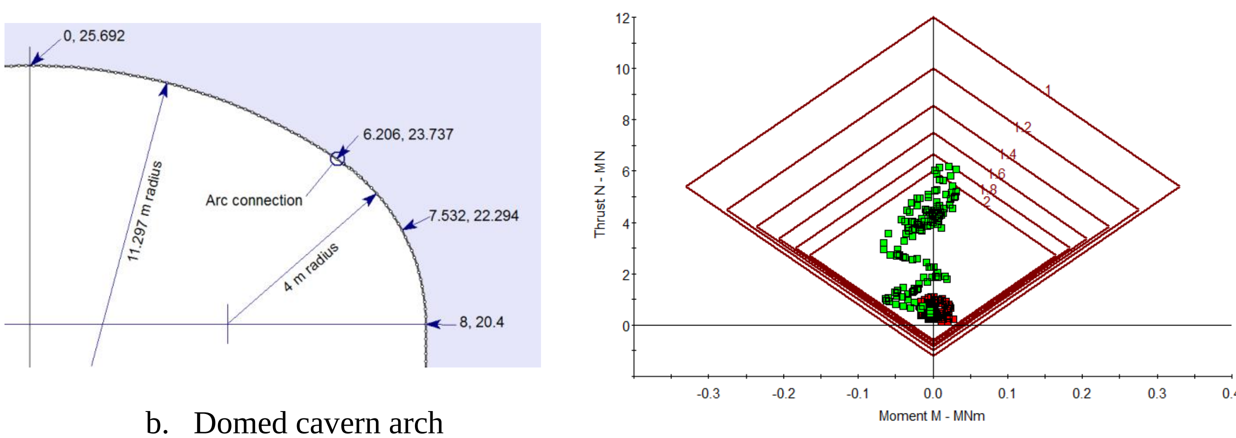


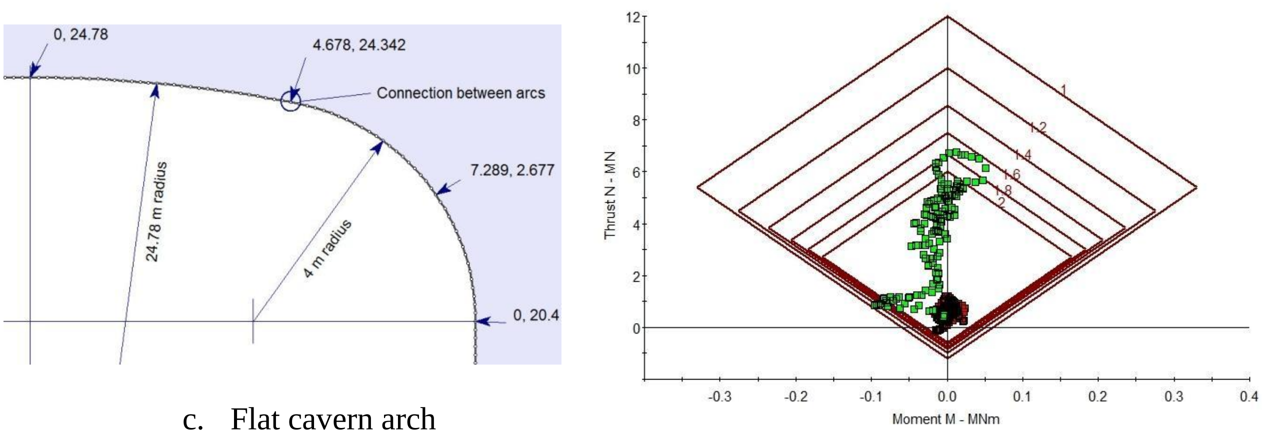
Figure 4: Analysis of optimum cavern arch shape



a. Circular cavern arch



b. Domed cavern arch



c. Flat cavern arch

Figure 5: Comparison between support capacity plots in which calculated axial thrusts and bending moments are compared to allowable values (plotted as factors of safety) for different cavern arch profiles.

Figure 5 shows, for the given in situ stresses, the circular cavern arch results in the generation of high axial thrusts which give a factor of safety of approximately 1.1 on the support capacity plot. These high axial thrusts are generated in the centre of the arch where the horizontal in situ stresses are concentrated.

On the other hand, the flat arch in Figure5c has unfavourable combinations of bending moments and axial thrusts in the haunch of the arch due to downward flexure of the centre of the arch.

Figure 5b shows a domed arch profile which is close to optimum in that favourable combinations of axial thrusts and bending moments exists for the entire arch profile. This profile has been adopted for all subsequent analyses in this note.

Sequential excavation of cavern

In designing the reinforcement and support for the rock mass surrounding a large cavern it is important to recognize and to simulate the sequence of excavation and support installation as realistically as possible. A simplified isometric view of an excavation sequence is given in Figure 6. Ideally, the simulation should be done in a three-dimensional model such as FLAC3D but, because of the cost and complexity of such models, they are not yet in common use by underground excavation designers. Today, most cavern designs, at least in the preliminary stages, are carried out using two-dimensional software such as Phase2, FLAC or UDEC.

In modelling a single tunnel or heading, the decreasing support provided by the tunnel face as it advances away from the section under consideration can be simulated by one of two methods. The first involves the use of staged distributed loads which are defined by the "Field stress vector". This method works well for single tunnels but, for multiple adjacent tunnels, the method is not practical since only a single set of field stress vectors can be specified. The second method uses soft inclusions to limit the deformation of the tunnel and, by successively reducing the modulus of the inclusion, the characteristic curve for the tunnel can be calculated. This method is much more versatile than the field stress vector method and is illustrated in the following figures.

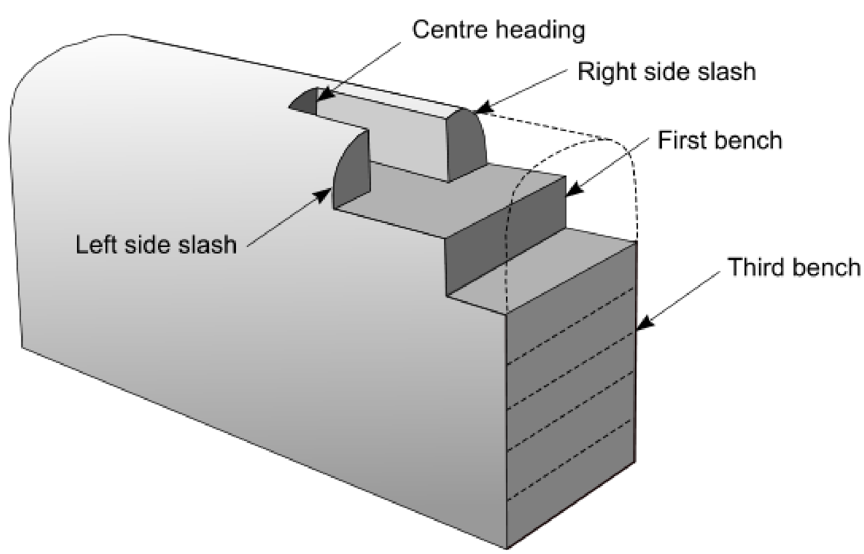


Figure 6: Simplified isometric view of the excavation sequence for a cavern.

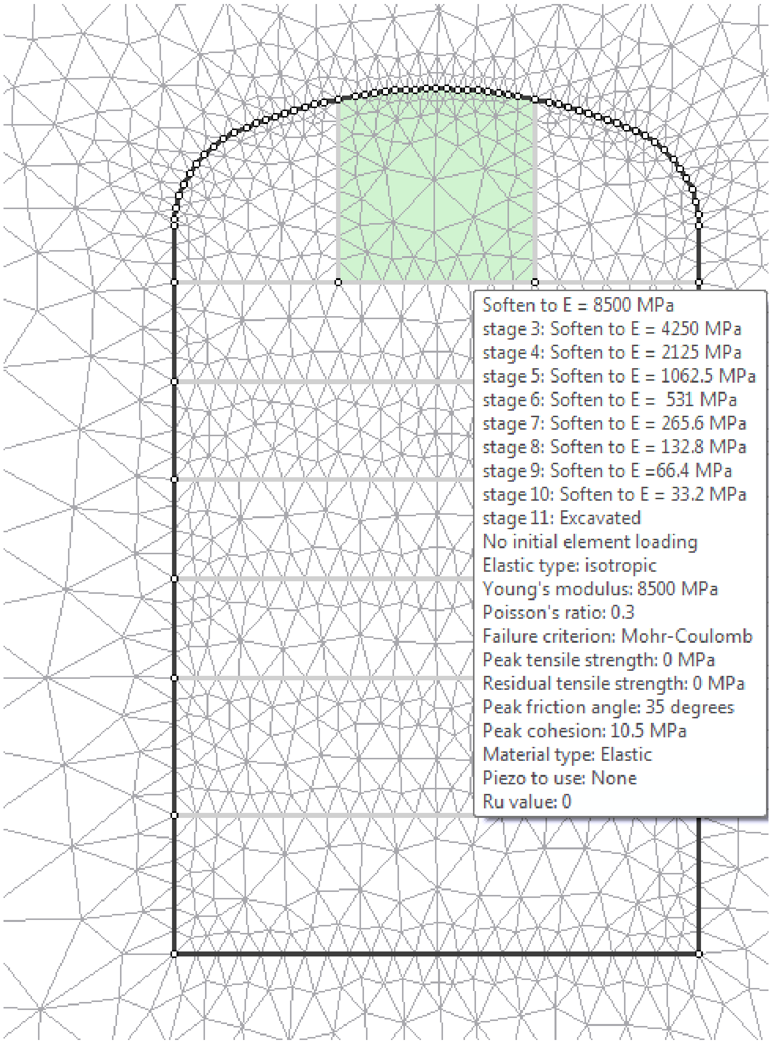


Figure 7: Successive reduction in the deformation modulus of a soft inclusion in the top central heading of a cavern excavation. The first initialization stage to consolidate the model used the modulus of the surrounding rock mass (E = 17,000 MPa) for the inclusion. The figure illustrates the second stage where the modulus has been halved and the list shows that the modulus in each successive stage of one half that of the previous stage. This process gives the characteristic curve for the cavern top heading plotted in Figure 9.

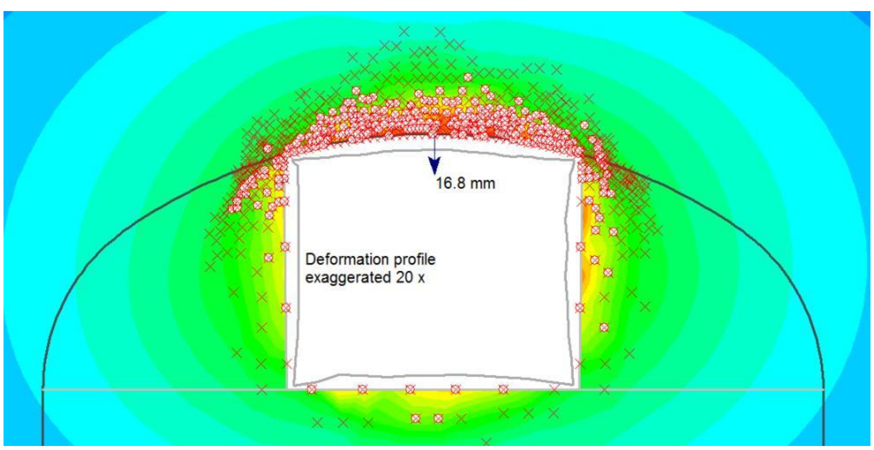


Figure 8: Zone of failure surrounding the top heading and maximum displacement in the roof (16.8 mm) at stage 11 of the process illustrated in Figure 7.

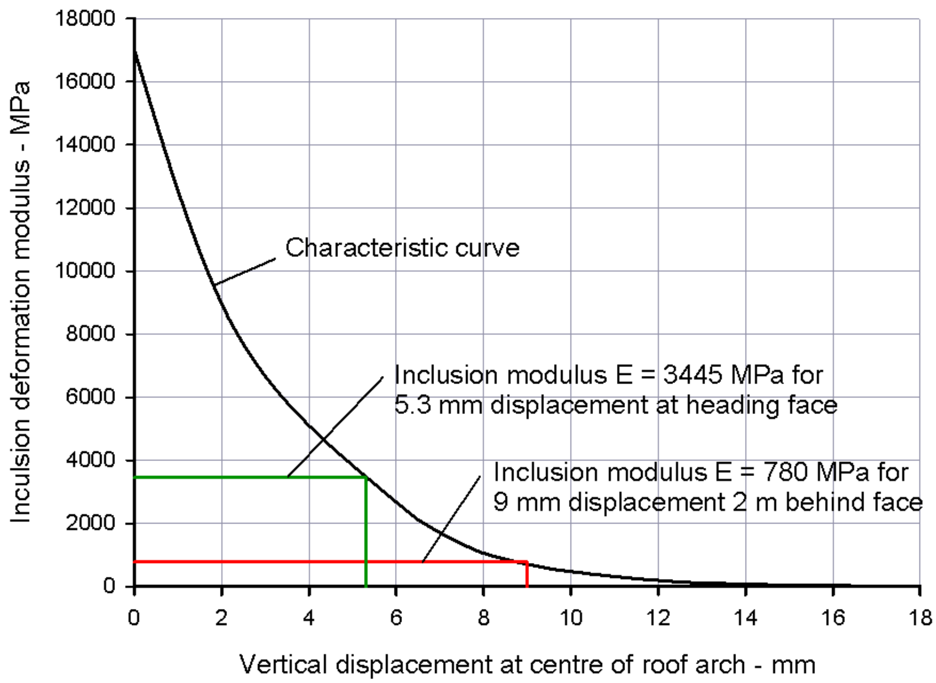


Figure 9: Characteristic curve for the top heading plotting vertical deformation at the centre of the top heading roof versus the deformation modulus of the soft inclusion. The coloured lines are obtained from Figure 10.

In order to make use of the information contained in Figure 9 it is necessary to establish some type of relationship between the deformation modulus of the soft inclusions and the distance from the advancing heading face. This problem has been studied in detail by Vlachopoulos and Diederichs[1] who produced a family of Longitudinal Displacement Profiles (LDP) for different ratios of failure zone to tunnel diameters for circular tunnels. These curves provide a reasonable estimate of the relationship between displacement and distance from the tunnel face for an almost square tunnel and they have been adopted for use in this analysis.

Figure 10 shows the LDP curve for a tunnel where the diameter of the failure zone is approximately 1.5 times the diameter of the tunnel (as shown in Figure 8).

Figure 10 shows that about 30% of the maximum displacement has occurred at the tunnel face and that, in the case considered, 9 mm of displacement occurs at 2 m behind the tunnel face, a typical minimum distance at which rockbolts or cables and the first shotcrete layer are applied. At 10 m behind the face the roof displacement is 16.3 mm and this is typical of the position at which the final shotcrete layer may be applied. These deformations are applied in Figure 9 (coloured lines) to determine that an inclusion deformation modulus of E = 780 MPa gives the 9 mm of deformation at 2 m behind the face while the displacement of 16.3 mm at 10 m behind the face is close enough to the final displacement of 16.8 mm that it can be assumed that the excavation has been fully excavated.

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[1] Vlachopoulos, N., and M. S. Diederichs (2009) Improved Longitudinal Displacement Profiles for Convergence Confinement Analysis of Deep Tunnels, *Rock Mech. Rock Eng*. 42(2), 131-146.

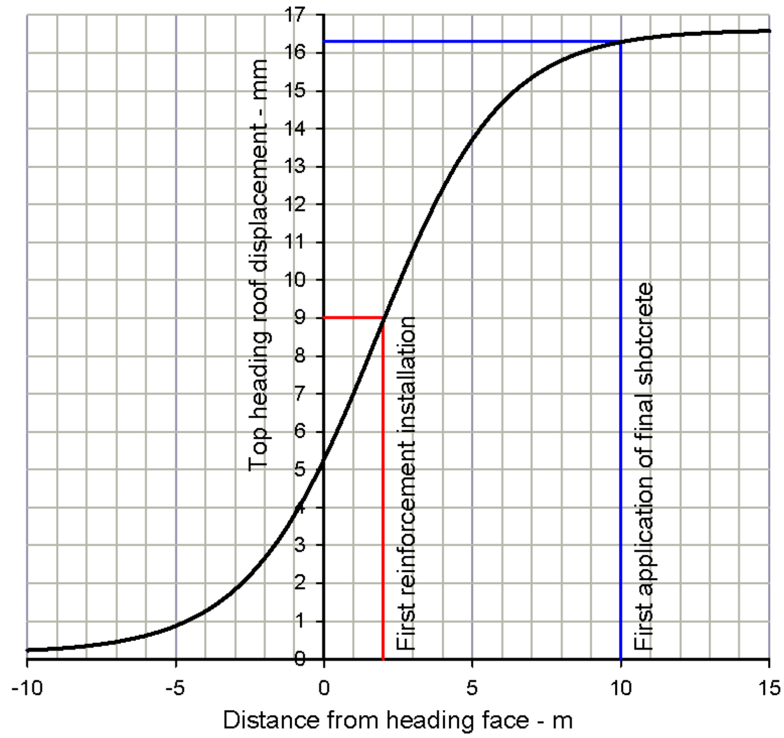

Figure 10: Longitudinal displacement profile for the cavern top heading tunnel

In applying this method to a multi-stage cavern excavation such as that under consideration here, the displacements for the installation of rockbolts or cables and a final shotcrete layer should be determined for each excavation stage by the full modulus reduction process. Simply reducing the deformation modulus of the inclusion from E = 17,000 MPa to E = 780 MPa in one step will not give exactly the same deformation as reducing the inclusion modulus in four or five steps as was done in determining the characteristic curve. The reason for this is quite simple. At each stage, when the material and the modulus are changed, the program resets the stresses inside the zone to zero. Taking the stress out induces deformation which is not recovered when the next inclusion is emplaced. This extra deformation is not accounted for if the modulus is reduced to 780 MPa without the intermediate steps.

Applying this process to each of the 9 stages involved in excavating this cavern results in a very large number of steps and in time consuming model set-up and computation. From a practical point of view, the results are not significantly different from those obtained by eliminating the intermediate steps. The induced deformations will be slightly smaller but the forces in the reinforcement and the shotcrete lining will be slightly larger and thus are conservative. In the analysis presented here the intermediate steps in the inclusion softening process have been left out.

Choice of reinforcement and support systems

For a large cavern in a jointed rock mass in which the intact rock is hard and strong, as is the case in this example, the primary tool for stabilizing the cavern is reinforcement. The use of rockbolts, cables or tiebacks serve the same purpose as reinforcing steel in concrete in that the strength of the rock mass is improved and it is better able to support itself. Hence, a good starting point in designing a large cavern is to find the reinforcing system that will stabilize the surrounding rock mass.

Tensioned and grouted rockbolts or cables are the typical reinforcing elements used in the construction of large caverns. In the case of large caverns at significant depth below surface, the stresses in the rock mass may be such that the strain in the rock and that in the steel reinforcement are incompatible and failure is induced in the grout bond or even in the reinforcement itself. In this case it is necessary to "de-bond" the reinforcement to allow a "tieback" to be created. This is a rockbolt or cable grouted into the rock in such a way that the end of the reinforcement is anchored and the length between the anchor and the faceplate is decoupled from the grout by enclosing it is a plastic sleeve. This decoupled length acts as a spring which transmits a uniform restraining force between the anchor and the faceplate while leaving the rock mass free to deform independently. A simplified illustration of a two strand cable tieback anchor is reproduced in Figure 11.

In the example under discussion in these notes it has been found that the use of tiebacks is required to stabilize the highly stressed rock mass surrounding the cavern.

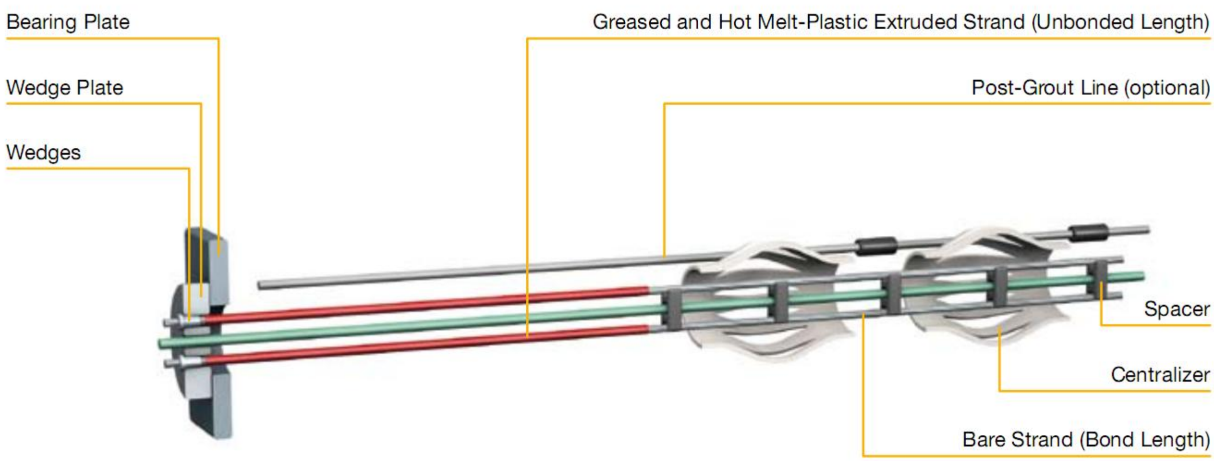

Figure 11: Simplified illustration of a two strand cable tieback. Reproduced from Dywidag-Systems International Strand Anchor Systems Brochure (http://www.dsiamerica.com).

Design of rock mass reinforcement and support

In assessing and optimizing the type of reinforcement and support that can be applied to a large cavern, a model such as that illustrated in Figure 12 is used.

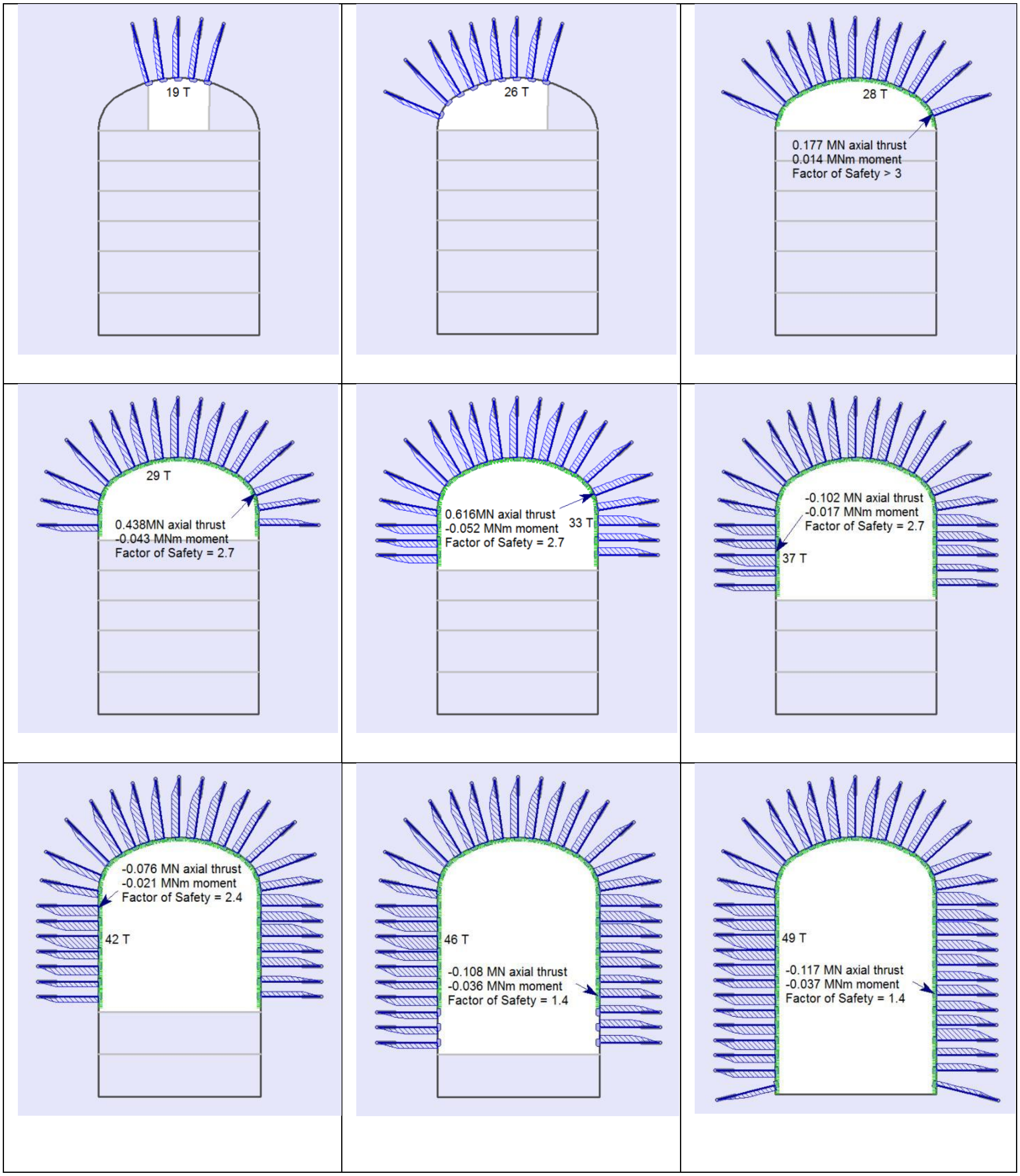

Figure 12: Sequential installation of reinforcement. In this example, 6 m long 60 ton capacity cable tiebacks have been installed on a grid spacing of 1.5 x 1.5 m with the final 30% of the cable grouted to form an anchor. The location and magnitude (in tons) of maximum load induced in the cables and, in addition, the most critical axial thrust, bending moment and factor of safety for the 30 cm thick shotcrete lining is shown at each installation stage.

The aim of the study illustrated in Figure 12 is to determine the capacity, length and spacing of commercially available rockbolts, cables or tiebacks and the thickness and strength of the shotcrete lining that will control the extent of rock mass fracturing and give a deformation pattern that is as smooth and uniform as possible. The actors of safety of the reinforcement and support have to be within acceptable limits.

The results of this study suggest that 6 m long 60 ton cables with the end 30% grouted to form an anchor and the remaining length de-bonded work well as tiebacks for this cavern. These tiebacks are spaced at 1.5 m around the cavern. A 30 cm thick of shotcrete with a uniaxial compressive strength of 40 MPa at 28 days has been assumed for the lining, as discussed below.

It can be seen that the maximum load generated in the cables installed in the cavern sidewalls is 49 tons which, at 80% of the working load of the cables. This occurs in the tall flat sidewalls of the cavern and it is probably too close to the working load to be considered acceptable for long term life of the cavern. Consequently, slightly longer and stronger cables may be considered at this location. On the other hand, the maximum load in the cables in the cavern arch is about 30 tons and it is probable that the capacity of these tiebacks can be decreased to about 45 tons (working load). Further optimization of these reinforcing systems can be carried out by the reader to suit local requirements at the cavern site.

The design of the shotcrete lining is more complicated in that a number of practical considerations have to be taken into account. Typically, a thin (approximately 5 to 10 mm) layer of shotcrete is applied at the same time as the reinforcement. This layer may be applied over firmly attached wire mesh and its purpose is to prevent the fallout of small blocks of rock between the reinforcing elements. This layer is ignored in the overall design process since it is generally damaged by nearby blasting, the installation of the reinforcement and by displacement of the underlying rock mass. This damage is not important since it generally falls off or it is cleaned off before the installation of the final shotcrete layer.

Ideally, the final shotcrete layer should be installed as late as possible so that it is not damaged by ongoing deformations as the cavern is excavated downwards. This depends upon access to the surfaces to be shotcreted and the equipment available for the application of the shotcrete. Today, most shotcrete for caverns is wet mix and it is applied by means of a shotcrete robot.

The properties of the final shotcrete layer can be engineered to meet a wide range of requirements. Wet mix shotcrete is the preferred choice because of its superior properties and also because it generates less dust than dry mix shotcrete. The incorporation of steel or polypropylene fibres, silica fume and various chemical plasticizers or retardants can be used to control the properties very well as discussed in the paper by Franzén (1992)[2]. This paper shows that shotcrete strengths of up to 60 MPa can be attained by appropriate mix designs.

[2] [2] Franzén, T. 1992. Shotcrete for underground support: a state-of-art report with focus on steel-fibre reinforcement. *Tunnelling and Underground Space Technology.* Vol. 7, No 4, 383-391.
http://www.ita-aites.org/fileadmin/filemounts/general/pdf/ItaAssociation/ProductAndPublication/WorkingGroupsPublication/WG12/Tust_Vol_7_4_383-391.pdf

In the cavern under discussion it has been decided to use an unreinforced silica-fume wet mix shotcrete with a 28 day uniaxial compressive strength of 40 MPa and a modulus of deformation of 7000 MPa. A 30 cm layer of this material is applied as a final lining. The modelling process is illustrated in Figure 12 which shows the critical combinations of axial thrust and bending moments and the resulting factor of safety generated in the lining at different stages of construction.

Note that the final shotcrete layer has been applied to the cavern arch after this has been fully excavated down to the level of the first bench. In modelling the shotcrete a 30 cm gap is left at the junction of the arch excavation and the first bench and also between subsequent benches. Shotcrete is seldom applied in this area since it is difficult to clean properly and also because the shotcrete will be damaged during the excavation of the next bench, this gap is shotcreted during the application of the final shotcrete layer to the excavated wall of the next bench.

The final shotcrete on the cavern walls is applied when each bench has been excavated and when the step between successive bench excavations (in plan) is sufficiently far away that it will not provide support for the rock mass onto which the shotcrete is applied.

Careful examination of the magnitudes of axial thrust and bending moment in each stage reveals a very complex pattern which can only be understood if it is recognized that those combinations that place the highest demand on the low tensile strength of the shotcrete will give the most critical conditions in the lining. For the assumed conditions, the lowest factor of safety in the lining is induced in the lower cavern sidewall due to inward deformation of this wall as the cavern is excavated downwards. This factor of safety can be increased by using a thicker or a more heavily reinforced lining in near the base of the cavern.

Conclusions

The use of the two-dimensional finite element program Phase2 for the design of a relatively large underground cavern has been demonstrated. With careful preparation of the model this program can be very effective in carrying out a rational design that takes into account most of the practical issues that are encountered in such a design. The most severe limitations are related to the two-dimensional representation of a three-dimensional problem and this makes it difficult to take into account the actual sequencing of the excavation and support installation that occurs during the excavation of such a cavern. In critical cavern designs it is appropriate to use full three-dimensional analyses for the final support design.

Figure 12 shows the axial forces in the tiebacks and the factors of safety for the 30 cm thick shotcrete lining, determined from the support capacity plot in Figure 1. In both cases that “factors of safety” achieved are only marginally acceptable and the reader will recognize that further optimization is still possible. The extent to which this optimization is justified depends upon the level of confidence that the designer has in the input information on rock mass properties, in situ stresses and the extent to the design can be implemented to reproduce the design results.

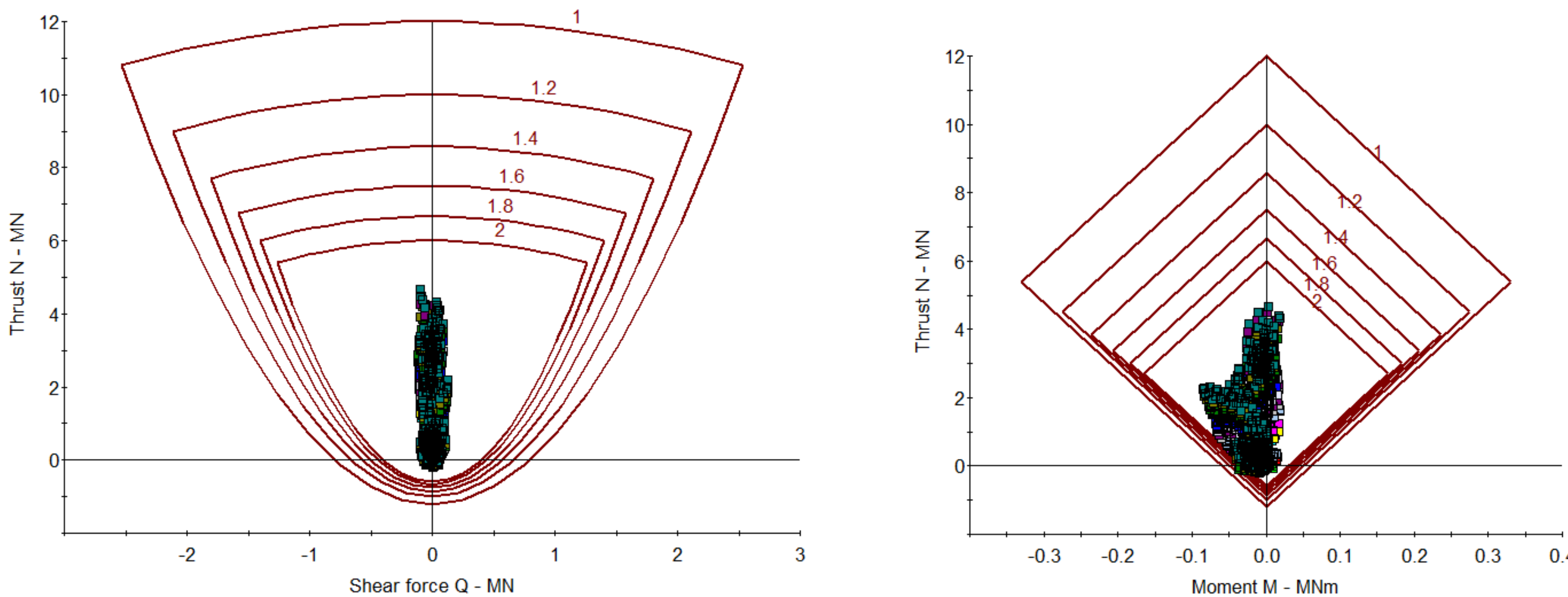

Figure 13: Support capacity plot for the 30 cm thick 40 MPa shotcrete lining with factor of safety contours plotted. The induced axial thrusts and bending moments in each segment of the lining for every stage of the excavation and lining installation process are shown as symbols in this plot. The lowest actor of safety is found to be 1.4 as shown in the last two stages of excavation illustrated in Figure 12.