13

Large Powerhouse caverns in weak rock

13.1 Introduction

In the context of this discussion, weak rock is rock that will fail when subjected to the stress levels induced by the excavation of large underground caverns at depths of 100 to 300 m below surface. Sedimentary rocks such as bedded sandstones, shales, siltstones and mudstones are amongst the rocks which fall into this category. Tunnels and caverns associated with underground hydroelectric projects are sometimes excavated in rock masses of this type.

The design concepts discussed are based upon experience drawn from a number of hydroelectric projects. The choice of the size and shape of power and transformer caverns, the location of these caverns relative to each other and to the ground surface, the influence of joints and bedding planes on the stability of the excavations and the choice of the most appropriate support systems are issues which are common to all of these schemes. In applying these principles to a new scheme, the reader should be aware that each scheme will have its own set of rock mass properties, in situ stress conditions and design constraints imposed by mechanical, electrical and hydraulic considerations. Consequently, the general design concepts outlined here have to be modified to suit each scheme.

13.2 Rock mass strength

In most strong igneous and metamorphic rocks such as dolerites, basalts, granites, gneisses and quartzites, the stability of large caverns at depths of less than 500 m below surface depends almost entirely upon structurally controlled wedges and blocks that are released by the creation of the excavations. In these conditions, the excavation profile can be controlled with good blasting procedures, and the rock mass and any support placed in it are subjected to relatively small displacements. The strength of the rock itself plays a minor role in the behaviour of the rock mass which is controlled by intersecting joints, schistosity, bedding planes, shear zones and faults. Away from the major structures, the rock may be capable of standing unsupported during excavation for considerable periods of time, and the excavation and support of tunnel intersections with the cavern poses no particular problem. The evaluation of the stability of these excavations is carried out by means of limit equilibrium analyses. The factor of safety is calculated by comparing the shear strength of the discontinuities, which bound the potentially unstable blocks and wedges, with the driving forces due to the gravitational weight of these blocks and wedges as discussed in Chapter 5.

In the case of weaker sedimentary rocks, the strength of the rock material is generally lower and the rock mass is frequently more heavily jointed and more deeply weathered. The control of overbreak during excavation by drilling and blasting will be more difficult, and the rock mass and the supports placed in it may be subjected to deformations of up to 50 or 100 mm at the surface of the excavation. The sequence of excavation and support of tunnel intersections with the cavern will have to be controlled and will require careful engineering design. For these caverns, support will usually be required immediately after excavation. In designing these supports, failure of the rock material and sliding and rotation of individual blocks of rock within the rock mass have to be considered in addition to failure along structural features such as bedding planes, shear zones and faults.

The Hoek and Brown failure criterion, described in Chapter 11, provides a basis for estimating the strength of rock masses of the type under consideration here. For all the examples discussed, it has been assumed that the rock mass is a fair to poor quality siltstones and that its properties are defined by:

Uniaxial compressive strength of intact rock	σ_c = 100 MPa
Constant m_i for intact siltstone	m_i = 10
Geological Strength Index	GSI = 48
Rock mass constant	m_b = 1.56
Rock mass constant	s = 0.003
Deformation modulus	E = 8900 MPa
Poisson's ratio	ν = 0.3
Friction angle	ϕ = 31°
Cohesive strength	c' = 4 MPa

Figure 13.1 gives plots of the relationship between the maximum and minimum principal stresses and the shear and normal stresses at failure defined by the Hoek-Brown failure criterion.

13.3 In situ stress conditions

A study of the results of in situ stress measurements from around the world suggests that the horizontal stress is generally significantly greater than the vertical stress at depths below surface of less than 1000 m (Brown and Hoek (1978), Sheory (1994)). The vertical stress is normally assumed to be equal to the product of the unit weight of the rock mass and the depth below surface and measured in situ stresses are usually in reasonable agreement with this assumption. The ratio of average horizontal stress to vertical stress can be as high as 3 and values of 1.5 or 2 are frequently assumed for preliminary analyses.

It is always advisable to measure the in situ stresses in the vicinity of major underground caverns as early in the project feasibility study as possible. During early site investigations, when no underground access is available, the most commonly used method for measuring in situ stresses is hydrofracturing (Haimson (1978)). The hydraulic pressure required to produce fresh cracks and subsequently close and re-open them is used to estimate in situ stress levels. Once underground access is available, overcoring techniques can be used, as discussed in Chapter 10.

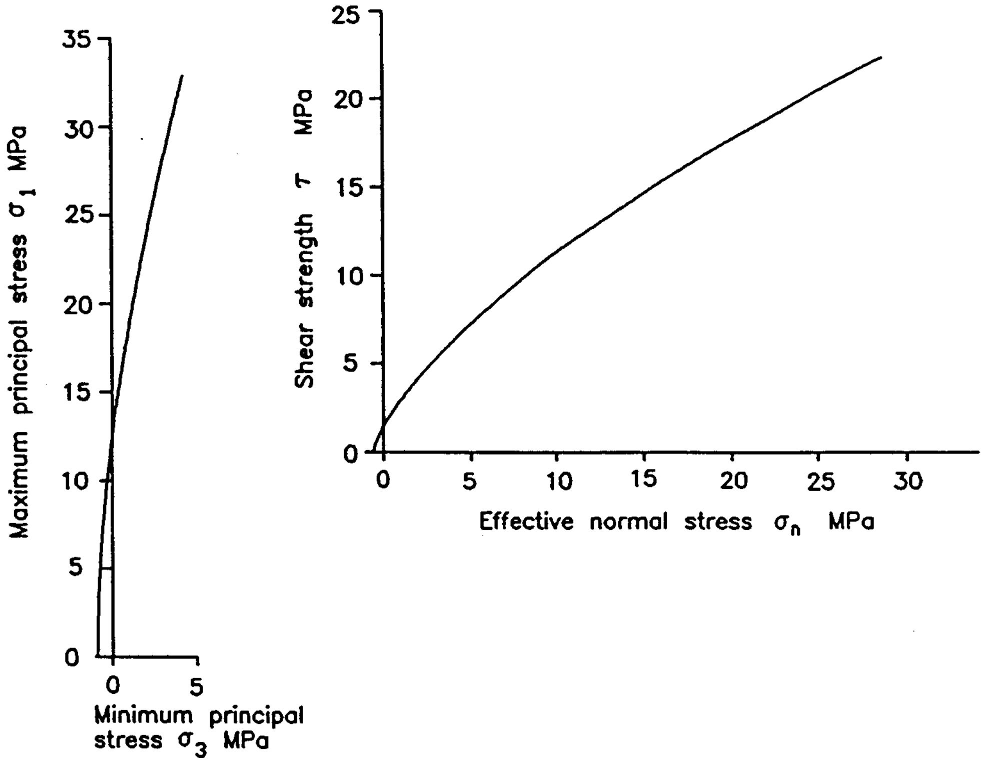

Figure 13.1: Plot of relationships between maximum and minimum principal stresses and normal and shear stresses for failure of a fair to poor quality siltstone. The properties of the siltstone are defined on the previous page.

13.3.1 Stresses around underground caverns near the toes of slopes

Since hydroelectric projects are frequently located in mountainous areas, the influence of surface topography upon the in situ stress field has to be taken into account in deciding upon the exact location of the underground powerhouse.

Figure 13.2 shows the maximum and minimum principal stresses in a gravitationally loaded slope with a far field horizontal to vertical in situ stress ratio of 3 : 1. The in situ stresses, particularly the minimum principal stress σ_3, are significantly altered in the vicinity of the slope face as compared with the far field stresses. These local changes in the in situ stress field influence the stresses induced in the rock mass surrounding an underground cavern located near the slope toe[1].

Figure 13.3 illustrates the results of a boundary element analysis in which an underground powerhouse cavern has been located at different distances from the toe of the slope analysed in Figure 13.2. Contours showing zones in which the tensile and shear strength of the rock mass have been exceeded are plotted in this figure. Failure trajectories in these overstressed zones indicate the direction in which failure of the rock would propagate, assuming the rock mass to be homogeneous.

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[1]These analyses were carried out using the elastic boundary element program EXAMINE^{2D} developed in the Department of Civil Engineering at the University of Toronto. The program is available from Rocscience Inc., 31 Balsam Avenue, Toronto, Ontario, Canada M4E 3B5, Fax 1 416 698 0908, Phone 1 416 698 8217, Email: software@rocscience.com, Internet http://www.rocscience.com.

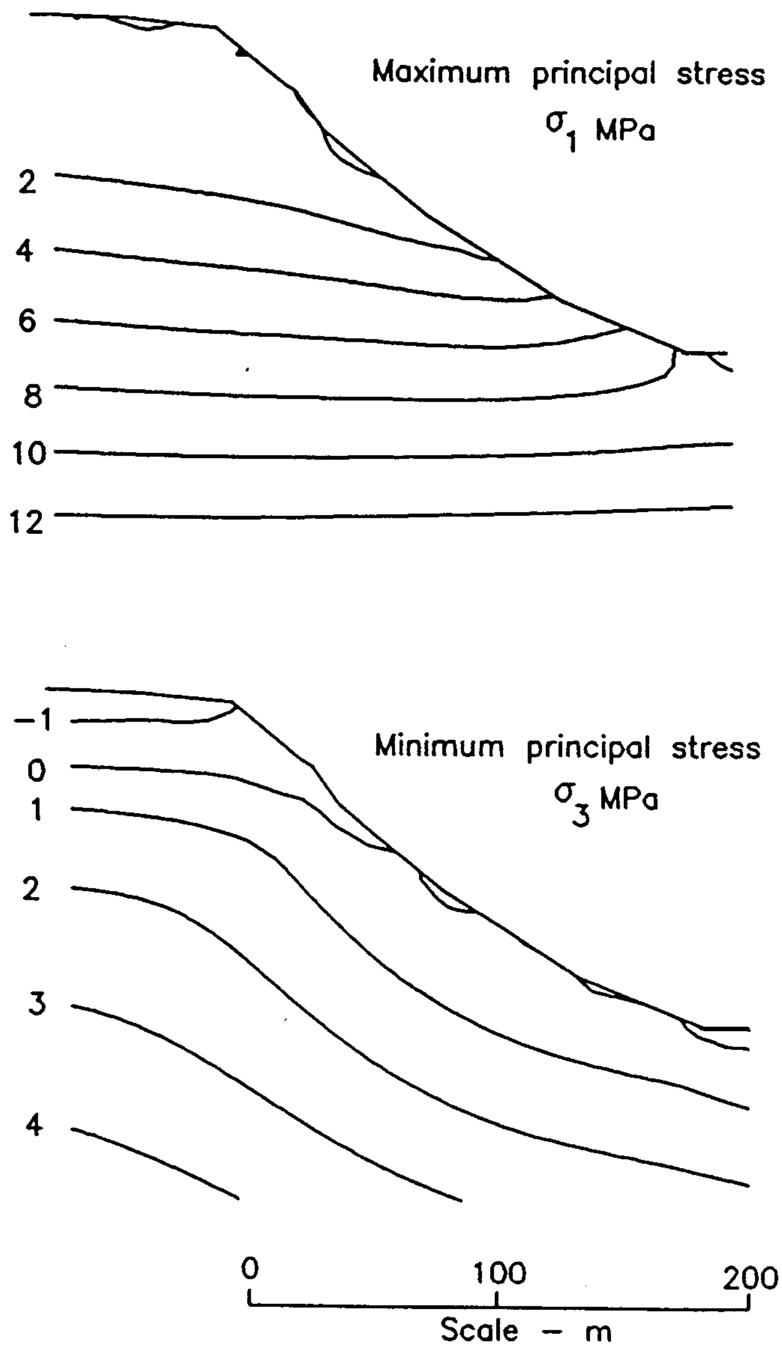

Figure 13.2: Distribution of maximum and minimum principal stresses in a gravitationally loaded slope with far field in situ stresses defined by a ratio of horizontal to vertical stress of 3:1

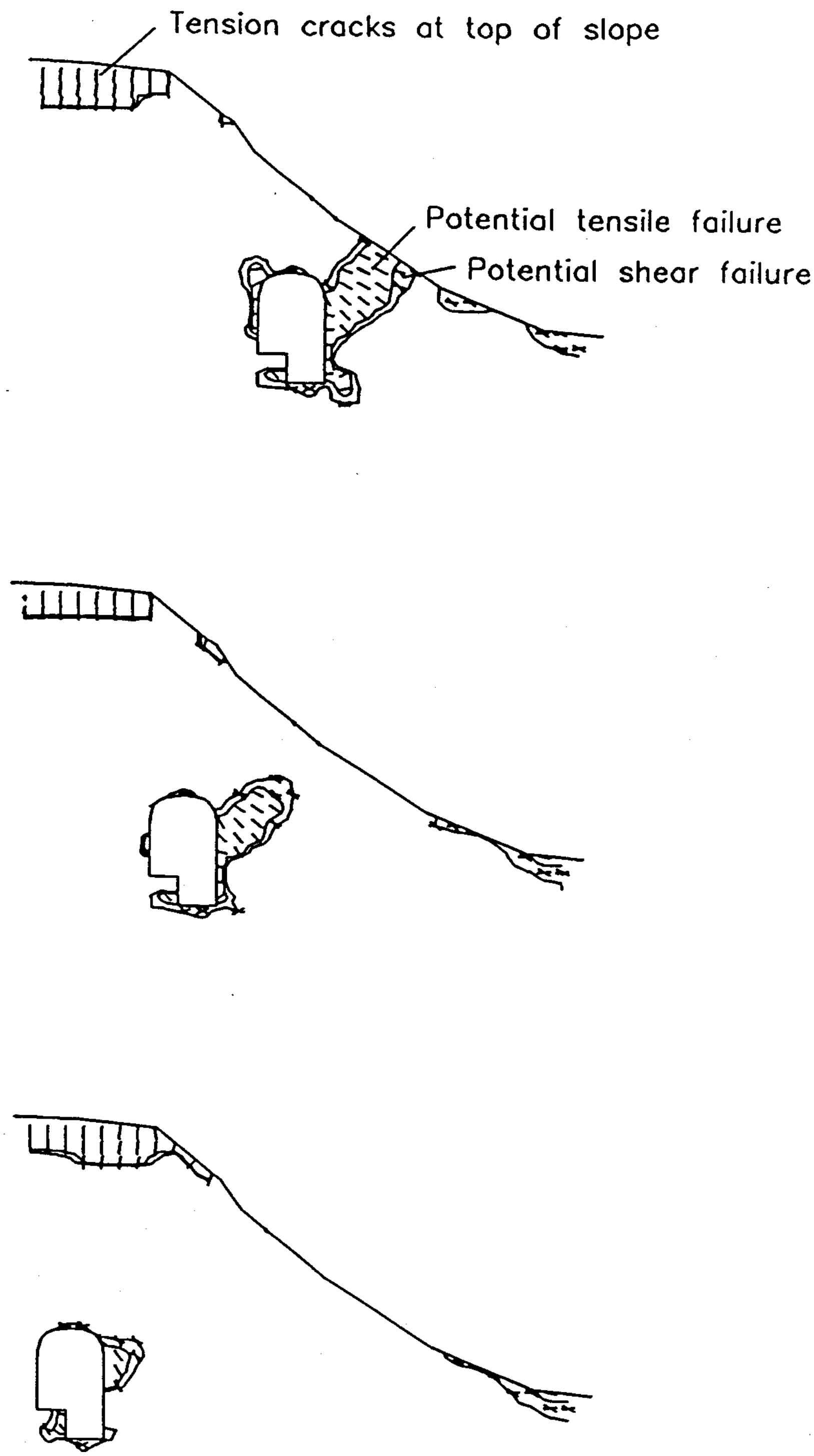

Figure 13.3: Zones of overstress and failure trajectories for a siltstone rock mass surrounding an underground cavern at different distances from the toe of the slope.

In all cases, vertical tension cracks would be generated at the crest of this particular slope and a minor amount of shear failure would occur near the toe. Such failures are common in slopes in weak rock masses and, once formed, are generally not a cause for concern since they are local in nature and result in stress relief and the re-establishment of equilibrium. Tension cracks, running parallel to the crests of slopes in sedimentary rock masses are common and have been observed to remain stable for many years. Usually, they have had no significant influence on the overall behaviour of the slope.

The zones of overstress in the rock surrounding the cavern are significantly different in the three cases illustrated. For the case in which the power cavern is closest to the slope toe, the extension of the overstressed zone to the slope face could result in the generation of local slope instability and could also result in the formation of a highly permeable zone between the slope face and the downstream wall of the cavern. In addition, the extent and the asymmetrical shape of the zone of overstressed rock suggests that substantial support in the form of long grouted cables would be required to stabilise the rock mass surrounding the cavern. The author considers this cavern to be too close to the toe of the slope and would recommend moving it further into the rock mass.

The zones of overstress in the case of the cavern located furthest from the slope toe are much smaller than for the other two cases and the rock mass surrounding this cavern could probably be stabilised with a relatively modest array of grouted cables.

In considering the three options illustrated in Figure 13.3, it must be remembered that geotechnical factors are not the only items which have to be considered in deciding upon the cavern location. Another important factor is the length of the tailrace tunnels. Hydraulic engineers usually want to keep these as short as possible in order to avoid the need for a downstream surge shaft to compensate for pressure fluctuations. Once the tailrace tunnel length approaches 100 m the need for a downstream surge shaft has to be considered.

13.3.2 Determination of steel lining length for pressure tunnels

Although not related to cavern design, the determination of the length of steel linings for pressure tunnels is an in situ stress related problem that deserves special consideration. The very high cost of pressure tunnel steel linings can sometimes impose significant constraints upon the feasibility of hydroelectric projects.

In most weak rock masses, concrete linings are required in all hydraulic conduits in order to provide protection against erosion of weak seams and to improve the hydraulic characteristics of the tunnels. Assuming that the tunnels have been correctly supported by means of grouted rockbolts and cables during construction, the concrete linings can be of minimum thickness and lightly reinforced. It is assumed that these concrete linings will crack under operating pressure and that they will be slightly leaky. Provided that the cracks do not propagate a significant distance into the rock mass and cause hydraulic jacking, the slight amount of leakage is not a problem. However, conditions which give rise to hydraulic jacking have to be avoided and this is usually achieved by the provision of an internal steel lining in the tunnel.

The criterion for deciding when a tunnel should be steel lined is when the minimum principal stress in the rock mass falls below the maximum dynamic water pressure in the tunnel. This is a function of the maximum static head of water in the tunnel, the operation of the gates and the characteristics of the turbines. An allowance of 20% over the maximum static head is usually considered adequate for a pressure tunnel associated with the operation of a Pelton wheel since this does not induce large pressure fluctuations. In the case of a Francis turbine, larger pressure fluctuations can be induced and an allowance of 30% above the maximum static head is normally used.

Figure 13.4 illustrates the case of a proposed pressure tunnel arrangement for a surface powerhouse at the toe of a slope. A headrace tunnel at elevation 680 m feeds water into a vertical shaft and then into a horizontal pressure tunnel at elevation 550 m. The maximum reservoir pool level is 780 m and this results in an internal static pressure in the upper headrace tunnel of (780 - 680) x 0.01 = 1.0 MPa, where the unit weight of water is 0.01 MN/m^3. Assuming that an allowance of 30% over this static pressure is required for a Francis turbine system, a steel lining will be required for any section of the upper headrace tunnel where the minimum principal stress in the rock mass is less than 1.3 x 1.0 = 1.3 MPa. As shown in Figure 13.4, these conditions require about 80 m of steel lining where the headrace tunnel passes beneath the valley upstream of the surge tank. An option which should be considered in this case would be to increase the grade of the headrace tunnel so that the cover depth in the vicinity of the valley is increased. A relatively modest increase of 15 to 20 m in cover depth would probably eliminate the need for the steel lining resulting in significant cost savings and the removal of a construction impediment.

For the lower elevation tunnel, the internal static pressure in the tunnel is given by (780 - 550) x 0.01 = 2.3 MPa. Allowing an additional 30% for dynamic pressure, the minimum principal stress at which the steel lining should commence is 3 MPa. As shown in Figure 13.4, the lower pressure tunnel should be lined from x co-ordinate 500 m to the surface powerhouse.

The minimum principal stress contours plotted in Figure 13.4 were determined by means of a boundary element analysis assuming a gravitational loading of the slope and a far field horizontal to vertical in situ stress ratio of 1.5:1. This analysis assumes that the rock mass is homogeneous and isotropic and that the minimum principal stress lies in the plane of the drawing. It is essential that an accurate topographic map of the surrounding area be checked to ensure that there are no valleys or low points in a direction normal to the plane of the drawing which could give rise to stress relief in that direction. If in doubt, similar stress analyses to that illustrated in Figure 13.4 should be carried out for other sections perpendicular to the tunnel axis to check whether the minimum principal stresses are low enough to cause problems.

Since these stress analyses assume ideal rock conditions and do not take into account possible leakage paths along faults, shear zones or other geological discontinuities, I strongly recommend that the findings of the type of stress analysis illustrated in Figure 13.4 be confirmed by hydraulic acceptance tests. These involve drilling boreholes from the surface to the points at which the steel linings are to be terminated, packing off the lower 1 to 3 m of the borehole and then subjecting the packed-off sections to hydraulic pressures increased incrementally up to the maximum dynamic pressure anticipated in the tunnel. The pump pressures and flow rates should be carefully monitored to determine whether any excessive joint opening or hydraulic fracturing occurs during the pressure tests. The test pressure should be maintained for at least an hour to establish that there is no significant leakage through the rock mass from the points at which the steel linings are to be terminated. Only when these tests have confirmed the theoretical calculations can the steel lining lengths be established with confidence. I am aware of several pressure tunnel failures where lining length calculations were carried out but no pressure acceptance tests were performed. Had such tests been done, they would have revealed anomalies or deviations from the conditions assumed in the calculations and these deficiencies could have been allowed for in deciding upon the steel lining lengths.

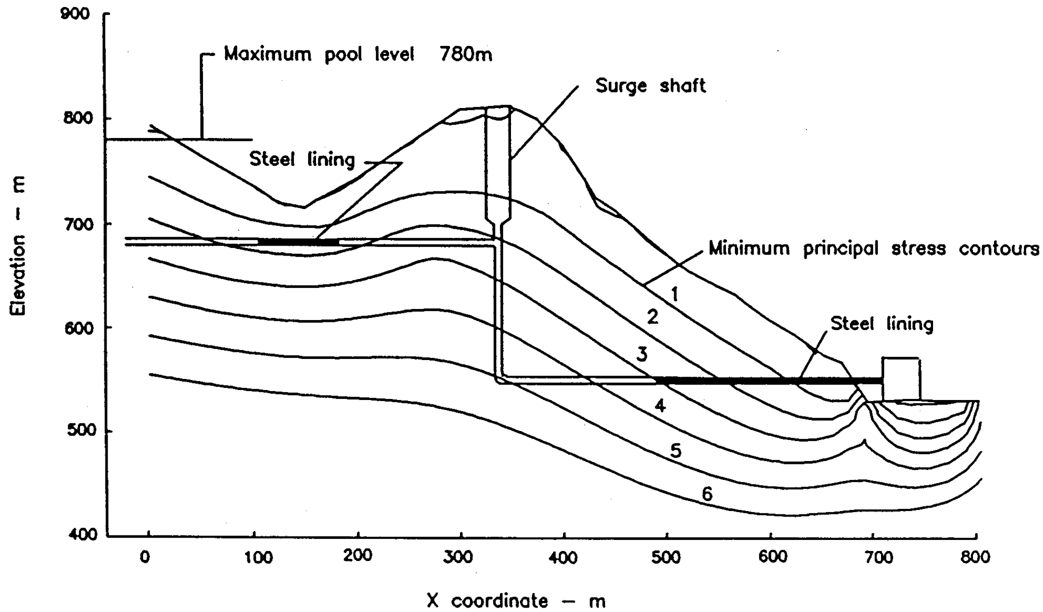

Figure 13.4: Contours of minimum principal stress σ_3 (MPa) in a gravitationally loaded slope with a far field in situ stress field defined by a ratio of horizontal to vertical stress of 1.5 : 1. Lengths of steel lining required in the pressure tunnel are shown.

13.4 Pillar size between excavations

In some cases, more than one cavern is required in an underground hydroelectric project. For example, the transformers may be placed in a smaller cavern parallel to the power cavern. This has the advantage of reducing the size of the main cavern and of isolating the transformers in case of fire. When this arrangement is used, there is frequently a demand from the electrical engineers to place the two caverns as close together as possible in order to reduce the length and hence the cost of the busbars that link the generators to the transformers. However, placing the two caverns close together may give rise to unfavourable stress conditions in the pillar between the caverns.

Figure 13.5 illustrates the results of a series of analyses in which the distances between two parallel caverns were varied. These analyses assume that the rock mass is a fair to poor quality siltstone, as defined earlier in this chapter, and that the ratio of horizontal to vertical in situ stress is 1.5:1. The caverns are located at a depth of 280 m below surface.

The contours plotted in Figure 13.5 define the ratios of the rock mass strength to the maximum principal stress induced in the rock mass surrounding the caverns. Zones defined by contours with strength/stress ratios of less than 1 are zones of potential rock mass failure. Because of the complex process of stress re-distribution associated with progressive failure of the rock mass surrounding the excavations, these zones of overstressed rock do not necessarily coincide with the actual extent of rock fracture. The zones of overstress do, however, give a reasonable basis for comparison and experience suggests that the designer should attempt to keep these zones as small as possible with a priority on minimising the extent of potential zones of tensile fracture[2].

In the case of the upper plot in Figure 13.5, the distance between the two caverns is approximately equal to one half of the height of the larger of the two caverns and the zone of overstress extends across the entire pillar. Note that the central portion of the overstressed zone has strength/stress values of less than zero and will be prone to tensile failure. This pillar would almost certainly suffer severe damage and would require very substantial support in the form of tensioned grouted cables spanning the width of the pillar.

The lower plot in Figure 13.5, with the pillar width approximately equal to 1.5 times the height of the larger of the two caverns, shows that the zones of overstress are of limited extent and that the core of the pillar has strength/stress ratios in excess of 4. This means that the stress fields surrounding the two caverns are almost independent of one another. The extent of the overstressed zones suggests that a relatively modest amount of support would be required to stabilise the rock mass surrounding the caverns.

The plot in the middle of Figure 13.5 represents a situation that could be considered a reasonable compromise for many underground hydroelectric projects. The distance between the two caverns is approximately equal to the height of the larger of the two caverns and this is generally acceptable in terms of busbar length. The stress fields surrounding the two caverns obviously interact to a certain extent but the zones of overstress are not so large that major changes in the support pattern would be required. The zone of potential tensile failure between the two caverns has been eliminated in this layout.

For caverns in weak rock masses such as those considered in this chapter, I recommend that pillar widths should not be less than the height of the larger of the two caverns, and that, wherever possible, they should be slightly greater. In very poor quality rock masses, in which the overstressed zones are larger, it may be advisable to increase the pillar width to 1.5 times the height of the larger cavern. In all cases, a comparative study similar to that illustrated in Figure 13.5 should be carried out in order to confirm these decisions.

13.5 Problems in using a concrete arch in weak rock

Many underground caverns have been constructed with roof support provided by a cast-in-place concrete arch. As illustrated in Figure 13.6, the cavern arch is excavated to its full width and inclined haunches are provided to carry the reaction of the concrete arch. The reinforced concrete arch is cast in place when the floor of the excavation is level with the bottom of the inclined haunches. The lower part of the cavern is then excavated, usually by benching downwards.

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[2] This analysis was originally carried out using the program EXAMINE^{2D} that is only suitable for elastic analyses. Since that time, new programs such as PHASE^2 have become available and these can be used for a full progressive failure analysis. It is recommended that analyses of the type described here should be carried out with a program such as PHASE^2, which is available from Rocscience Inc., 31 Balsam Avenue, Toronto, Ontario, Canada M4E 3B5, Fax 1 416 698 0908, Phone 1 416 698 8217, Email: software@rocscience.com, Internet http://www.rocscience.com.

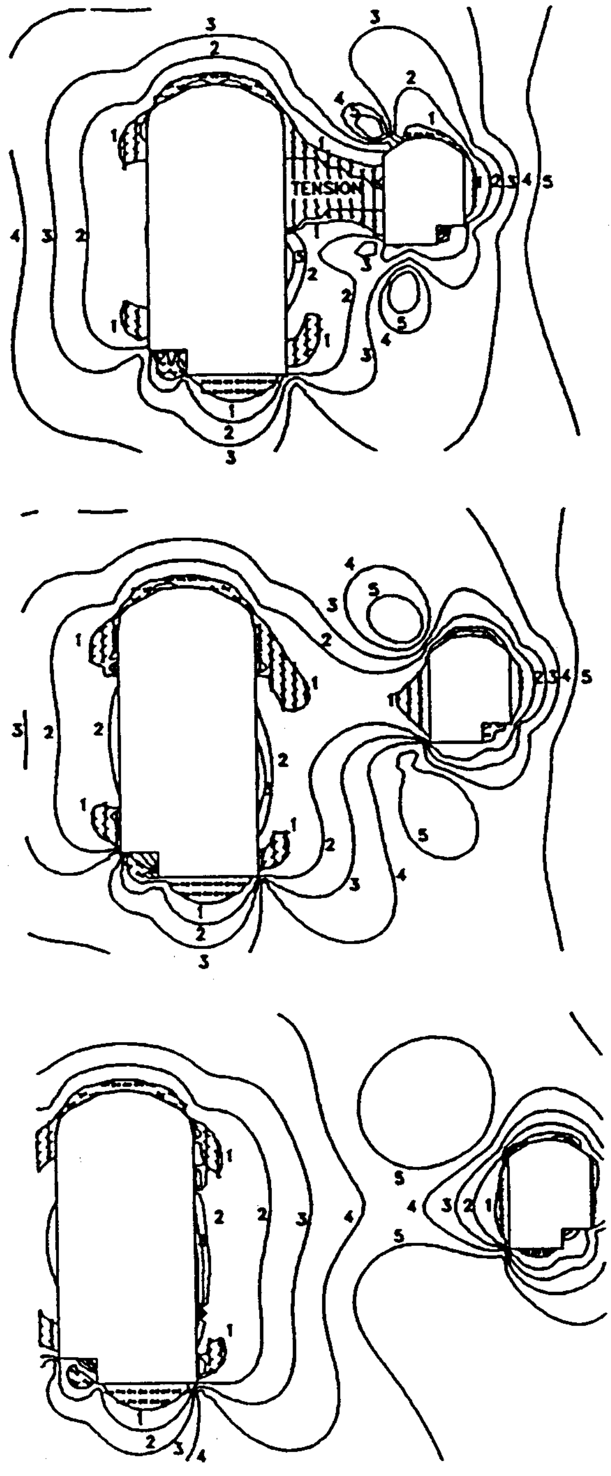


Figure 13.5: Contours of strength/stress ratios and failure trajectories in the rock mass surrounding two adjacent caverns in a siltstone rock mass with different spacing between the caverns.

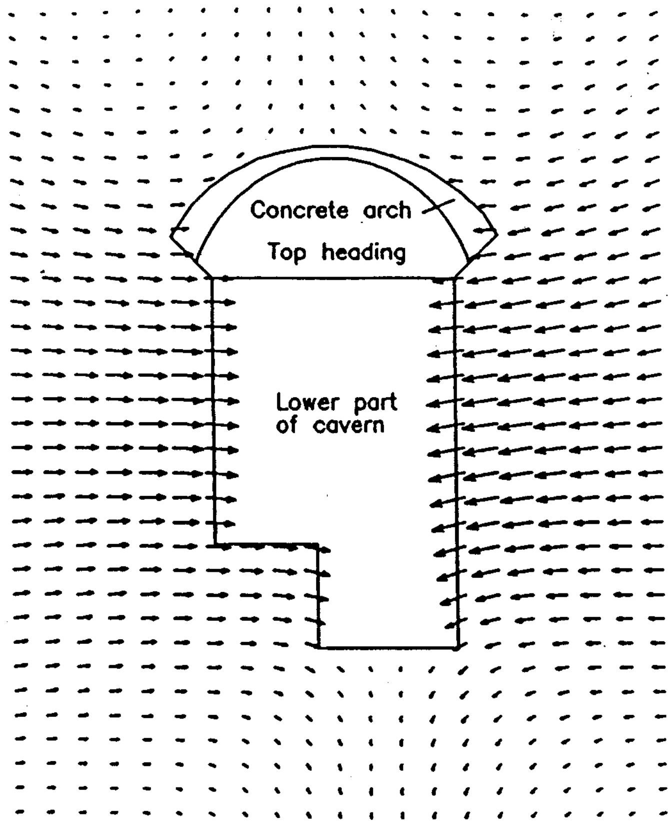


Figure 13.6: Plot of net displacements induced by excavation of the lower part of a large cavern in which the roof support is provided by means of a cast-in-place concrete arch.

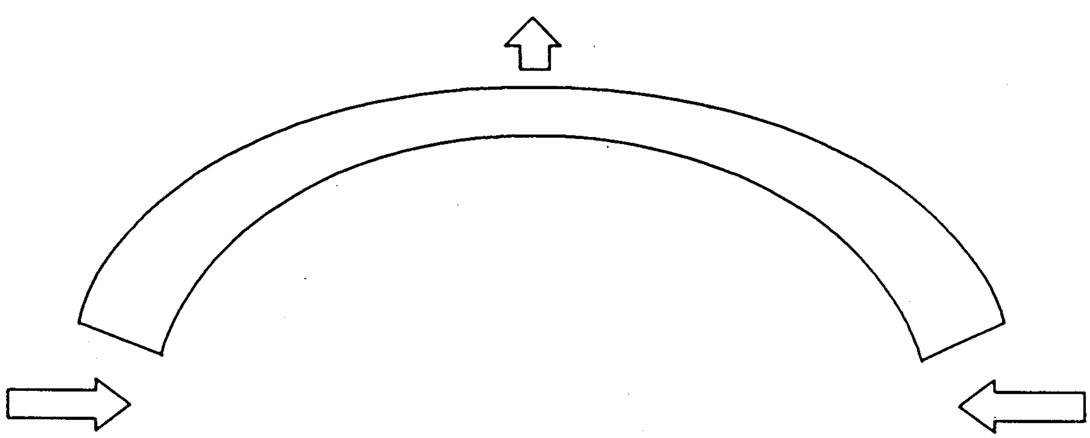


Figure 13.7: Displacements imposed on a cast-in-place concrete arch as a result of excavation of the lower part of the cavern

The concrete arch provides support for any rock that may become loosened in the cavern roof. The problem with this arch is that, if it is very rigid as compared with the surrounding rock mass, the deformations induced as a result of the excavation of the lower part of the cavern can cause excessive bending in the concrete arch. In addition, large amounts of temporary support in the form of rock reinforcement may be required to allow excavation to proceed to the stage where the concrete arch can be constructed. This reinforcement will have to be placed in the same manner as would normally be employed if it formed the permanent support of the rock arch. This would involve excavating a small top heading in the central crown, installing the reinforcement, widening the heading out into the haunches, installing more reinforcement, and so on. An example where pre-treatment and pre-support of a very poor quality rock mass was carried out to avoid temporary support problems during arch excavation is presented in a paper by Cheng and Liu (1990).

Figure 13.6 is a plot of the displacements induced by excavation of the lower part of the cavern. This plot was obtained by subtracting the displacements induced by the creation of the top heading from the displacements induced by the excavation of the entire cavern. These displacement vectors show that the upper part of the sidewalls displace inwards about 60 mm while the centre of the arch displaces upwards about 10 mm. Figure 13.7 is a diagrammatic representation of the displacements imposed on the concrete arch as a result of the displacements in the rock mass. Depending upon the magnitude of the displacements in the rock mass and the curvature and thickness of the concrete arch, the stresses in the concrete and in the reinforcing steel can exceed the safe working loads in these materials. This can give rise to critical conditions during construction since the repair of a damaged concrete roof arch is an extremely difficult and expensive process. Cases exist where severe cracking of the concrete arch led to the installation of additional steel arch support in local areas of the power cavern roof.

In general, I consider that the use of concrete roof arches should be avoided when designing large underground powerhouse caverns in weak rock masses. Experience has shown that the use of a more flexible support system such as that provided by grouted cables and a surface layer of shotcrete provides a more satisfactory solution. Where local problems occur, these can usually be dealt with by the installation of additional cables or the application of an additional layer of shotcrete. As long as access to the roof is maintained, remedial works can be carried out without disrupting the other construction activities in the cavern. Two schemes, with which I have been involved, have utilised platforms on top of a temporary crane to ensure that such access is available.

13.6 Crane beams

Underground powerhouse and transformer caverns require cranes of significant capacity to move heavy pieces of equipment around during installation and maintenance operations. Since these cranes are designed by structural engineers, it has been the practice that the cranes are supported on beams supported by columns. In many cases these structures are designed to be completely independent of the surrounding rock mass, just as they would be in a surface powerhouse structure. In my opinion, this is an inappropriate design approach since it does not utilise the enormous carrying capacity of the rock mass surrounding the cavern. Whenever possible the cranes beams should be suspended directly from the cavern walls as described in the examples given below.

The concerns which are normally expressed by designers, who are not familiar with the installation of suspended crane beams, are:

1. What capacity and length of cables or rockbolts are required to support the beams and what assurance can be given about the security of this support system?
2. How can displacements in the rock mass surrounding the cavern be accommodated in the case of crane beams attached to the walls?

The design of the rockbolts or cables required to support the beams follows the same procedure as would be used for the stabilisation of wedges in the roof or sidewalls of the cavern. The forces that have to be supported by the bolts or cables are calculated from the dead-weight of the beams themselves and the crane, loaded to maximum capacity. The crane load is distributed over the distance covered by the crane wheels along the beam. In addition, forces must be added to allow for the dynamic effects of the starting and braking of the crane, both along and across the cavern. These are simple force calculations that have to be carried out for any crane design.

Once the forces to be resisted by the rockbolts or cables are known, the capacity, length, number and orientation of the support elements can be chosen as would be done for the support of a rock wedge. The most important component of this support design is a thorough assessment of the geological conditions in the walls to which to beams are to be attached. The walls themselves should be stable and there should be no geological conditions, such as unfavourable oriented shear seams, which could cause instability as a result of the additional forces to be applied by the crane beam support.

Normally, long post-stressed cables are used for supporting the crane beams. However, there are several cases in which normal tensioned and grouted rockbolts have been used. Provided that the rock conditions are suitable, there should be no objection to the use of rockbolts for this purpose.

Many suspended crane beams have been used throughout the world and, provided that the design is carried out in a responsible manner, as described above, there is absolutely no reason why the system should be any less secure than a more traditional column crane.

On the question of displacements, this is very easily dealt with by providing the facilities for adjusting the position of the crane rail on top of the beam. The crane beams are normally cast in place when the cavern crown has been fully excavated and when the first bench has been taken. At this stage about 50 % of the displacement in the rock mass surrounding the cavern has already taken place. If the crane rails are installed at this stage, so that a construction crane can be used, provision must be made for the additional displacements that are induced as the cavern is benched downwards to its full height. Typically, these additional displacements will amount to a few centimetres on each wall. If the anchor plates holding the crane rails in place are provided with slotted holes to accommodate this additional displacement, there is no difficulty in adjusting the span between the crane rails as required. The anticipated displacements for each excavation stage can be estimated by numerical analyses as described elsewhere in these notes.

Some examples of typical suspended crane beans are given in Figures 13.8, 13.9 and 13.10.

In the Drakensberg Pumped Storage Project in South Africa, anchored crane beams were installed in about 1975 during excavation of the cavern. These beams were cast in place directly against the shotcrete layer which, along with rockbolts, formed part of the cavern support system. The rock in this case is a very poor quality interbedded sedimentary sequence of sandstones, mudstones, siltstones and shales. Corrosion protected cables, 15 m long and of 100 ton capacity, were installed at approximately 1.5 m spacing along the beam. Lateral displacement of the beams, both during and after construction, was accommodated by providing slotted holes in the crane rail fixtures. The system has operated without problems for over 20 years.

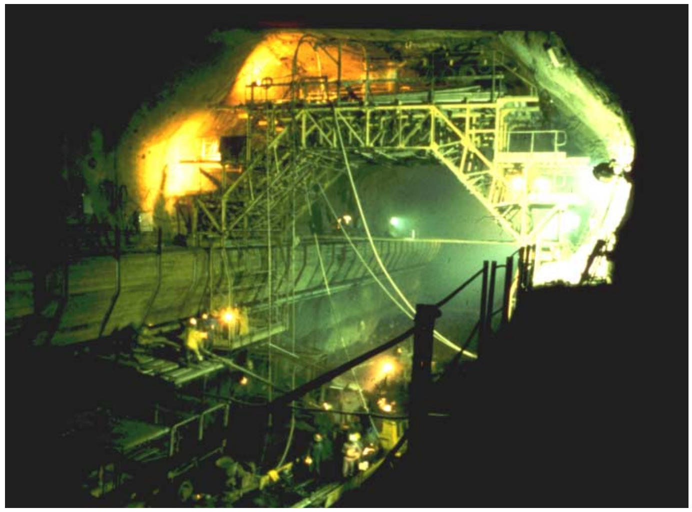


Figure 13.8: Example of a cast in place crane beam in the Drakensberg Pumped storage project in South Africa. The beam is supported against a vertical face of interbedded sandstone, siltstone and mudstone by means of stressed and grouted cables. A temporary construction crane that was used to access the roof is shown in the photograph.

In the Singkarak Hydroelectric Project in Indonesia, anchored crane beams have been installed directly against the rock face, which is gneiss of reasonable quality. This installation is shown in the photograph reproduced in Figure 13.9, taken during construction of the cavern.

In the case of the Thissavros project in Greece, illustrated in Figure 13.10, the rock mass is gneiss but the cavern is oriented parallel to the strike of two major discontinuity systems. Hence, while the cavern walls were supported by 6 m long rockbolts, it was felt that the additional forces from the fully loaded crane could induce wedge instability in the walls. For this reason, the cast in place suspended crane beams were used only for a light construction crane, as illustrated in Figure 13.10. Concrete columns were then added, once the benching of the cavern was completed, to carry the full crane loads.

The principal reason for the choice of this system on many projects is that it allows for early installation of the cranes. Frequently, a light construction gantry, which runs on the crane rails, is installed immediately after the construction of the crane beams. This provides access for the monitoring of instruments in the cavern roof and for repair work to the roof support system if required. The main crane can be assembled early in the schedule and it is then available to assist in excavation of the lower benches and in the concrete work in the base of the cavern. These scheduling advantages offer considerable benefits when compared with a normal column supported crane.

Figure 13.9: Example of suspended crane beams in the powerhouse cavern of the Singkarak Hydroelectric Project in Indonesia.

Figure 13.10: Example of anchored crane beams in the underground powerhouse cavern for the Thissavros Hydroelectric project in Greece. In this case the beams shown were used for a construction crane and concrete columns were added later to carry the full crane loads.

13.7 Choice of cavern shapes

In strong rock masses, for which rock mass failure is not a problem, the conventional shape chosen for an underground powerhouse cavern is similar to that illustrated in Figure 13.11. The arched roof provides stability in the rock above the cavern roof and also provides convenient headroom for an overhead crane. The sidewalls are simple to excavate by vertical drill-and-blast benching and provide clean uncomplicated walls for crane column location and the accommodation of services.

The problem with this cavern shape when used in weak rock masses, particularly with high horizontal in situ stresses, is that tall straight sidewalls are deflected inwards (see Figure 13.6) and tensile failure is induced as shown in Figure 13.11. Zones of failure are illustrated in this figure and the maximum sidewall movement is 38.5 mm. The stabilisation of the rock mass surrounding this cavern will require significant reinforcement in the form of grouted cables or rockbolts.

An alternative cavern shape is illustrated in Figure 13.12. This elliptical shape has been used on schemes such as Waldeck II in Germany (Lottes, (1972)) and Singkarak in Indonesia (see Figure 13.9). As shown in Figure 13.12, the dept of the failure zones in the sidewalls has been reduces as compared to that in the conventional cavern. This results in a more stable overall cavern and a reduced support requirement.

While this cavern shape is better from a geotechnical point of view, it has some practical disadvantages. The cavern shape is such that the construction has to be more carefully executed than the conventional straight-walled cavern and items such as the cranes and services have to be designed to fit into the cavern shape. These differences can create significant problems where the skill of the labour force is limited.

In the weak rock schemes in which I have been involved, the conventional cavern shape has been chosen in preference to the elliptical shape because the overall advantages of the elliptical cavern have not been considered of critical importance when compared with the simplicity of the conventional cavern shape. Present support design techniques, discussed later in this chapter, are relatively unsophisticated and the stress changes resulting from a change in cavern shape are probably too small to have very much impact upon the support design. Consequently, it is doubtful whether the overall costs and time involved in the construction of conventionally shaped caverns would be higher than those that would be incurred in constructing elliptical caverns.

I recommend that each scheme should be investigated on its own merits, taking into account construction problems as well as geotechnical factors. In some cases, the use of an elliptical cavern shape may be justified but, in general, the conventional cavern shape illustrated in Figure 13.11 will be found suitable for all but the very weakest rock masses.

Before leaving this topic, attention is drawn to the unfavourable stress and potential failure conditions created by the stepped base of the cavern. A step of some sort is generally required to house the draft tubes and the lower parts of the turbines. It could be argued that this step should be created by cast-in-place concrete after a cavern of optimum shape has been excavated. In practice, this somewhat theoretical approach is found to be both unnecessary and uneconomical since failure in the base of the cavern is relatively easy to control. The instability of this bench is due to stress relief resulting from the creation of an unsupported vertical face and minimal support in the form of untensioned grouted steel rods (dowels), installed from the cavern floor before excavation of the lower benches, will counteract this instability.

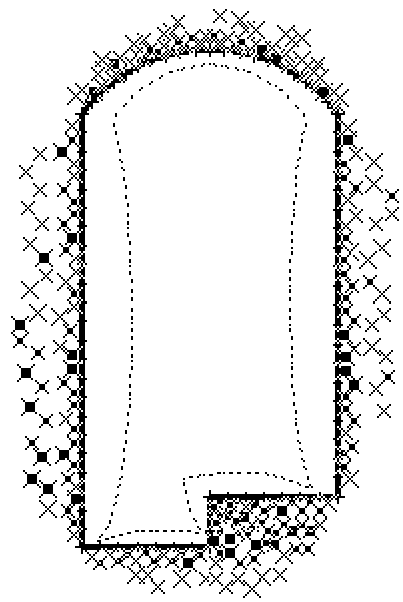


Figure 13.11: Conventional arched roof cavern shape showing zones of failure at a depth below surface of 280 m with a horizontal stress of 1.5 times the vertical stress. The deformed excavation shape is shown and maximum sidewall displacement is 38.5 mm. Analysed using PHASE[2].

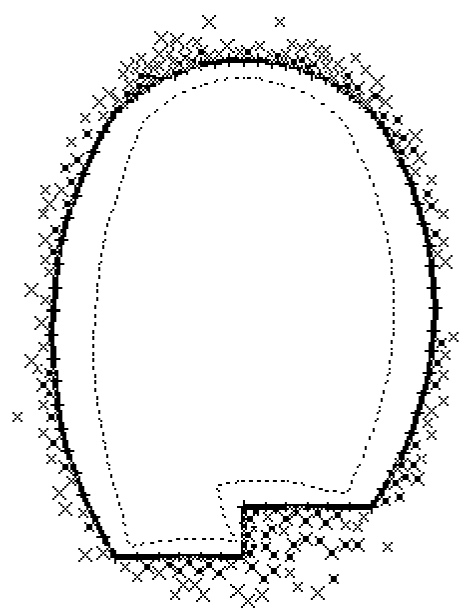


Figure 13.12: Elliptical cavern showing zones of failure for the same rock mass strength and in situ stress conditions as for the conventional cavern illustrated in Figure 13.11. The maximum sidewall displacement is 33 mm.

Good blasting practice is helpful here as damage to the bench can be reduced significantly by pre-splitting the bench face when the general level of excavation is still 2 or 3 m above the top of the step. Damage caused by pre-split blasting is confined to the rock above the step that is removed subsequently by the final blast down to the elevation of the step. The potential shear failure of the cavern floor and the lower part of the downstream sidewall does not pose any serious threat since it is self-supporting and because the lower part of the cavern is generally filled with concrete to form the turbine foundations, drainage tunnels and other service structures.

13.8 Influence of joints and bedding planes

In all of the analyses presented so far it has been assumed that the rock mass is weak but that it is isotropic and homogeneous. In other words, there are no dominant weakness directions in the rock mass. These assumptions are seldom valid in actual rock masses since joints and bedding planes are usually present, particularly in the case of sedimentary rocks. While these features do not have the same impact as faults or shear zones, they can introduce a directional pattern of weakness in the rock mass and this should be taken into account in the cavern design.

Since each rock mass will have its own unique set of structural features, it is essential that an analysis be carried out for each and every project. This chapter is concerned with general concepts rather than specific details and a simple example that illustrates these concepts is presented in Figure 13.13.

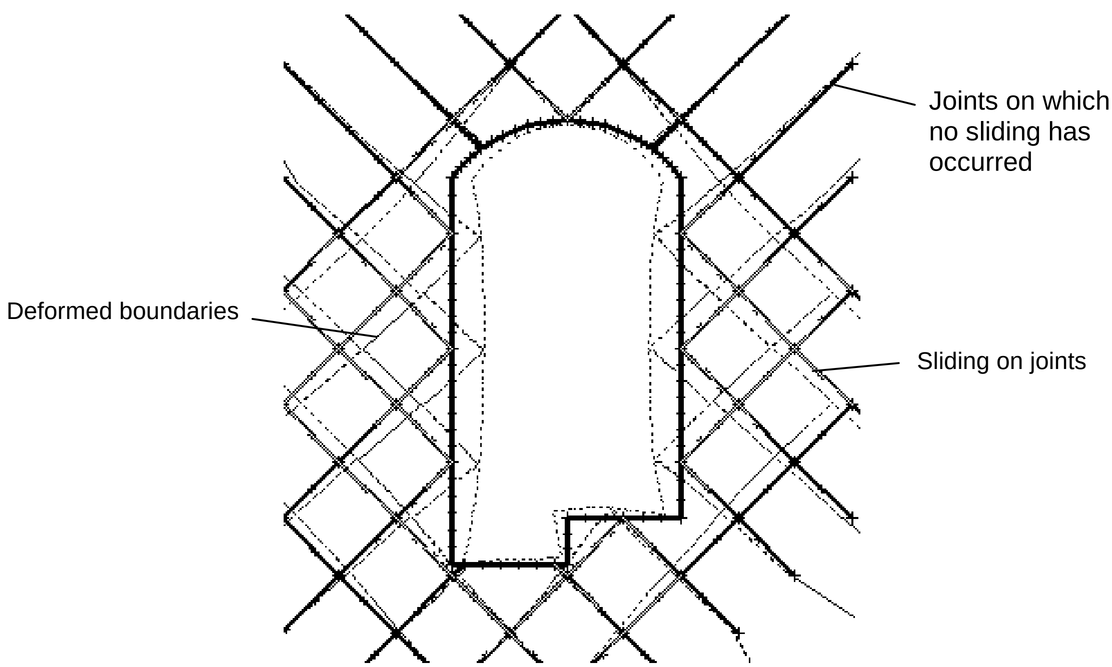


Figure 13.13: Influence of two sets of joints, inclined at 45°, on the displacements on the rock mass surrounding a conventional cavern. The rock mass properties and the in situ stresses are the same as those assumed in the previous section. The joints have a cohesive strength of 0.2 MPa and a friction angle of 30°.

The rock mass is assumed to be the same fair to poor quality siltstone as defined earlier and used in all previous analyses. Two sets of joints have been assumed and the shear strength of these discontinuities is defined by a Mohr-Coulomb angle of friction $\phi = 30°$ and cohesive strength of c = 0.2 MPa.

The influence of the joints in this case is dramatic in that all failure has been concentrated on the joints and there is no yield in the blacks separated by the joints. In other words, the problem has been transformed into one of structurally controlled failure as discussed in Chapter 6.

This transformation is controlled by the low shear strength of the joints as compared with that of the rock material. While the friction angles are almost the same, the cohesive strength of the joints is 0.2 MPa while that of the rock is 4 MPa. This means that the failure will occur preferentially on the joints whenever it has the chance to do so. However, in situations in which the joint strength is closer to that of the rock material forming the blocks in between the joints, the failure will tend to be more evenly spread between the two.

The inclination of the joints with respect to the principal stress directions is also important. Joints inclined to the direction of the maximum principal stress will tend to slide more easily than those parallel and normal to this direction.

The lessons to be learned from this rather simple example are that each case has to be considered carefully to establish whether it should be treated as a structural failure problem or whether it can be assumed that the rock mass is sufficiently homogeneous that the concepts discussed earlier in this chapter can be applied.

13.9 Design of reinforcement

The aim of any underground support design should be to help the rock mass to support itself to the greatest extent possible. This involves ensuring that the shapes and layout of excavations have been optimised, that the rock is damaged as little as possible during excavation, that the support characteristics have been carefully chosen to match the behaviour of the rock and that the sequence of excavation and support installation have been taken into account in the support design. There is no one ‘correct’ way to design support and there are many possible solutions which could be applied equally successfully to each underground cavern project. The support design concepts presented in this chapter are those which I have found to work effectively and economically in both mining and civil engineering applications.

A good starting point for any support design is a study of the literature to determine what others have done in similar circumstances. These precedents have also been usefully summarised in the rock mass classification schemes of Bieniawski (1989) and Barton, Lien and Lunde (1974). Figure 13.14, adapted from Barton (1989), gives the type of support which has been successfully used in underground excavations in weak rock masses. In this figure, *fibercrete* is an abbreviation for steel fibre reinforced shotcrete and *bolts* refer to either grouted bolts or cables which may or may not be tensioned, depending upon the sequence of installation.

Note that the design of cast concrete linings, shown in the upper left hand segment of Figure 13.14, must take into account the relative deformability of the concrete and the surrounding rock mass. As discussed earlier, the choice of an inappropriate shape and thickness for a concrete lining can result in serious overstressing of the concrete. This problem is more acute for a cavern roof than for a tunnel lining. The roof arch is vulnerable to deformations resulting from benching after its construction and there is no opportunity to close the arch by constructing a concrete invert, as is commonly done for a tunnel lining.

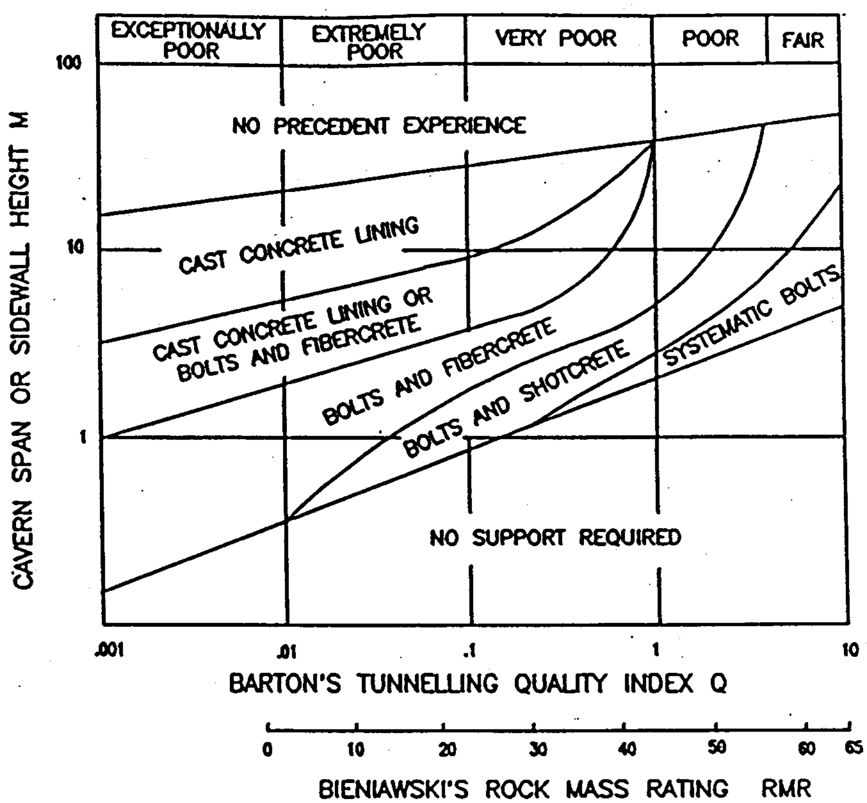


Figure 13.14: Summary of precedent experience on support of weak rocks in terms of rock mass classifications. Adapted from Barton (1989).

I unreservedly recommend the use of one of the existing classification schemes to obtain a first approximation to the level of support necessary for a large cavern in a weak rock mass. However, caution is advised against the uncritical acceptance of the recommendations coming from them. The collection of the data necessary to calculate RMR or Q values for a rock mass usually provides enough information for the design engineer to formulate a clear impression of the particular modes of failure that his design must resist. Careful consideration of the potential behaviour patterns of the rock mass, particularly the influence of geological structures, may lead to significant modification of the support recommendations obtained from the application of classification systems.

13.9.1 Estimating support pressures

One approach in estimating support pressure is to consider the extent of the zones of overstressed and blast damaged rock above the cavern roof and to assume that this rock acts as a dead weight which has to be supported. Considering the example illustrated in

Figure 13.11 for a rock mass with an RMR value of 48 and Q = 1.5, the zone of overstress extends approximately 3 m into the rock mass above the crown of this 20 m span cavern.

In addition to the stress induced fracturing that may occur, the fracturing and loosening of the rock mass due to blasting should also be considered. From experience it is suggested that blast damage may extend 1.5 to 3 m into the rock adjacent to the roof depending upon how much care has been taken to control the blasting.

Assuming that 3 m of rock have been damaged by either stress induced or blast induced fracturing, a dead weight of broken rock of up to 8 tonnes/m^2 of exposed surface of the cavern roof has to be supported. Where this support is to be provided by means of rockbolts or cables, a factor of safety of 1.5 to 2 is usually allowed to account for installation problems and to provide some reserve support capacity. Hence, the total capacity of the installed bolts or cables should be of the order of 12 to 16 tonnes/m^2 (0.12 to 0.16 MPa).

Figure 13.15, adapted from a paper by Barton (1989), shows that this estimate of a support pressure of 12 to 16 tonnes/m^2 for a rock mass with Q = 1.5 is in line with recommendations based upon previous experience.

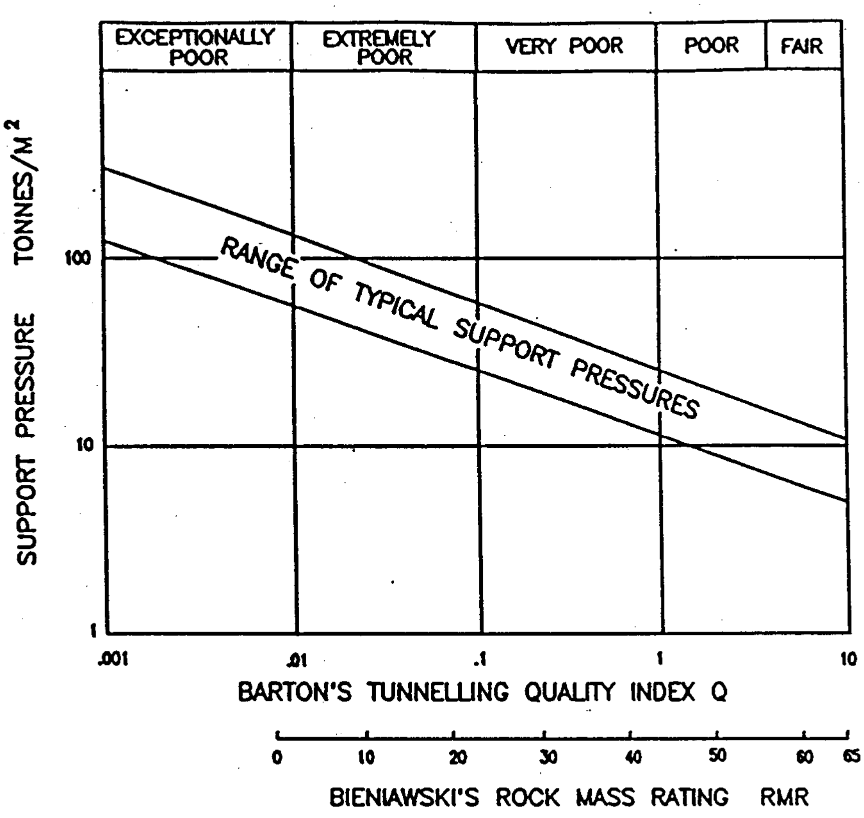


Figure 13.15: Relationship between support pressure and rock mass quality. Adapted from Barton (1989).

Some simplified closed form theoretical solutions have been developed which permit the study of the interaction of different types of support with the zone of failed rock surrounding underground excavations, and these solutions have been summarised by

Brown et al (1983). All of these solutions consider the development of a ‘plastic’ failure zone in a homogeneous rock mass surrounding a circular tunnel in a hydrostatic stress field and these assumptions impose severe limitations upon the application of these solutions to practical rock support design. Nevertheless, these simplified models have proved to be very useful in the development of an understanding of the basic concepts of rock support interaction and I urge interested readers to become familiar with these models and concepts. A discussion on this topic together with a listing of the simple calculation steps required can be found in Hoek and Brown (1988).

Today, the availability of powerful numerical analysis tools such as FLAC and PHASE^2 makes it possible to study support options in great detail. Such analyses are no longer constrained by the capability of the programs but rather by the quality of the input information. The reader should avoid the temptation to believe the results of a single analysis, however convincing the output may appear. It is always necessary to carry out parametric studies to cover the range of possible input parameters. It is only in this way that a sound understanding of the behaviour of the rock mass surrounding the excavation can be established.

13.9.2 Design of rockbolt and cable support

The excavation of a large cavern in weak rock will usually require the installation of systematic rock support or reinforcement during excavation as an integral part of the excavation cycle. Even when the use of cast concrete arches and/or sidewalls is envisaged for final support, consideration must be given to the requirements of temporary stability during excavation, which will often require extensive support if safe and controlled excavation is to be achieved. In many cases, the only significant difference between supports used for temporary and permanent purposes relates to corrosion protection requirements. Corrosion protection is an important design consideration in environments where corrosive water is encountered during preliminary investigation or excavation. In modern practice, corrosion protection measures usually require at least one physical barrier (single corrosion protection) to protect the individual rockbolts or rock anchors against corrosion (British Standards, 1987). This physical barrier often takes the form of a corrugated plastic sheath which completely encapsulates the steel bar or cable from which the anchor is formed.

Rock mass reinforcement for large caverns usually involves the installation of rockbolts formed from deformed steel bar or cables made from pre-stressing strand. Rock bolts are generally cheaper and quicker to install but cables can provide higher capacity and may be easier to install when long reinforcement elements are required. Combinations of rockbolts and cables are commonly used to combine the best features of both systems. The rockbolts are installed close to the face for immediate support while the cables are installed subsequently as the primary reinforcement system.

The possibility has to be considered that, during excavation, cables will pick up excess load over their installation load to the extent that they become seriously overstressed. In this circumstance, prediction of the final cable load is an important input into decisions on the appropriate installed load, the timing of the installation and tensioning and the choice between fully grouted or adjustable cables. In the author's experience, fully grouted cables are more convenient for the contractor and provide stiffer support, particularly in response to deformations occurring at a large angle to the cables.

However, the use of adjustable (i.e. re-tensionbable) cables may be appropriate in circumstances where very large deformations are anticipated. These cables are made by placing a plastic sheath over a significant portion of the length of the cable. This sheath breaks the bond between the cable and the grout and allows the cable to deform independently of the surrounding rock.

It is important to recognise that there are two types of rockbolt or cable support commonly used in underground excavations. In good quality rock masses in which the stability is controlled by intersecting joints, bedding planes, shear zones and faults, the support has to be designed to reinforce specific blocks and wedges which may fall or slide into the excavation. This type of support, frequently referred to as 'spot bolting', involves the installation of a few bolts or cables at clearly defined locations, with their length, orientation and capacity chosen to provide adequate support for the wedge or block under consideration.

While 'spot bolting' may be required for isolated blocks or wedges in weak rock masses, it is the second type of support, frequently called 'pattern bolting', which is more relevant to this chapter. 'Pattern bolting' involves the installation of rockbolts or cables in a regular pattern that is designed to reinforce the entire rock mass in much the same way as reinforcing steel acts in reinforced concrete. Typically, 5 m long 20 tonne capacity rockbolts installed in a 2 m x 2 m grid pattern could be used over the entire roof and wall area of a large underground cavern. These bolts would provide a support pressure of 2.5 tonnes/m^2 if loaded to 50% of their capacity.

The first question to be decided is the length of the rockbolts or cables. Analyses of the extent of zones of overstressed rock, such as those presented in Figures 13.11 and 12.12, are useful in determining the approximate extent of rock requiring support. Generally, the bolts or cables should extend 2 or 3 m beyond the limit of the zone of overstressed material. As previously stated, great care must be taken in using this approach to select reinforcement lengths since relatively modest changes in rock mass properties or in situ stresses can result in significant changes in the zones of overstress. Consequently, parametric studies in which these input data are varied over the maximum credible range are essential if the reinforcement lengths are to be based upon such studies.

An alternative approach is to use previous experience. Figures 13.16 and 13.17 give the lengths of roof and sidewall rockbolts and cables in some typical large powerhouse caverns in weak rock masses. Plotted on the same graphs are empirical relationships suggested by Barton (1989) for bolts and cables. For underground powerhouse excavations these relationships are simplified to :

Roof	rockbolts	L = 2 + 0.15 x SPAN m
	cables	L = 0.4 x SPAN m
Walls	rockbolts	L = 2 + 0.15 x HEIGHT m
	cables	L = 0.35 x HEIGHT m

The choice of rockbolt and cable spacing is based upon the following considerations:

a) In order to ensure that the bolts or cables interact with each other to form a zone of uniformly reinforced rock (Lang, 1961), the spacing S of the bolts or cables should be less than one half of the length L, i.e. S < L/2.

b) For a support pressure P and a working load in the bolt or cable T, the spacing for a square grid is given by $S = \sqrt{T/P}$.

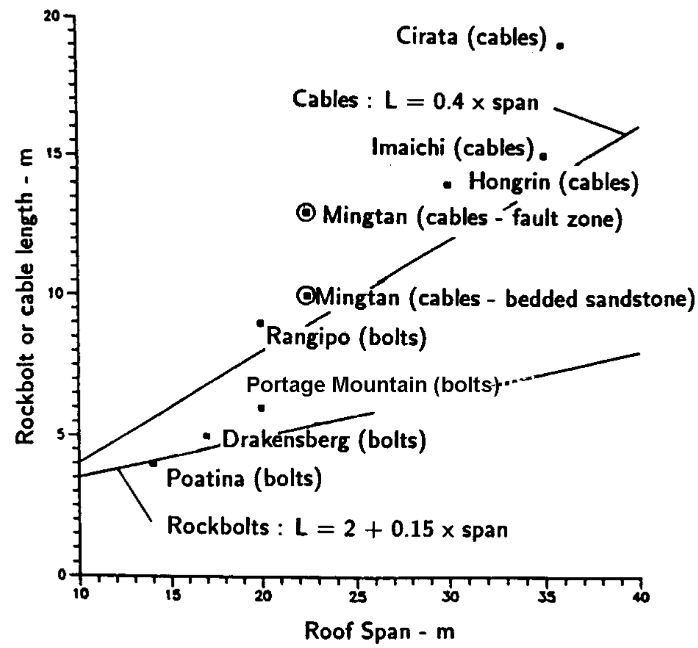


Figure 13.16: Rockbolt and cable lengths for roof support in some large caverns in weak rock.

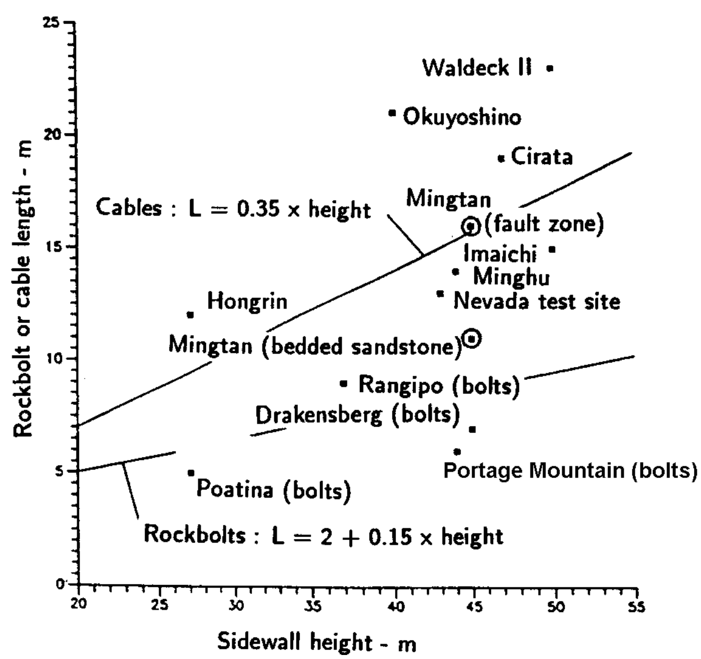


Figure 13.17: Rockbolt and cable lengths for sidewall support in some large caverns in weak rock.

Typical bolt and cable spacing range from 1 to 3 m with 1.5 m being a common spacing for bolts and 2 m being widely used for cables. Where additional support capacity is required to support local areas of weaker rock, bolts or cables placed at the centre of each grid square will sometimes suffice. Alternatively, when cables are used, additional strands can be placed in each hole to increase the capacity of the cables.

13.9.3 Use of shotcrete linings

In the past two decades, shotcrete has developed into a versatile support system that is ideally suited to the requirements of cavern support in a deformable rock mass.

Two significant benefits of shotcrete are that it can be applied quickly to freshly exposed rock during excavation and that it develops strength steadily after application. Green shotcrete is resilient to damage from nearby blasting and associated stress redistribution and gains strength at the same time as load is being transferred onto it. High early strength can be achieved by the addition of accelerators without a serious loss in long-term strength. Dosage of the accelerator has to be controlled carefully to ensure good intermixing and avoid local overdosing.

When used in combination with systematic rock reinforcement, shotcrete can provide immediate temporary support during excavation or form the permanent lining for the roof and sidewalls during the design life of the cavern. Irrespective of the form of temporary support, shotcrete application for permanent support purposes can often be delayed until all adjacent excavation has been completed. This allows the permanent lining to be sprayed in a better working environment than that existing close to the working face and this allows better control over thickness and quality. Some designers prefer to utilise additive free shotcrete for final lining application on the grounds that high early strength is not needed for the final lining and the addition of additives may be detrimental to the long-term performance of the shotcrete.

Research and development into shotcrete mixes, additives and equipment have progressed to the stage that shotcrete quality now rests almost entirely with the choice of compatible equipment and the equipment operators. Wet mix application requires careful attention to the supply of mix and air to ensure that the material leaves the nozzle in a continuous uninterrupted stream that can be applied by the operator or nozzleman in such a fashion as to maximise compaction and minimise rebound. With the dry mix method the supply of water also has to be controlled by the nozzleman.

For the engineer who is not an expert in shotcrete technology, there is a bewildering choice of equipment and additives, starting with the basic selection of the wet mix or the dry mix method. No hard and fast rules apply to this selection, other than to say that, depending on local circumstances, either system may be suitable to form the temporary or permanent lining of a large cavern. In general, higher production rates are possible with the wet mix processes, but this is not necessarily a major factor in considering the system to be used for cavern lining purposes. Remotely controlled robot applicator systems are widely used, but hand held nozzles will also provide a satisfactory product, if used properly. Liquid plasticizers and water reducing agents are commonly used as an aid to application and dust suppressants have recently been developed to improve the working environment for the nozzleman. It is common policy to limit the total amount of all additives in a mix to a figure of the order of 5 or 6% by weight of cement.

A significant advance in shotcrete technology has been brought about by the introduction of micro silica into the mix at up to 10% by weight of cement. This results in a significant reduction in rebound, and an increase in the thickness that can be built up in a single application. It is also beneficial for application onto damp or wet surfaces. The addition of micro silica produces a denser product with an increase in early strength, and does not appear to have a detrimental effect on long-term strength. Problems with the use of steel fibre reinforced shotcrete have been greatly reduced by the use of micro silica. Rebound of both shotcrete and the fibres is significantly less than it used to be. Balling of the steel fibres and excessive equipment wear have also been largely overcome so that, over the last five years, micro silica fibre reinforced shotcrete technology has become a viable and frequently preferable option to the accepted use of steel mesh embedded in plain shotcrete.

For steel fibre shotcrete, the number of shotcrete applications can sometimes be reduced when compared with the more complex installation of layers of plain and/or mesh reinforced shotcrete. Although the initial bending strength of the two products is similar, performance is improved because the post crack load bearing capacity is significantly better in the case of steel fibre shotcrete. Typical current practice involves the use of fibres in the range of 20 to 40 mm long and approximately 0.5 mm diameter. Current research is examining the use of longer fibres (to improve post crack strength further) and materials other than steel.

Decisions on lining thickness are usually based on a combination of empirical and practical considerations rather than concern about stress levels in the lining. Where concern about stress does exist, delayed application of the final layer or the application of an additional layer are available options. The thickness built up in a single application is typically of the order of 40 to 80 mm and total thickness of the order 100 to 200 mm. With layers less than 40 mm thick concern will sometimes exist that an effectively continuous layer will not be achieved if application to a very irregular surface is required.

When designing permanent shotcrete linings, it is important to specify quality control or acceptance tests as a design requirement. The use of steel fibre shotcrete does not readily allow rigorous checking during the execution of the work in the manner that rockbolt work or reinforced concrete construction does, since it is dependent on the skill of the operator. Thus, it is essential to do routine acceptance testing by coring through the completed lining in order to check the density and strength of the sprayed product, the adhesion to the rock surface, the inter-layer adhesion where two or more layers have been applied, and the total thickness achieved. Inter-layer adhesion can be a particular problem if a long time period elapses between the application of temporary support and permanent support shotcrete, since it is difficult to remove the grime that accumulates on the surface of the initial layer if diesel powered equipment is used in association with blasting, mucking and support activities. Where large deformations are expected after the completion of the final lining, some designers may still prefer a fully engineered solution, with mesh layers incorporated into a shotcrete lining and positive connection of the mesh to rockbolts and cables, over the use of simple fibre reinforced linings.

Uncertainties of this type in relation to shotcrete linings give rise to a tendency to overspecify the product in terms of the strength properties to be achieved at 3 days or 7 days. While it is generally true that high early strength is a desirable feature of shotcrete, nothing is gained by forcing the contractor to produce higher strengths than are needed. For example, a permanent lining sprayed onto a rock surface, which already had a reasonable level of support for temporary purposes, may not need to be accelerated at all. In this circumstance, the 7 and 28 day strengths generally accepted in structural concrete usage may be as good a criterion as the 3 and 7 day strengths which have come to be associated with shotcrete usage. It has already been stated that concern about the long-term effects of additives on strength has caused some designers to opt for an additive free shotcrete for final lining purposes.

13.9.4 Support installation sequences

Should rockbolts and cables be tensioned and at what stage of the excavation sequence should a shotcrete layer be applied ? These are questions which arise during discussions on the design of support for underground excavations in weak rock masses. They are dealt with here by means of a practical example based upon the support installation sequence for the power cavern of the Mingtan Pumped Storage Project in Taiwan. This project is described in detail by Cheng and Liu (1990) and further details can be found in a paper by Moy and Hoek (1989). Figure 13.18 gives a summary of the principal support installation stages for this 25 m span 46 m high cavern in fair to poor quality bedded sandstone.

During a preliminary contract an existing exploration/drainage gallery and two longitudinal working galleries were utilised to install grouted cables in the pattern illustrated in Figure 13.18a. These 50 tonne capacity cables were double corrosion protected and installed downwards from the exploration/drainage gallery, located 10 m above the crown of the arch, and upwards from the two working galleries. A light straightening load of 5 tonnes was applied to each cable before grouting and hence the cables were effectively untensioned but straight and fully grouted into the rock mass. Since these cables were installed before any excavation of the cavern had taken place, no significant displacements had occurred in the rock mass at the time of cable grouting.

Excavation of the cavern roof, illustrated in Figure 13.18b, induced significant displacements (Moy and Hoek (1989)) and these tensioned the grouted cables. Had the cables been tensioned before grouting, the additional tension induced by the displacements in the rock mass could have resulted in overstressing of the cables. The purpose behind the installation of these untensioned grouted cables was to reinforce the rock mass in much the same way as the placement of reinforcing bars in concrete acts to strengthen the concrete. The process was intended to improve the overall quality of the rock mass so that the main excavation contract could proceed with fewer rock stability problems than would have been the case had the pre-reinforcement not been in place.

The cavern roof was excavated by means of a central 6 m x 6 m heading which was subsequently slashed out to the full cavern width as illustrated in Figure 13.18b. Upon exposure of the final roof surface at each stage of this excavation process, a 50 mm layer of steel fibre reinforced micro-silica shotcrete was applied within 5 to 10 m of the face. The purpose of this shotcrete layer was to provide support for the small blocks and wedges which would otherwise have been free to fall from between the reinforcing cables. In addition, the shotcrete provided immediate sealing against moisture changes which could cause slaking in some of the siltstone rock units exposed on the surface.

A relatively thin shotcrete layer was used at this stage in order to allow for displacements which would be induced by adjacent excavation of the upper part of the cavern. Even if minor cracking of the shotcrete had been induced by these displacements, the presence of the steel fibre reinforcement provided a high post- crack deformation capacity for the shotcrete and hence maintained its support capacity.

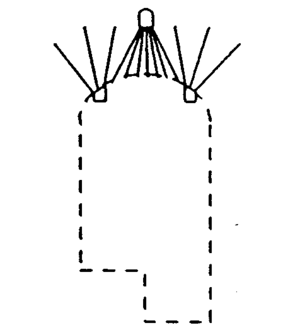
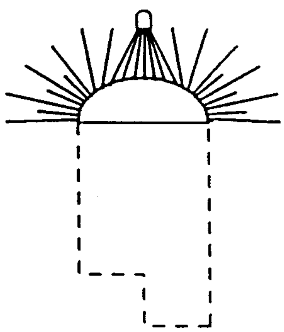
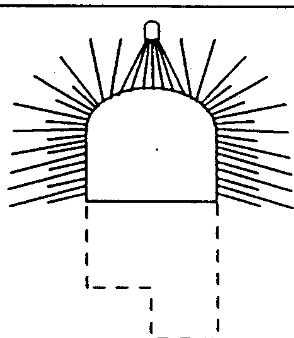
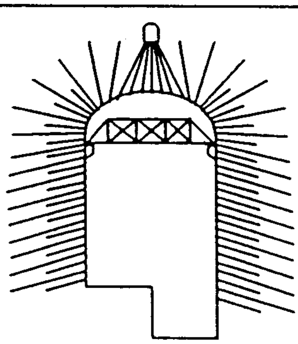
a 	Installation of double corrosion protected cables from exploration/drainage gallery located 10 m above center of roof arch and from two longitudinal working galleries. The 50 tonne capacity cables were installed on a 2 m x 2 m grid pattern and a small straightening load of 5 tonnes was applied before grouting. Dashed lines show cavern profile before excavation.
b 	Excavation of cavern roof by center heading and slashing with application of first 50 mm thick layer of steel fiber reinforced micro-silica shotcrete. Faceplates were added to projecting ends of cables and tensioned to 20% of ultimate capacity to ensure positive anchorage. Where required, 5 m long 25 mm diameter mechanically anchored, tensioned and grouted rockbolts installed at centers of 2 m x 2 m grid of cable reinforcement.
c 	Excavation of cavern by 2.5 m vertical benches. Double corrosion protected 112 tonne cables, inclined downwards at 15° to cross bedding planes, were installed on a 3 m x 3 m grid in the sidewalls. Before grouting, these were tensioned to 38 to 45% of yield strength, depending upon their location relative to the bench. Intermediate 6 m long 25 mm diameter tensioned and grouted rockbolts were installed between cables. Final shotcreting of the roof was carried out at an early stage of benching.
d 	Complete excavation of the cavern with 150 mm total thickness of steel fiber reinforced micro-silica shotcrete on the roof and upper sidewalls and 50 mm thickness on the lower sidewalls. Access to roof for inspection and minor remedial bolting maintained from temporary crane.

Figure 13.18: Support installation sequence for the power cavern of the Mingtan Pumped Storage Project in Taiwan.

Figure 13.19: Cables and shotcrete were used to support the roof of the power cavern in the Mingtan Pumped Storage Project in Taiwan.

Figure 14.30: Installation of cables in the sidewall of the power cavern in the Mingtan Pumped Storage Project in Taiwan.

As soon as the ends of the pre-placed reinforcing cables were exposed, faceplates were installed on them and a tension of 20% of the ultimate capacity of the cable was applied to ensure positive anchorage. Note that this tension acts over a very short length of each cable near its exposed end since the remainder of the cable is fully grouted into the rock mass. Experience in the mining industry has shown that the installation of faceplates on pre-placed untensioned grouted cables is very beneficial in providing support for the near surface blast damaged material which otherwise tends to fall away from the ends of the cable. In most areas in the Mingtan cavern, 5 m long 25 mm diameter mechanically anchored rockbolts were placed at the centre of each square in the 2 m x 2 m grid of cables. These bolts were tensioned to 70% of their yield load before grouting since they would not be subjected to significant displacements during the excavation of the lower part of the cavern.Excavation of the lower part of the cavern was carried out by means of 2.5 m high vertical benches. Sidewall support was provided by a 3 m x 3 m pattern of tensioned, grouted, double corrosion protected 75, 112 or 131 tonne cables, installed at a downward angle of 15° to ensure crossing of bedding planes which strike across the cavern axis (Moy and Hoek (1989)). These cables were tensioned at between 38 and 45% of their yield load, depending upon their level in the cavern sidewall and their location in relationship to the position of the bench floor at the time of installation. The tension was reduced for those cables which were installed close to the bench floor in the lower walls of the cavern. Mechanically anchored rockbolts, 6 m long and 25 mm in diameter, were installed between the cables as illustrated in Figure 13.18c. These bolts were tensioned to 70% of their yield load before grouting.

At an early stage of benching, when excavation had progressed to a stage beyond which further deformations induced in the roof were relatively small, an additional thickness of 100 mm of steel fibre reinforced micro-silica shotcrete was applied to the roof and upper sidewalls. The total thickness of 150 mm of shotcrete represents the final lining for the roof and upper sidewalls of this cavern. Full details of the shotcrete specifications and mix design have been given by Moy, Hsieh and Li (1990).

The lower cavern sidewalls were reinforced with cables in the same way as the upper sidewalls shown in Figure 13.18c. Only 50 mm of steel fibre reinforced micro-silica shotcrete was used on the lower sidewalls since most of these surfaces were subsequently covered by concrete as the turbine foundations were cast in place.

13.10 Excavation methods

The stability of a large underground excavation is very much dependent upon the integrity of the rock immediately surrounding it. In particular, the tendency for roof falls is directly related to the interlocking of the immediate roof strata. For weak rock masses with clay filled joints and localised soft or altered zones, the capacity of the rock mass to contribute to its own support temporarily during excavation can be totally destroyed by careless excavation or poor sequencing of excavation and support activities. Thus, the choice of excavation method assumes a degree of importance which has not always been catered for by specifications and construction procedures.

The most frequently adopted method for cavern excavation is drilling and blasting. Strictly speaking, blasting control is not a cavern design requirement but it exerts the biggest single influence on the outcome of the excavation process, and should be considered accordingly. In a weak rock mass, the sequencing of excavation and support will typically follow the top heading, slash and 2.5 to 5 m bench procedure described in the previous section.

The control of rockbolting (e.g. maintaining design spacing) and shotcrete operations (e.g. achieving uniform design thickness) is far more easily effected when good control is maintained over excavated profiles, so blasting specifications are normally written in terms of maximum permissible overbreak or the presence of ``half barrels" (charge hole drill marks) on the profile produced by the blast. Even in weak rocks, some half barrels can be expected with a well-balanced blast, especially if pre-splitting or smooth blasting is utilised, but care should be taken to ensure that the specifications do not demand results which are impossible to achieve in practice. After the initial heading, extensive free faces exist and provide a void into which the broken rock can move. In this circumstance, the blast energy absorbed into the rock mass should be minimal.

For a well-balanced blast, each successive delay should produce even breakage and leave the appropriate burden to be removed by the next delay. Whether a pre-split or a smooth blast is employed, the location and charging of the easier holes drilled closest to the perimeter holes is critical to the outcome of the blast, since overcharging or poor alignment of these holes will result in damage beyond the final perimeter which cannot be rectified once it has occurred. Hole alignment of the perimeter holes themselves is of obvious importance, and can be maintained by the use of parallel drill hole facilities on modern jumbos. Alternatively, drilling inspectors can help the operator maintain the required hole alignment during the early stages of drilling. For vertical drilling, templates made up from flat plates with steel tubes welded onto them to act as a guide for the drill, can be used when the hole is collared.

Where good drilling control and apparently well-balanced charges do not produce good blasting results, it is sometimes useful for the inspection teams to request a piecemeal blast. This requires each delay of holes to be fired individually, so that the profile created by each delay can be inspected to ensure that breakage has occurred in the way it should. If such a process confirms that the holes and charges are well-balanced but the production blasts still do not produce the desired results, then there may be a problem with the initiation system, e.g. excessive scatter on the delay detonators producing out of sequence firing.

The ultimate in damage control is machine excavation, and for cavern excavation, this usually implies the use of road headers. The possibility of using road headers for general cavern excavation should be considered wherever the intact rock strength is less than 60 MPa, and the viability of the method, as opposed to drilling and blasting, will be dependent on a comparison of the costs and required excavation rates rather than the ability of the road header to cut the rock. The absolute limiting rock strength for cutting with a road header has been put at 125 to 130 MPa, and this can only be achieved with great difficulty over short distances with pick destruction being the limiting factor (Pearce, 1988). The lack of disturbance to the rock and the possible reductions in support required are major advantages in the use of road headers.

The use of road headers becomes even more attractive when there is a need to control vibrations induced by blasting. This can be the case when cavern excavation is required adjacent to an existing underground installation, or when caverns are to be excavated relatively close to the surface. Langefors and Khilstrom (1973) and others have published blast damage criteria for building and surface structures. Almost all of these relate blast damage to peak particle velocity resulting from the dynamic stresses induced by the explosion. Where these generally applicable guidelines impose unreasonable restriction on the blasting requirements of a project, monitoring at the site allows site specific limits of the charge weight to be determined. These limits are defined by the charge weight in the cavern which will produce unsatisfactorily high particle velocities at the surface or the adjacent underground structures. An example of this type of monitoring is described in a paper on the Tai Koo cavern in Hong Kong (Sharp et al (1986)).

13.11 Cavern instrumentation

The installation of rock mass monitoring systems around a large cavern in a weak rock mass is considered essential in order to ensure that control is maintained over stability conditions during and immediately following excavation. The stress redistributions that accompany excavation in a weak rock mass can produce large deformations, which will in turn modify the loads carried by the rock reinforcement system and the stress carried by the shotcrete lining. Given the uncertainties of support design, the design engineer will require confirmation that his assumptions, on the level of deformation and load and stress changes that will occur, are not invalidated by the actual response to excavation.

Of the three effects listed above, the most reliable data usually come from displacement monitoring, since this can be conducted on a scale comparable with the size of the excavation and the volume of rock affected by stress redistribution. By comparison, measurements of stress change in the rock or the lining can only be conducted at isolated points which may not be representative of the average condition. Load monitoring in the rock reinforcement is possible for unbonded bolts and anchors, but the results are of questionable applicability to fully bonded reinforcement for which highly localised strains and load changes may occur where the bolt or cable crosses a specific joint.

Displacement monitoring may be relative or absolute. An example of the former is the installation of a multipoint extensometer in the sidewall of a cavern, with the deepest anchor inside the zone of rock where movement may be expected. Movements beyond the deepest anchor will not be registered by the extensometer. An example of absolute movement monitoring is the measurement of the horizontal convergence of the two sidewalls of the cavern by means of a tape extensometer stretched between the walls. However deep the movement, it will all be registered by the tape extensometer. Where relative displacement is monitored, the opportunity exists to extrapolate to the absolute displacement by calibrating a numerical model against the relative movements monitored, and using the model's predictions outside the monitored zone. Numerical models also allow the estimation of any movements occurring before the installation of the instruments, which are usually installed from inside the excavation. In some circumstances, it is possible to install extensometers, before the commencement of cavern excavation, from adjacent exploration or drainage galleries, and the advisability of doing this should always be assessed. In deciding the layout of extensometers, it is usually advantageous to be able to distinguish between movements inside and outside the reinforced zone, since the former will affect the loads carried by the reinforcement and the latter will not. During the later stages of excavation of a cavern in weak rock, large movements may continue at depth in the upper sidewall, but this may be of no concern to the designer if the reinforced zone has stabilised.

Stress changes in the rock can be calculated from monitored displacements by the assumption of a value for the rock mass modulus. The use of analyses of this kind during the early stages of excavation sometimes indicates the need for additional supports or modifications of the design requirements during a later stage of excavation. This will be backed up by load change data obtained from load cells fitted to isolated elements of the reinforcement system.

Large stress changes in shotcrete linings are usually fairly apparent from the occurrence of cracks in the lining. For this reason, visual observation maintains its status as an important data gathering method. It is also still the most effective way of assessing the groundwater conditions in the rock mass surrounding an opening. Where groundwater discharges into a cavern, piezometer installations are advisable to check that excessive pressures cannot build up in the roof or behind the sidewalls.

13.12 Summary and conclusions

The design of large powerhouse caverns in weak rock masses differs from that of caverns in stronger rocks in that failure of the rock mass surrounding the excavations and large deformations of the roof and walls will have to be accommodated in the design. This requires an understanding of the behaviour of weak rock masses and of the interaction of the support with these rock masses during excavation and subsequent operation of the caverns.

Estimating the strength and deformation characteristics of weak rock masses is an uncertain process and large variations in properties can be anticipated, particularly in bedded sedimentary rocks. This means that precise analysis of the stresses and deformations induced by the excavation of the caverns is not possible, and the designer has to rely on parametric studies in which the in situ stresses and material properties are varied over their maximum credible range in order to establish general behavioural trends. Examples of such parametric studies, using a two-dimensional elastic boundary element analysis, have been presented in this chapter. More refined studies, using non-linear progressive failure analyses, are only justified when sufficient data have been gathered from the monitoring of actual excavation behaviour to provide realistic input data for such analyses. An example of this more refined type of analysis is presented in Cheng and Liu (1990).

Issues such as the location of the caverns relative to the toes of slopes and the determination of the lengths of steel linings in pressure tunnels, while not central to the question of cavern design, have important practical and financial implications and have been considered briefly in this chapter.

The principal issues which have been addressed are those of the failure and deformations induced in the rock mass surrounding large caverns and how these are dealt with in the choice of the excavation shape and the type of reinforcement used. Concrete arches, traditionally used to provide support for the rock mass above large powerhouse caverns, can suffer from excessive bending as a result of the large deformations which occur in these weak rocks. Consequently, the author recommends that concrete arches should not be used or that, if they are used, very careful attention be given to matching the deformation characteristics of the arch to the displacements which occur in the rock mass. A preferred means of support involves the installation of grouted cables and rockbolts in the rock mass and the application of a surface layer of shotcrete to stabilise the near surface blast-damaged rock. This system is very flexible, as compared with the concrete arch, and can move with the rock mass to accommodate the large displacements associated with cavern excavation. Corrosion protection of the cables is essential since these provide the primary permanent support for the rock mass and must have a working life in excess of that of the cavern itself.

The sequence of installation of cables, rockbolts and shotcrete is an important issue which has been illustrated by means of a practical example. The questions of whether rockbolts and cables should be tensioned before grouting and when different shotcrete layers should be applied are all related to the development of the deformation pattern in the rock surrounding the excavation. Consequently, the sequence of support installation must be carefully matched to the sequence of excavation in order to provide adequate support without the risk of overstressing the support elements.

All of the care which has been taken in estimating the in situ stresses, the rock mass strength and deformation characteristics and in carrying out the support design can be wasted if excessive damage is inflicted on the rock by careless blasting. Techniques for controlling this blast damage are available and have proved to be very effective when Owners and Engineers work with the Contractor to ensure that these techniques are used during critical stages of a project.