Design of large underground caverns for hydroelectric projects with particular reference to structurally controlled failure mechanisms

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Abstract

Two distinct types of failures occur in the roof and walls of excavations in rock. In weak or very heavily jointed rock or in massive hard rock subjected to very high stress, failure of the rock mass surrounding the excavation is the dominant failure mode. In hard rock excavations at shallow depth, gravity controlled falling or sliding of blocks or wedges defined by intersecting structural discontinuities is the most common type of failure. In the former case, support by means of pattern bolting with shotcrete and mesh is the most common means of stabilizing the excavation with steel sets or concrete lining being used in extreme cases. In the case of structurally controlled failures, rock bolts or cables designed to support the weight of individual blocks or wedges are generally the most effective and economical form of support.

The differences between these two types of rock mass failure is examined and some of the practical aspects of underground excavation design involving these types of failure are discussed.

Introduction

Geotechnical engineers involved in the design of support for underground excavations are faced with the perplexing problem of deciding upon a suitable design approach. Developments in underground support design have included the use of rock mass classification systems (Barton, Lien and Lunde, 1974; Bieniawski, 1974), analytical-observational approaches such as the New Austrian Tunnelling Method (Rabcewicz, 1964), support-interaction concepts (Ladanyi, 1974; Hoek & Brown, 1980), and designs based upon structurally controlled failure mechanisms (Hoek, 1977; Croney et al., 1978). Combining these designs approaches with well documented case histories (Cording et al., 1971) means that the design engineer has access to a wealth of experience. However, the design engineer is still faced with the critical problem of matching the most appropriate design approach to the rock conditions and of recommending the most suitable design-contractual arrangement for the job.

This paper deals primarily with a specific type of underground support design problem, namely the identification of and support design for structurally controlled failure in hard rock excavations. This particular design approach is discussed in terms of the set of conditions to which it applies and those conditions where its use is inappropriate.

Support design philosophy in terms of rock conditions

The simplest classification of rock is to divide it into two categories of weak and strong. Weak rock is defined as that where the strength of the rock mass is less than or equal to the induced stresses around the underground opening. Strong rock would have a rock mass strength of two or three times the maximum stress around the opening. This simple classification makes the choice of support design philosophy relatively easy although it must be realized that there are intermediate cases in which the choice will be less obvious.

Rock masses may be weak because the strength of the component materials is low or because the rock mass is very highly jointed. In either case, the weak rock mass surrounding an underground opening requires uniform support such as that provided by a regular pattern of rock bolts accompanied by mesh-reinforced shotcrete. Support pressures, bolt spacings and shotcrete thicknesses are usually based upon precedent design practice with an increasing tendency to use rock mass classification systems to provide a rational basis for the choice of the most appropriate values. Analysis of the stresses induced around the opening can be used to identify critical zones in which additional support of pre-placed support is required to improve stability. The response of the rock mass to the excavation sequence and the acceptance of load by the support system is particularly important and the rock -support interaction concept (Ladanyi, 1974; Hoek & Brown, 1980) can be used as a guide in the overall support design. In weak rock, underground excavation stability is generally controlled by failure of the rock mass and structurally controlled failures of significant size are rare.

An example of underground excavation design in weak rock is that for the Drakensberg Pumped Storage Scheme in South Africa (Bowcock et al., 1977). Weak horizontally bedded mudstones, siltstones and sandstones are supported by pattern bolting and mesh reinforced shotcrete. In this case, structurally controlled failures were not significant and were not considered in the design.

At the opposite end of the spectrum are excavations in strong rock masses. In these cases, not only is the intact rock strong but the spacing of joints is relatively large. Under these circumstances, the stability of the rock mass surrounding the underground openings is generally controlled by structural failure mechanisms. The Dinorwic Pumped Storage Scheme in Wales and the Rio Grande Pumped Storage Project in Argentina are typical examples of large excavations (20 m and 25 m cavern spans respectively) excavated in strong rock (Douglas et al., 1977).

In the case of excavations in strong rock, blocks or wedges defined by intersecting structural features can either fall from the roof or slide from the roof or walls. The excavation designer has the choice of supporting individual blocks or wedges with specifically designed reinforcement (referred to here as structural bolting), a combination of structural bolting and pattern bolting or, in some cases, pattern bolting only.

The choice of a suitable design philosophy in weak and strong rocks tends to be relatively simple as compared to that required for intermediate strength rocks in which all of the failure mechanisms discussed earlier in this paper can occur. In such cases, a combination of several different support methods may be required and the choice of the optimum combination involves a great deal more judgement than the extreme cases of weak or strong rock. Having said this, the design engineer should still make a concerted effort to identify the role of support in different locations around the excavation boundary. Failure to do this makes it very difficult to judge the effectiveness of the design and to intelligently interpret the results from any monitoring system installed in the excavation.

Support design philosophy in terms of contractual arrangements and engineering supervision during excavation

The design of pattern bolting to support excavations in weak rock can be done prior to the commencement of excavation and changes in this pattern as excavation proceeds are likely to be relatively minor. Under these circumstances, traditional contractual arrangements developed for shallow tunneling can be applied without too much difficulty. The same is not necessarily true in the case of excavations in strong rock in which it may not be possible to design an effective support system in advance of the excavation and where traditional contractual arrangements may not be appropriate. The remainder of this paper deals with the design of support for excavations in strong rock and explores some of the options which are available to the design engineer.

The first major decision to be made prior to commencing the excavation is the level of geotechnical investigation required to provide preliminary design data. The preliminary investigations typically involve regional geology studies, mapping of surface outcrops and geological data collection from core drilling from surface. In the case of strong rock sites, these preliminary investigations should be aimed at obtaining information on major structural features such as faults which may intersect the excavations. The orientation, width and shear strength of these major features will have a significant influence on the stability of the openings intersected by them.

Samples form diamond core drilling can be utilized to determine the intact strength of the rock, the shear strength of joints on a small scale and the dips and dip directions of significant joint sets (this latter set of measurements may require the use of special core orientation techniques). The orientation of the major structural features together with the dips and dip directions of the joints can be used as a basis for optimizing the orientation of the excavations. Because of the constraints imposed by hydraulic and other non- geotechnical considerations, it is unlikely that the major excavations can be completely re-oriented to achieve maximum structural stability. However, even relatively minor re-orientation can sometimes result in significant improvements in stability and hence savings in support costs. With the exception of major faults, it is very difficult to trace specific structural features from one borehole to the next, almost irrespective of how close the holes are to one another. This means that a complete three-dimensional structural model cannot be established prior to the commencement of excavation. Prominent joint orientations can be defined but specific blocks cannot be identified. Hence, at least for excavations in strong rock, a surface drilling program quickly reaches a state of diminishing returns in terms of useful information recovered for the money spent. This money can often be directed more profitably towards increasing the engineering input into the excavation phase and finalizing the detailed design as the excavation proceeds. Expressing this in simpler terms, the exploration program prior to excavation should be directed towards defining the general rock conditions and specific major fault-related problems. These studies are particularly important in terms of estimating overall project costs but, having achieved this, the surface investigation program may well be suspended in favour of assessing the detailed support requirements on a ‘design-as-you-go’ basis.

Figure 1 shows a typical excavation sequence for the main cavern of an underground power project. The rock conditions exposed in the central top heading provide the design engineer with much of the information required to carry out a preliminary design of the detailed support requirements for the remainder of the cavern. It is important that as much flexibility as possible be retained during the subsequent excavations in case the projections made on the basis of the observations in the pilot heading prove to be inaccurate and the support design has to be changed to accommodate specific local conditions.

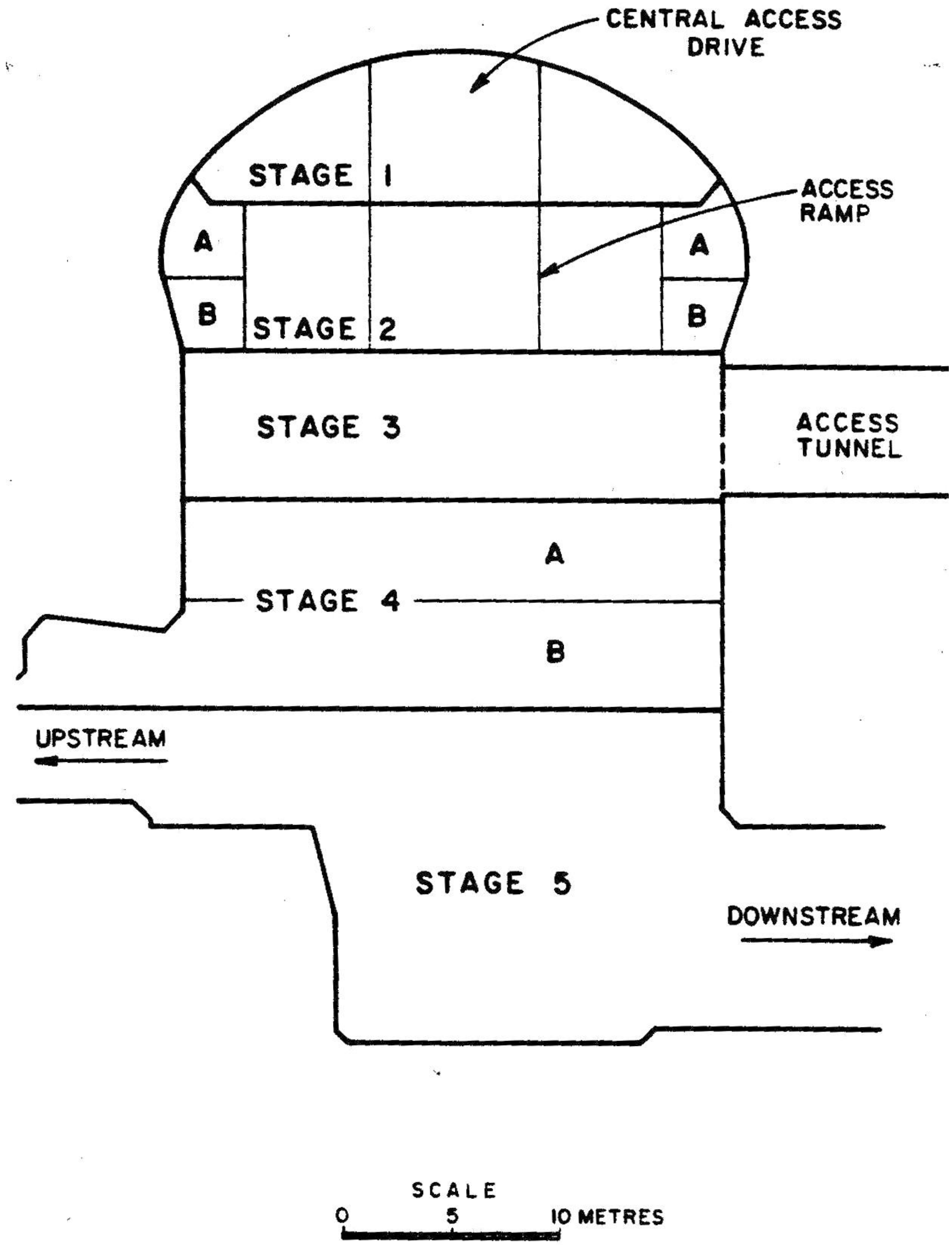

Figure 1. Excavation stages for cavern

In the ‘design-as-you-go’ approach, the pilot heading becomes an important component in the design process. Significant structural features should be mapped to identify specific blocks or wedges which are potentially unstable and which require support as they are progressively exposed during the subsequent excavation stages.

The techniques involved in mapping the roof and walls of the pilot heading and of projecting the mapped structural features onto the as yet unexcavated boundaries in order to determine the areas of potential instability, have been discussed by Croney et al. (1978). These techniques were used in the design of support for the main excavations of the Dinorwic Pumped Storage Scheme in Wales.

The ‘design-as-you-go’ approach was also found to provide a very efficient and economical basis for support design in the Rio Grande Pumped Storage Scheme in Argentina. The support design for the roof and walls of the main cavern was based solely on the need to support specific blocks or wedges. Pattern bolting was only used in some local areas of closely spaced jointing.

It should be noted that in spite of its many advantages, the ‘design-as-you-go’ approach may not be suitable for all excavations in strong rock. It requires a competent team of design engineers to be on site for most of the excavation phase and for them to work intimately with the contractor. In most cases, the mapping and bolt installation procedures need to be integrated directly into the excavation cycle. In some instances, temporary support requirements need to be estimated by the design engineer at the face and be installed immediately after the mucking cycle. These design estimates can be checked later and additional support can be installed if required. As discussed later, the analyses are not particularly complicated and require a knowledge of the orientation of the discontinuities defining the block, the unit weight of the rock, and the shear strength of the discontinuities. The analysis can readily be performed with the aid programmable calculators.

The ‘design-as-you-go’ approach also requires a very flexible contractual agreement between the contractor and the owner. Bolt lengths and diameters obviously need to be standardized to allow for efficient manufacture, but the final decision on bolt location and size needs to be made by the on-site design engineer. Similarly, the heading size needs to conform to the size of the blast hole drilling equipment but there needs to be enough flexibility for the design engineer to recommend decreasing the pilot heading size when the rock conditions are poor and increasing it again when the rock improves.

If a ‘design-as-you-go’ approach is considered inappropriate, possible for the reasons discussed earlier, the alternative is to design a regular bolting pattern which will provide adequate support for potentially unstable blocks, irrespective of whether or not these blocks are actually exposed in the roof or walls. A safe approach, although one that may be very conservative, is to determine, from the surface drilling program, the orientation of the discontinuities in the rock mass and assume that these are continuous and can occur at the worst possible location forming the maximum sized block for the opening. This is often referred to as the ubiquitous joint approach and a suitable bolt pattern can be designed to ensure the support of this maximum sized block. It is very unlikely that this maximum sized block will in fact be present, although geometrically similar smaller blocks may occur.

The most significant advantage of the pattern bolt approach is that the design can be finalized prior to commencing the excavation and the only supervision required is to maintain quality control of the installation of the support. As noted earlier, the disadvantage is that depending on the exact size and location of blocks, the design is likely to be very conservative. A combination of pattern bolting and structural bolting may in some cases prove to be a useful compromise, although the stability of blocks should be assessed soon after excavation to determine if the pattern bolting provides sufficient support or if additional structural bolting is required.

The structural support that has been described in the preceding discussions refers to major structural blocks. In addition, local support of the rock at the boundary of the excavation, which is often quite loose from blast damage or from small blocks having fallen out, is required. This can be achieved in most cases by pattern bolting between the structural bolts with short, low strength-capacity bolts, combined with mesh reinforced shotcrete.

Identification of structural failure mechanisms

The onus falls on the design engineer to identify potential failure mechanisms of blocks falling and sliding from the roof and walls of the excavation. This can sometimes be done by visual observation at the excavation face but should subsequently be checked after a detailed map of the discontinuities has been prepared, to ensure that none have been missed. This map should show the dip and dip direction of all faults and joints and the surveyed location of their traces on the excavation boundary. Figure 2 shows a typical structural map of part of the curve roof of a large excavation and identifies potential unstable blocks.

The minimum number of structural planes required to completely define a block is three, with the fourth plane of the block being the boundary of the excavation. Failures can occur involving one or two structural features if it is assumed that release surfaces are formed by failure of the rock or by the presence of minor structural weaknesses. Failure mechanisms with one or two structural features, and assumed release planes, are shown in Figure 3. Release planes form more readily if the main structural features are continuous over a considerable length, and failures of this type are often associated with faults, particularly where faults intersect the excavation at a relatively shallow angle.

Blocks defined by three structural planes can either fall or slide from the roof or slide from the walls. Sliding can occur on one plane with the other two planes acting as the release planes, or slide on the intersection of two planes with the third plane acting as the release plane. As discussed by Croney et al. (1978), blocks can also be formed by more than three structural planes, although this is not very common and will not be discussed further in this paper.

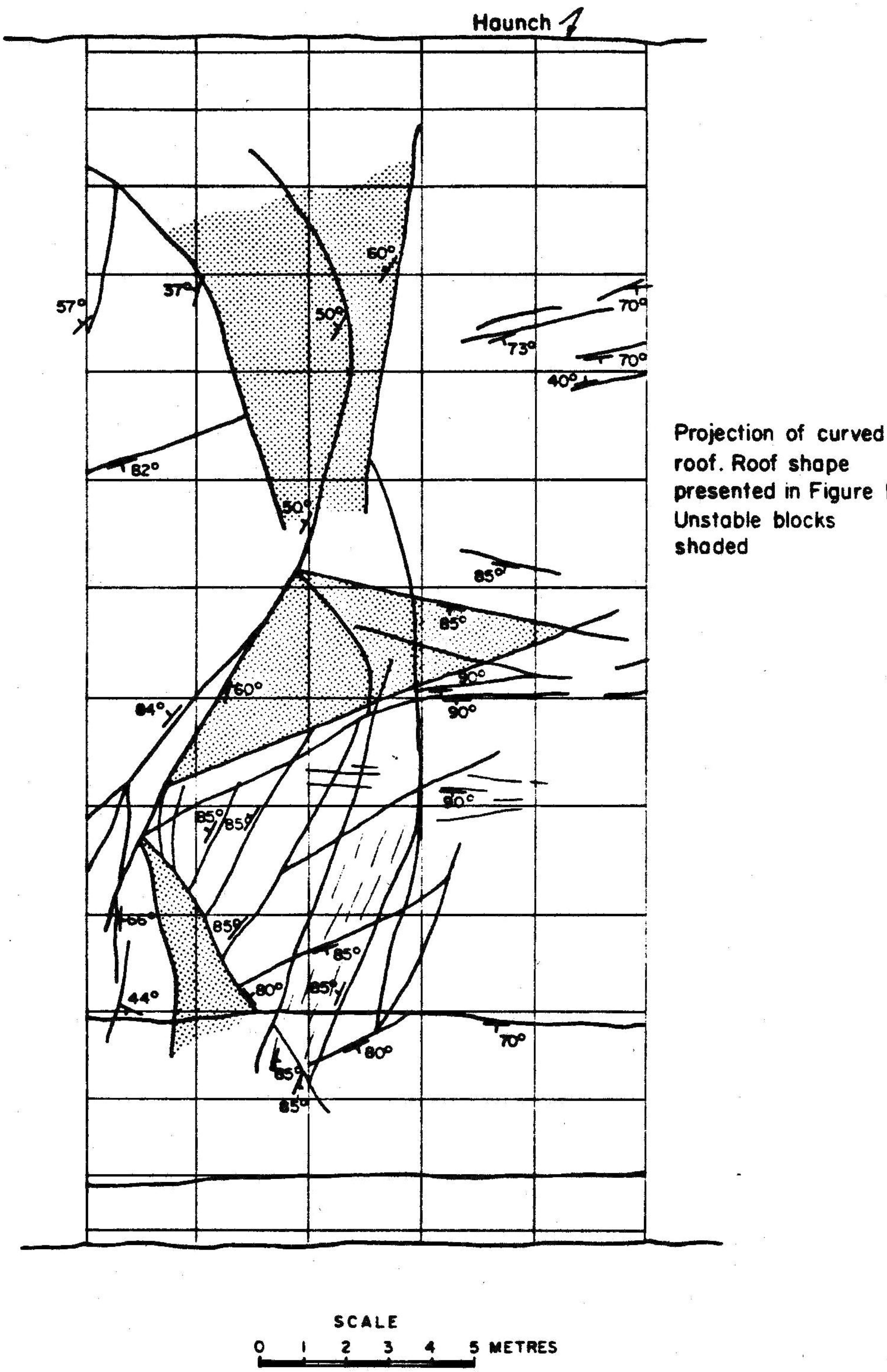

Figure 2. Structural plan of roof of excavation

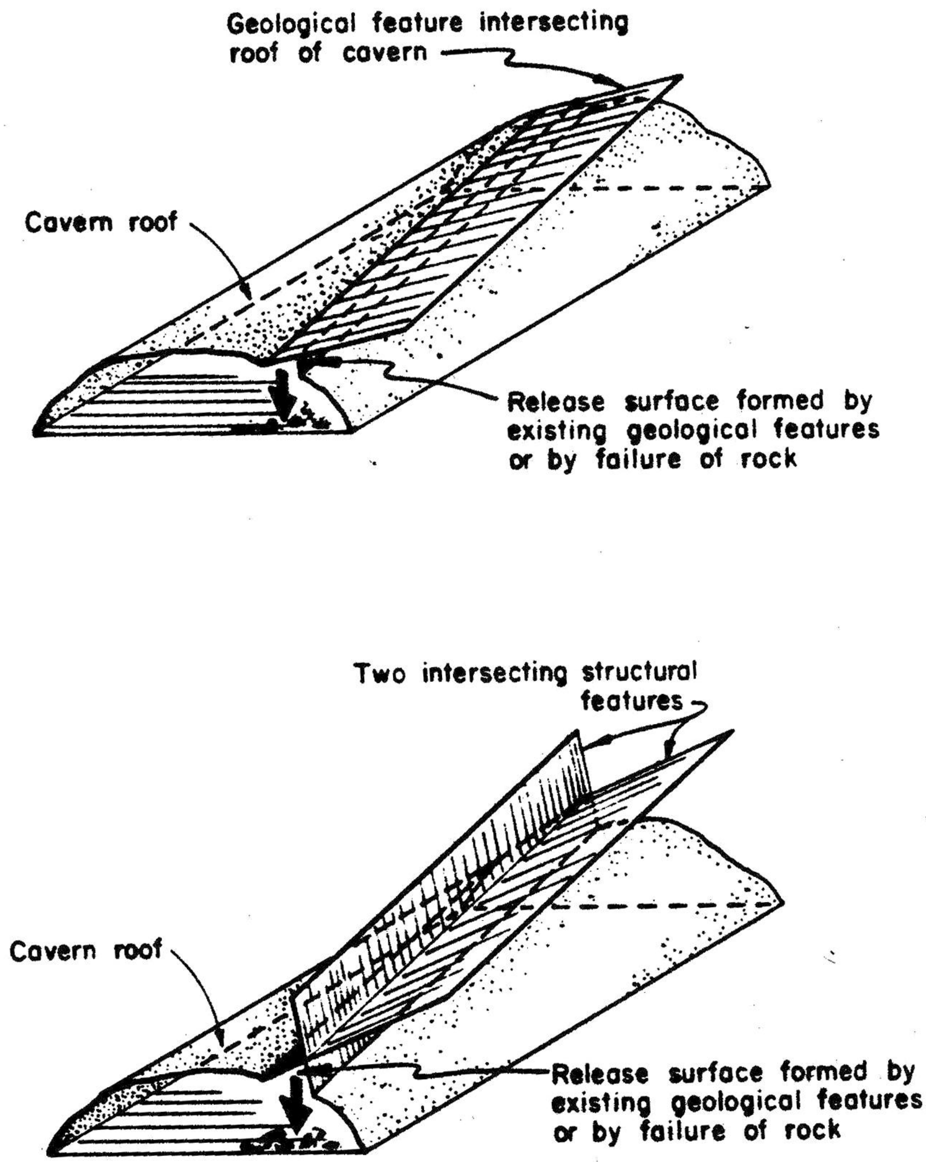

Figure 3. Failure mechanisms involving single and multiple structural features

Potential failure mechanisms can be identified relatively simply by preparing a stereographic plot of the intersection planes. Figure 4 shows stereographic plots for potential failure mechanisms in the horizontal roof of an excavation. The first three of these assume that release planes from by failure of the rock, possibly by a bending type collapse. In the other three mechanisms, the block is completely defined by existing structural planes.

For the case where a block formed by three structural features falls from the roof, a vertical line drawn through the apex of the wedge must fall within the base of the wedge. In the stereographic plot, the vertical line through the apex is represented by the center point of the net and for the block to fall, the joint planes form a closed figure which surrounds the center of the net. In all other cases of a block formed by the intersection of three planes, failure must occur by sliding. It should be noted that

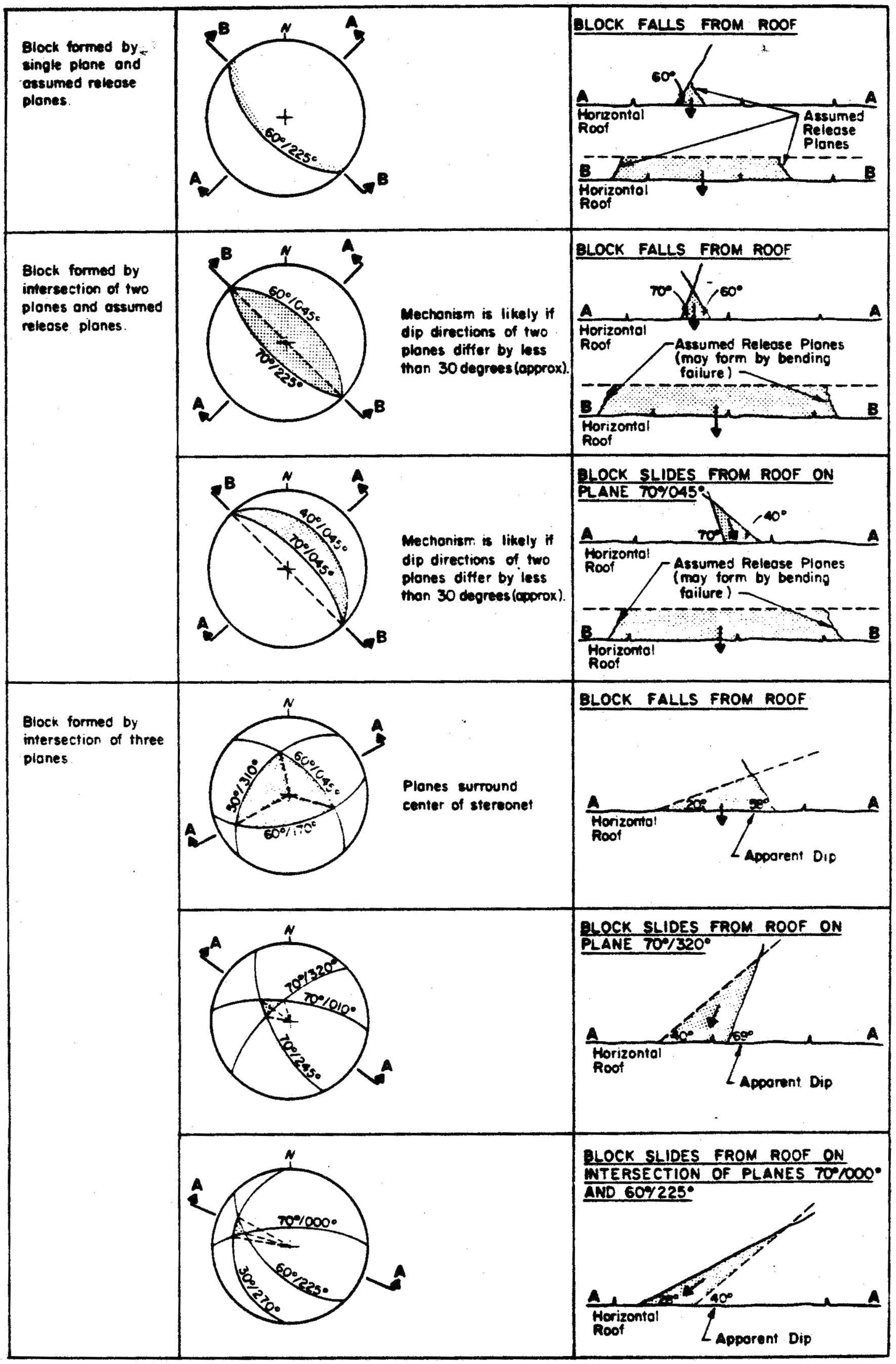

Figure 4. Potential failure mechanisms in horizontal roof

when considering the stability of specific blocks, it is important to look at the detailed geometry of the block and not just the stereographic plot. For example, for the two blocks shown in Figure 5 exposed in the horizontal roof of an excavation A is self-stabilizing while block B can fall under its own weight. The stereographic plot is identical for both cases.

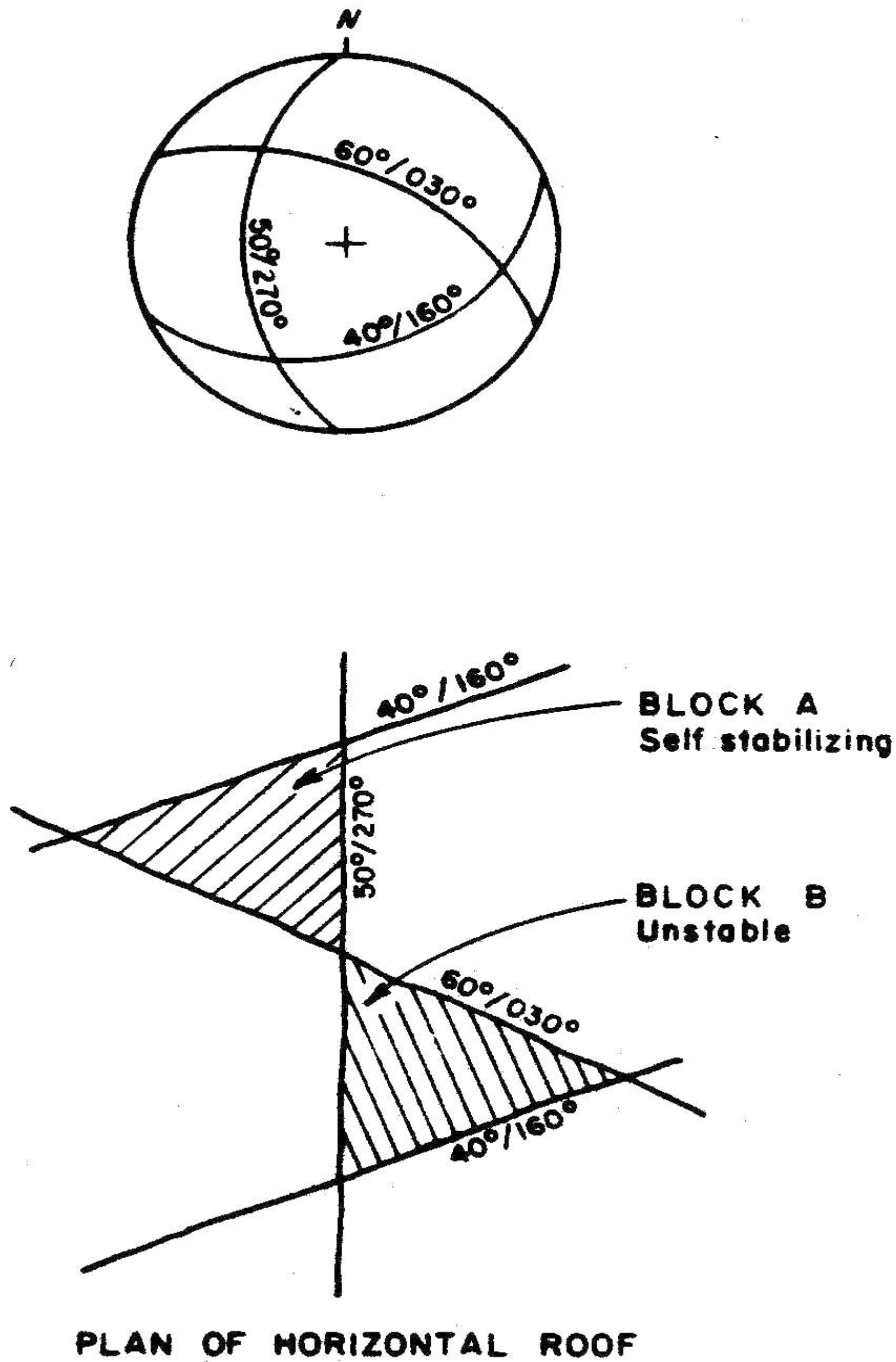

Figure 5. Self stabilizing and unstable blocks

Figure 6 shows possible failure mechanisms is the wall of an excavation, the wall being oriented at 30 degrees east of north. The wall presented in the figure is the eastern wall, and potential failure mechanisms in the western wall are mirror images of those in the east wall. The mechanisms involve either sliding on a single plane or along the intersection of two planes. The first two mechanisms assume that release planes form by failure of the rock, possibly by a bending type collapse. In the other four mechanisms, the block is completely defined by existing structural planes. Assessing potential failure mechanisms for an inclined wall or roof follows the same general principles outlined above.

Roof and sidewall failure analysis

From the stereographic plots shown in Figures 4 and 6, it is possible to determine if there is a potential for block failure. In the case of a block falling from the roof, no further stability analysis is required except to calculate the weight of the block and determine the required bolt capacity to support it. A factor of safety of between 1.5 and 2.0 is recommended, depending on the quality of the bolt installation. The weight

of the block can be calculated graphically or analytically by methods described in detail by Hoek and Brown (1980).

For blocks which slide from the roof or walls of the excavation, the stereographic plot is used to determine if there is a potential failure mechanism but the actual stability depends primarily on the orientation of the direction of sliding and the shear strength of the sliding surfaces. Sliding is either on a single plane or along the line of intersection of two planes.

If it is assumed that the sliding surface has a linear frictional strength with zero cohesion, sliding can only occur if the plane or the line of intersection along which sliding is to occur is steeper than the angle of friction.

It should be remembered in assessing the shear strength of the surfaces on which sliding takes place that the normal stresses on these planes, as a result of the weight of a typical block, are quite small. This means that a small degree of roughness of the planes may result in a high equivalent frictional strength. If the frictional strength is assessed fro direct shear tests, the normal stress at which the tests are performed should be correspondingly low in order to interpret the results correctly.

For the case of a sliding block, once the trace length of one of the discontinuities or alternatively the maximum span or height of the exposed face (the ubiquitous case) is known, the detailed block geometry can be determined graphically or analytically by methods described in detail by Hoek and Brown (1980). The block geometry allows the weight of the block and the orientation of the sliding surface to be determined, from which the factor of safety against sliding can be calculated. For sliding on a single plane with two other intersecting planes acting as release planes, the factor of safety is given by:

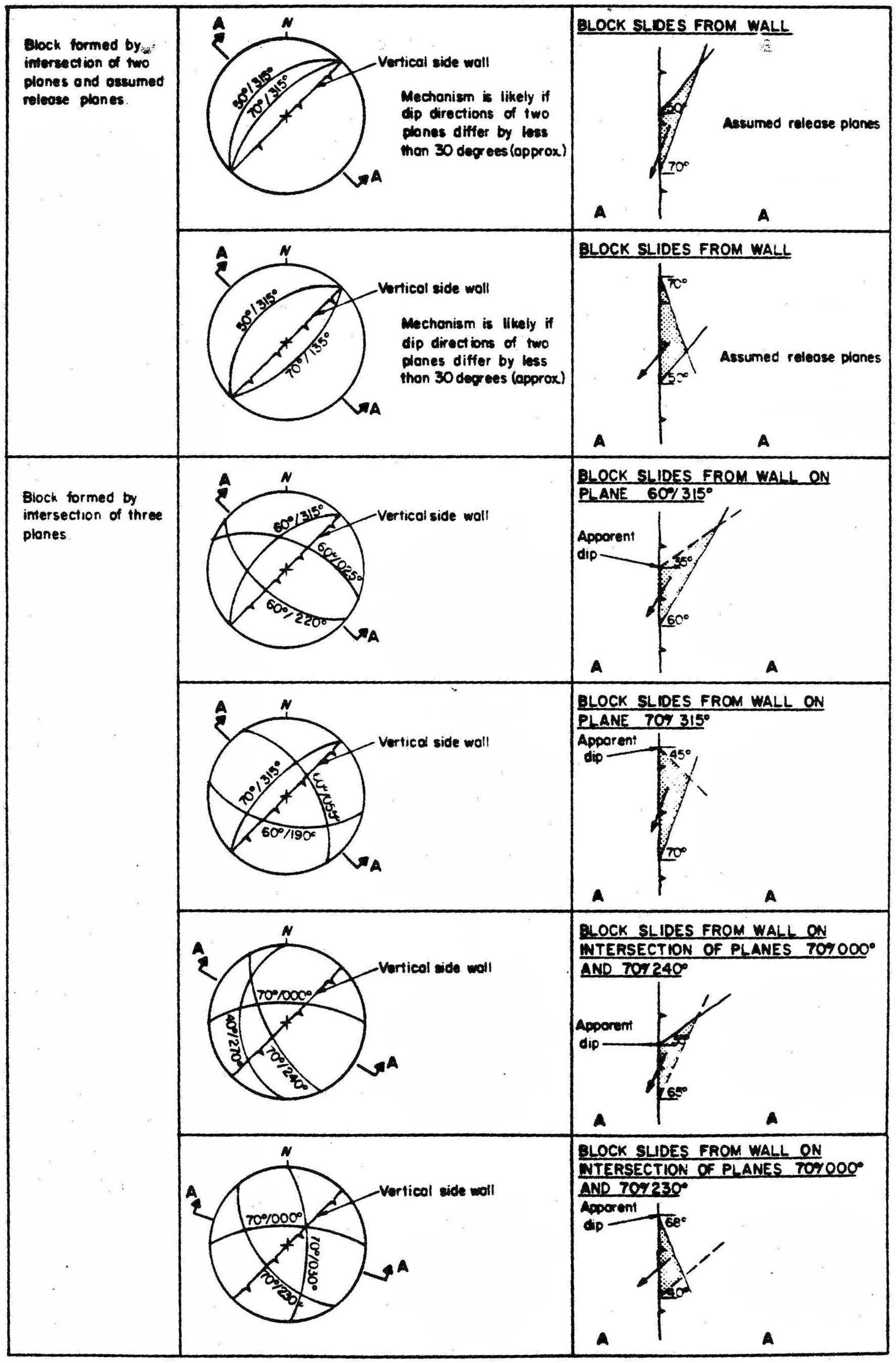

Figure 6. Potential failure mechanisms in vertical wall

$$F = \frac{cA + (W\ Cos\psi + Cos\theta)Tan\theta}{W\ Sin\psi - T\ Sin\theta} \tag{1}$$

where:

W is the weight of the wedge or block,
T is the load in the bolts or cables,
A is the dip of the sliding surface,
c is the cohesive strength of the sliding surface

Hence, the total bolt or cable load required is:

$$T = \frac{W\left(F \cdot Sin\psi - Cos\psi Tan\phi\right) - cA}{Cos\theta Tan\phi + F \cdot Sin\theta} \tag{2}$$

As in the case of blocks falling from the roof, a factor of safety of between 1.5 and 2.0 is recommended, depending on the quality of the bolt installation.

When the geometry of the wedge or block is such that sliding would occur along the line of intersection of two planes, the analysis presented above can be used to give a first approximation of the support load required. The plunge of the line of intersection should be used in place of the dip [EQUATION] of the plane in the above equations. This solution ignores the wedging action between the two planes and the answer obtained would be conservative, i.e. a lower factor of safety would be given than that which would be obtained from a full wedge analysis. This would result in a higher bolt load being calculated than would actually be required. In many practical applications, particularly for small blocks, the quality of the input data and the economic importance of the saving in rock bolts would not justify a more refined analysis. In the case of very large underground caverns, the sizes of wedges or blocks can be considerable and hence a more precise analysis may be justified.

The analysis presented in equations 1 and 2 take into account the normal forces on the sliding plane as a result of the weight of the block, but not the forces on the block due to the stresses around the excavation. It is difficult to quantify the stresses close to an excavation boundary because the rock mass tends to loosen and open up. The stiffness of the rock mass, the orientation of the discontinuities forming the block relative to the stress direction, and local damage of the rock as a result of blasting, strongly influence the stress distributions. Hence it is difficult to rely on the potential stabilizing effect of these boundary stresses. Other than possibly reducing the factor of safety slightly, it is not recommended that these stresses be included in the analysis of the stability of a block. If a block slides on a rough surface and shear dilation occurs, normal stresses can be generated on the release planes, and the stability of the block is improved. Once again, it is difficult to rely on this effect, and the potential increase in stability is generally ignored in any analysis.

Conclusions

In summary, it is important for the design engineer to understand the role the support that is installed to ensure the stability of underground excavations. In weak rock masses, rock bolts are installed to stitch together the rock where the stresses have exceeded the strength so that this rock is held in place. This failed rock has a reduced, but nevertheless finite strength, and this can generate confining stresses which help stabilize the remainder of the rock mass around the excavation. In strong rock masses, stability is often controlled by intersecting discontinuities which form blocks which can either fall or slide from the roof of an excavation or slide from the walls. Rock bolts can be used to support these potential failure mechanisms.

In cases where structural instabilities dominate the support requirements for large underground excavations, it is often difficult to design the support in advance of the excavation. The location, orientation and continuity of features in the roof and wall of an excavation can rarely be determined from surface drilling. An alternative approach is to carry out a modest exploration program in advance of the excavation and assess the detailed support requirements as the excavation proceeds. This 'design-as -you-go' philosophy requires a very flexible contractual agreement between the contractor and the owner and a competent support design team to assess the stability and support requirements after each blast. This design team is responsible for identifying potentially unstable blocks and providing adequate support to ensure the stability of the excavation.

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