A Reliability-Based Approach for the Assessment of Stability and Support Requirements in Jointed Rock Excavations

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SUMMARY A reliability-based approach has been adopted for the assessment of stability and support requirements for three case study jointed rock excavations. The assessment of excavation stability has been based on the distributions of individual volumes of potentially unstable blocks surrounding each excavation. The assessment of support requirements has been based on the distributions of block depths and their occurrence per excavation advance. The dimensions and occurrence of potentially unstable blocks has been quantified by examining the interaction of excavation geometry with the rock mass structural patterns on 2D joint trace maps for selected section views through each rock mass. Joint trace maps have been generated using joint statistics and a simple rock mass structural model. By repetitively superimposing the respective excavation geometries on the corresponding joint trace maps it was possible to identify potentially unstable blocks according to simple geometric criteria and quantify their dimensions to construct distributions of block geometry. The probabilities of encountering individual block volumes per excavation advance have been represented on cumulative frequency distributions to provide reliability-based predictions of excavation instability. Confidence levels have been represented on cumulative frequency distributions for block depths to provide reliability-based predictions of support requirements in terms of required lengths of rock bolts to support potentially unstable blocks. Predicted rock bolt spacings have been calculated based on the number of potentially unstable blocks identified per excavation advance. A reasonably good correlation has been found to exist between the reliability-based predictions and actual and/or recommended support requirements for the three case study rock excavations that have been investigated. This approach is ideally suited for the feasibility and preliminary design stages of a project to provide estimates of support requirements for costing purposes.

1. INTRODUCTION

The assessment of stability and support requirements in jointed rock excavations is characterised with an inherent geotechnical risk due to the common variability associated with rock mass parameters and the limited information that can only be gathered during site investigations from which design decisions must be based. To deal with this inherent geotechnical risk engineers are more frequently adopting an approach that utilises stochastic methods within a probabilistic framework to recognise the uncertainty and variability associated with rock mass parameters.

This alternative and somewhat new approach has become to be known as the "reliability-based" approach and is becoming increasingly accepted in geotechnical engineering applications (Harr, 1987). By adopting a reliability-based approach the geotechnical risk can be quantified and design decisions can be made according to specified levels of certainty. The degree of conservatism associated with designs should also reduce by adopting a reliability-based approach.

Reliability-based approaches for design purposes in rock engineering have been commonly adopted for surface excavations but to a much lesser extent for underground excavations. Chan and Goodman (1983,1987) have expanded on Block Theory (Goodman and Shi, 1985) by considering joint statistics for joint spacing and continuity in order to predict support requirements. More recently, Esterhuizen and Fourie (1988) have considered the variability of joint orientation to assess the potential for wedge failure in a vertical shaft. Stacey (1989) has investigated the rockfall potential in mining stopes by considering the variability of joint spacing.

This paper presents a reliability-based approach for the assessment of stability and support requirements in jointed rock excavations. The importance of being able to predict the expected stability and support requirements prior to construction of a rock excavation is reflected upon consideration of the fundamental cost and safety aspects of a rock engineering project. Excavation stability is assessed in terms of potentially unstable block volumes and support requirements are assessed in terms of the numbers and depths of potentially unstable blocks. The distributions of both block depths and volumes are considered in the reliability-based approach (Figure 1).

The contents of this paper include a discussion on rock mass structural models in Section 3 with a description of the model used for examined case studies. Sections 4 and 5 provide brief explanations of both the statistical and probabilistic assessments of stability and support requirements. Section 6 presents limited results from assessments of three case study excavations and a discussion of these results is included in Section 7. Finally, concluding remarks on the overall limitations, applicability and further development potential of the reliability-based approach are presented.

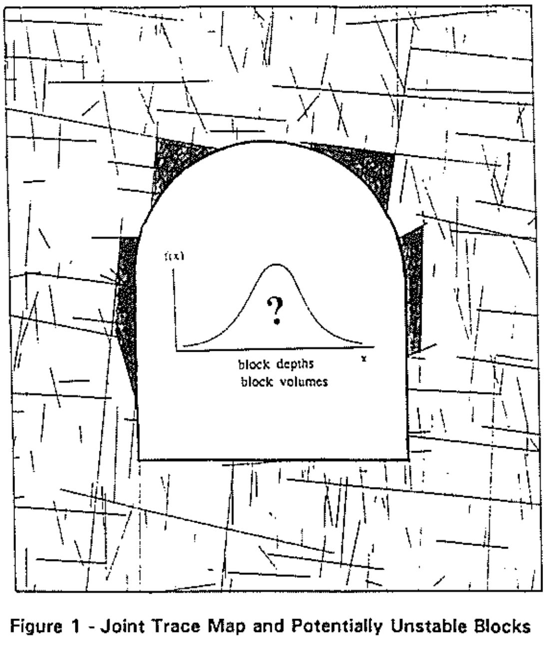

Figure 1 - Joint Trace Map and Potentially Unstable Blocks

2. ROCK MASS STRUCTURAL MODELS

2.1 Research to Date

The development of rock mass structural models was initiated for rock mechanics applications (Baecher et al.1977, Einstein and Baecher, 1983,) but more recent developments have concentrated for applications in the study of fracture flow (Dershowitz, 1984, Rouleau and Gale, 1987). The formulation of rock mass structural models has for the most part been based on the statistical characterisation of joint set parameters. Spatial interdependence between joint set parameters has been recognised through geostatistical models and the integration of these relationships has been identified as a further area for development.

Very few rock mass structural models developed to date have been utilized for assessing rock excavation stability and support requirements. Chan and Goodman (1983) developed a stochastic joint trace simulation model based on the principles of the Poisson Disk Model (Baecher et al.1977). The aim of the model was to assess support requirements in jointed rock excavations due to the presence of key blocks identified using Key Block Theory. Joint statistics for joint radii were considered however joint orientation was treated deterministically. Further refinement of this model by Chan (1984) defined measures of support requirements to characterise the number and dimension of key blocks considering the frictional properties of joints and the response of support systems.

Other rock mass structural models have been presented by McCullagh and Lang (1984) as well as Shi et al.(1985). As part of their continued research on the subject, Chan and Goodman (1987) developed a Monte Carlo procedure to be used in conjunction with the Poisson Disk Model for predicting the number and dimensions of key blocks. An investigation of the effects of excavation width, mean joint density and mean trace length on the measures of support requirements for the roof of a tunnel in a rock mass with three joint sets provided a theoretical verification of their procedure upon consideration of the results of the studies presented by McCullagh and Lang (1984).

2.2 A Simple Rock Mass Structural Model

2.2.1 Model Parameters

A simple rock mass structural model has been developed by Steffen, Robertson and Kirsten Consulting Engineers of Johannesburg, South Africa(Haines, 1983,1984). The model considers standard statistical descriptors of mean, maximum, minimum, standard deviation and a distributional form for joint dip, dip direction, dip continuity, strike continuity and spacing. The available distributional forms in the model for describing the joint set parameters are negative exponential, lognormal and normal. In general, the model simulates joints within a rock mass in three dimensions and generates two dimensional joint trace maps at selected views thereby requiring simplifying assumptions to be made on the three dimensional extent of joint bound blocks. The three dimensional form of the conceptual model is illustrated in Figure 2.

2.2.2 Generation of Rock Mass Structural Patterns

The generation process adopted in the simple rock mass structural model utilizes the Monte Carlo technique for sampling from the chosen distributional forms that represent the statistical nature of each of the joint set parameters. The first step in the generation process is the selection of the required orientation view or "slice" through the rock mass. Both joint trace map scale and plot size can be selected to conform to the purpose for which the joint trace map is intended.

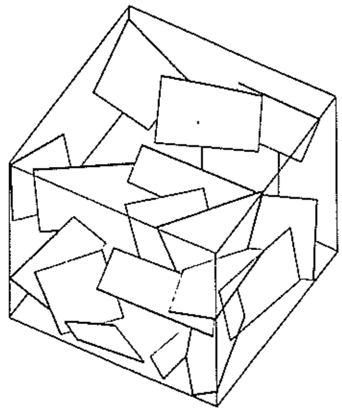

Figure 2 - 3D Conceptual Model

The generation process is based on the simulation of joints along one dimensional strips that are assembled into the 2D joint trace map for the selected view. The orientation at which the one dimensional strips are plotted are based on the mean projected orientation of each joint set for the selected view. The second step in the generation process is the calculation of the average trace length for each joint set as it would appear on the selected view. These values are determined from the distributions of apparent joint trace lengths which are automatically produced after consideration of the given ranges in the strike and dip lengths with respect to the orientation of the selected view. The direction of each plotting traverse is perpendicular to the average trace orientation of each joint set and the trace lengths plotted on the 2D map represent the external appearances of the joint planes as they would be intersected by an excavation surface. Figure 1 illustrates a complete joint trace map generated by the simple rock mass structural model with four joint sets.

3. STATISTICAL EVALUATION OF STABILITY AND SUPPORT REQUIREMENTS

3.1 Identification of Potentially Unstable Blocks

The identification of potentially unstable blocks from selected joint trace maps (Figure 1) requires the definition of representative geometric criteria. This simplified 2D approach assumes that identified potentially unstable blocks from joint trace maps are 3D joint bound blocks that will move into the excavation if not supported. In addition, no direct consideration is given to the influence of in situ stress or the cohesive or frictional components of strength along joint surfaces.

The definition of geometric criteria for the identification of potentially unstable blocks has been based on considering three different block type configurations. The three configurations have been considered irrespectively of the orientation of the excavation surfaces of interest and comprise triangular, rectangular and trapezoidal shaped blocks.

The variation in the geometry of individual blocks is inherent in the formation of potentially unstable blocks due to the interaction between the rock mass structural patterns and the excavation geometry in three dimensions. As part of the simplified 2D approach for the assessment of excavation stability and support requirements an attempt has been made to take into account of the possible variation in the geometry of individual blocks by considering the relative volumes of a unit rectangular block. From the recognition of the variable block shapes of elongate triangular wedges, rectangular blocks and irregular saw-toothed shaped polygons the volumes of blocks may be considered to vary significantly from 1/3 to the total volume of a unit rectangular block.

3.2 Quantification of Potentially Unstable Blocks

The quantification of potentially unstable blocks is conducted on a statistical basis to produce distributions for block depths, block areas and individual block volumes. Repetitive procedures including the random superimposition of excavation geometry on the joint trace map, the identification of potentially unstable blocks, the measurement and tabulation of the geometry of the blocks as well as their occurrence are required before the statistical nature of block geometry is characterized with the construction of standard histograms.

The choice and number of joint trace maps that are selected are a function of the excavation geometry but generally cross, longitudinal and plan sections are adequate. The measurement of block depths is conducted on vertical cross sections and block depth is defined as the maximum distance measured at an angle approximately perpendicular to the excavation profile. In most cases the maximum distance corresponds to the apex position of the joint bound block. Block area is measured on longitudinal and plan sections and is defined as the area completely bounded by joint traces.

The occurrence of potentially unstable blocks is characterized by the number of joint bound blocks identified in the longitudinal or plan sections. The number of blocks is further defined in relation to an excavation advance dimension such as a bench height for a chamber excavation or a blast advance round for a tunnel. The excavation advance dimension therefore defines an area on the respective selected sections for the identification of potentially unstable blocks.

The quantification of potentially unstable block volumes is based on the consideration of the histograms constructed for block depths, block areas and the block volume factor. The distribution of individual block volumes is constructed by calculating the mathematical products of the sampling of the histograms for block depths, block areas and the block volume factor. The block volume factor varies between 1/3 and unity and is considered to include the possible variation in block geometry. A diagrammatic representation of the construction of the distribution of individual potentially unstable block volumes is illustrated in Figure 3. A similar procedure is used to derive a distribution for total block volumes based on the distributions for individual block volumes and the number of potentially unstable blocks.

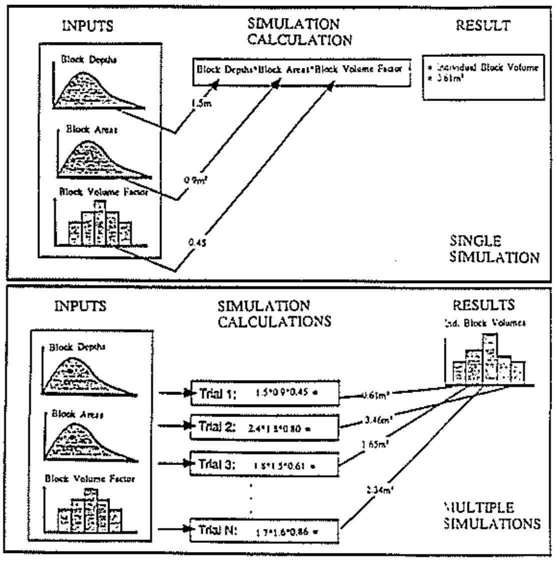

Figure 3 - Construction of Block Volume Distribution

The number of simulation calculations required using the Monte Carlo sampling technique to reasonably estimate the maximum individual potentially unstable block volume has been investigated and to be within a 20% error a total of N=5000 simulations were found to be required. The maximum individual potentially unstable block volume is of concern when it is required to specify the capacity of the rock bolts to be installed.

4. PROBABILISTIC EVALUATION OF STABILITY AND SUPPORT REQUIREMENTS

4.1 Describing Probability and Confidence Levels

The reliability-based approach to the assessment of excavation stability and support requirements is based on the characterization of the various statistical distributions describing block geometry in probabilistic terms. This characterization requires the construction of the relative cumulative frequency distributions for individual block volumes and block depths. The construction of a relative cumulative frequency distribution is a straight forward exercise from knowledge of the relative frequency histogram. From the relative cumulative frequency distribution it is also easy to determine the probabilities that a random variable will take a value less than or equal to a selected numerical value or a range of values In the context of evaluating excavation stability in terms of individual potentially unstable block volumes and support requirements in terms of block depths it is of concern to examine critical values that are not to be exceeded within a specified degree of certainty. In several engineering applications it is statistical confidence levels that can be considered for characterizing the degree of certainty in probabilistic terms.

4.2 Reliability-Based Predictions of Instability

The probabilistic assessment of excavation instability involves the consideration of the relative cumulative frequency distribution for individual block volumes. The shape of the cumulative frequency distribution for individual block volumes is a function of the respective relative frequency histogram which in turn is an unknown function of the distributional forms for block depths, block areas and the block volume factor. It has been found that if one of the distributions for block depths, block areas or block volume factor is negative exponential, then the distributional form for individual block volumes is also likely to be of this form.

The method of presenting a quantitative assessment of excavation stability is based on the examination of the probabilities of encountering potentially unstable volumes of rock as joint bound blocks. The percentage probability of encountering a particular potentially unstable volume is equal to the difference of the selected critical relative cumulative frequency percentage from 100%. The selection of critical relative cumulative frequency percentage values is somewhat arbitrary however for most engineering applications it has ben common to examine values of the random variable of interest corresponding to the 95% and 99% probabilities.

4.3 Reliability-Based Predictions of Support Requirements

The probabilistic assessment of excavation instability involves the consideration of the relative cumulative frequency distributions for block depths and block numbers. Support requirements are based solely on rock bolts with specified lengths and spacings for restraining potentially unstable blocks. The require rock bolt length is defined as being equal to the maximum block depth plus a nominal length for anchoring into solid rock behind the potentially unstable block.

The probabilistic assessment of rock bolt lengths is based on selected probabilities from the relative cumulative frequency distribution for block depths with consideration for a specified nominal anchor length. Consideration of the nominal anchor length and an overall practical rock bolt length can be accommodated and represented as an upper bound curve superimposed on the relative cumulative frequency distribution for block depths.

The probabilistic assessment of rock bolt spacings is based on selected critical probabilities from the relative cumulative frequency distribution for the number of potentially unstable blocks in relation to a specified excavation advance dimension. The selected probabilities will correspond to a particular number of potentially unstable blocks that are then addressed in relation to the exposed surface area of the excavation that is a result of the excavation advance dimension.

Rock bolt spacings are based only on square patterns and are calculated by dividing the number of potentially unstable blocks at the selected critical probability level into the surface area of excavation and then taking the square root of the resultant quotient. This spacing value can then be adjusted accordingly to a practical value. The procedure for determining rock bolt spacings assumes that only a single rock bolt is required to support a single potentially unstable block and this assumption can be verified by considering the distribution of individual block volumes.

5. CASE STUDY ROCK EXCAVATIONS

5.1 General

In order to investigate the applicability of the reliability-based approach for the assessment of stability and support requirements three case study rock excavations have been examined. The three case study rock excavations comprise a short and relatively small size tunnel, a medium size underground chamber and a small rock slope cutting. A complete reliability-based assessment of excavation stability and support requirements for each of the three case study rock excavations is beyond the presentation limitations of this paper. As such, representative aspects of each of the overall assessments for each excavation have therefore been chosen and are presented.

5.2 Rock Tunnel

The small size tunnel is situated in dolomitic rock characterized with four prominent joint sets. Joint trace maps corresponding to the cross, longitudinal and plan sections of the tunnel were generated to identify potentially unstable blocks. A distinction was made to identify block depths from both the crown and sidewall zones of the tunnel cross section. Graphical representations of crown and sidewall block depths, block areas and block numbers as histograms were constructed in order to generate a distribution for individual block volumes per 3m excavation advance.

For this case study excavation the distribution of crown block depths is presented in Figure 4 and shows that crown block depths vary between 0.3m and 5.4m with a mean block depth of 1.1m. The corresponding cumulative frequency distribution for both crown and sidewall block depths is illustrated in Figure 5. Block depths at 75%, 90%, 95% and 99% confidence levels are highlighted and reflect the significantly greater block depths for the sidewall zone as compared to the crown zone of the tunnel excavation. The length of rock bolts required corresponding to the highlighted confidence levels and including a 300mm anchor length are 1.7m, 2.3m, 2.9m and 4.2m for the crown zone and are 2.8m, 3.5m, 4.2m and 4.7m for the sidewall zone respectively.

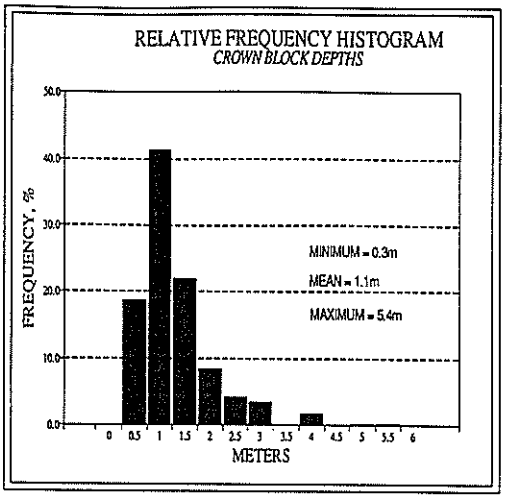

Figure 4 - Frequency Histogram for Crown Block Depths

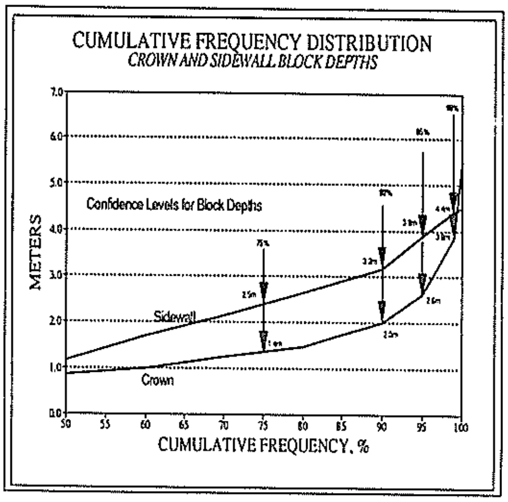

Figure 5 - CFD for Crown Block Depths

5.3 Underground Chamber

The medium size underground chamber is situated in an igneous complex of carbonatite rocks characterized with four prominent joint sets. Joint trace maps corresponding to the four walls and the plan section of the chamber were generated to identify potentially unstable blocks. Graphical representations of wall block depths, block areas and block numbers as histograms were constructed in order to generate a distribution for individual block volumes for each of the four walls per 2.5m bench excavation lift and for the roof zone per 3m excavation advance.

For this case study excavation the distribution of individual block volumes for the northeast wall is presented in Figure 6 and shows that the mean and maximum individual block volumes are 1.7m^3 and 7.3m^3 respectively. The corresponding cumulative frequency distribution for the individual block volumes of all four of the chamber walls is illustrated in Figure 7. The 25%, 10%, 5% and 1% probabilities of encountering potentially unstable block volumes per excavation advance are highlighted and reflect the significantly greater block volumes for the crown zone as compared to the four walls of the chamber excavation. At the highlighted probabilities of occurrence the individual block volumes for the crown zone correspond to 2.3m^3, 3.6m^3, 4.6m^3 and 6.8m^3 respectively. Similar cumulative frequency distributions however do exist for the four walls of the chamber excavation.

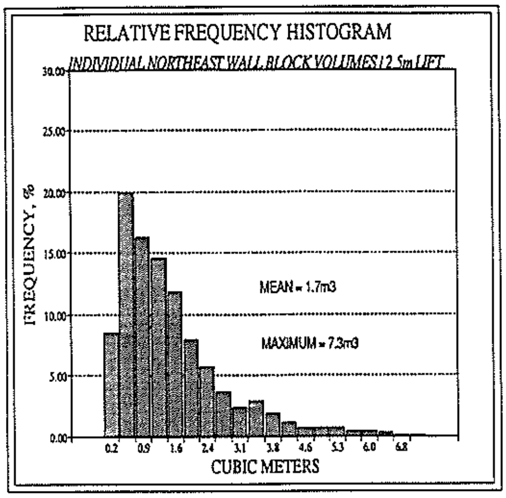

Figure 6 - Frequency Histogram for Block Volumes

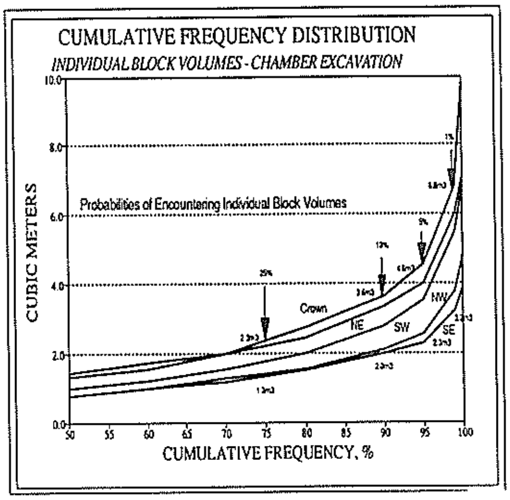

Figure 7 - CFD for Individual Block Volumes

5.4 Rock Slope

The small rock slope cutting is situated within an alternating bedded formation of quartzites and conglomerates characterized with five prominent joint sets. Joint trace maps corresponding to the cross, longitudinal and plan sections of the rock slope cutting were generated to identify potentially unstable blocks. A comparison was made to identify block depths from both 40 degree and 60 degree slope angles. Graphical representations of slope face block depths, exposed slope face block lengths,slope face block area and slope face block numbers as histograms were constructed in order to generate a distribution for individual block volumes per 20m excavation slope length.

For this case study excavation the distribution of slope face block depths for the 60 degree slope angle is presented in Figure 8 and shows that slope face block depths vary between 0.3m and 5.2m with a mean block depth of 2.0m. The corresponding cumulative frequency distribution for slope face block depths for both the 60 degree and 40 degree slope cuttings is illustrated in Figure 9. Slope face block depths at 75%, 90%, 95% and 99% confidence levels are highlighted and reflect the significantly greater slope face block depths for the 60 degree slope as compared to the 40 degree slope cutting. The length of rock bolts required corresponding to the highlighted confidence levels and including a 300mm anchor length are 2.9m, 3.9m, 4.5m and 5.3m for the 60 degree slope cutting and are 2.4m, 3.2m, 4.1m and 4.7m for the 40 degree slope cutting respectively.

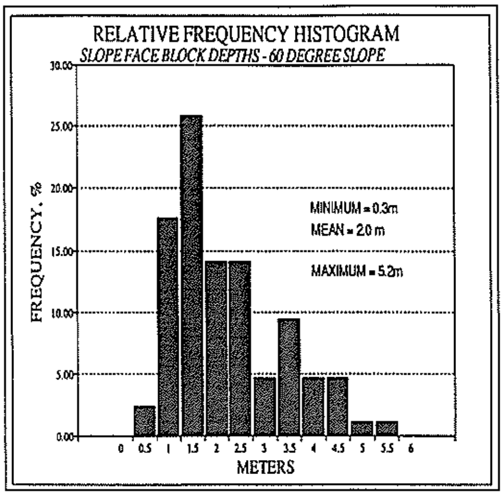

Figure 8 - Frequency Histogram for Slope Block Depths

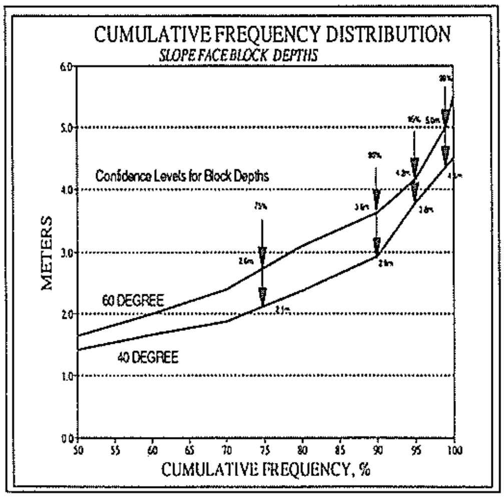

Figure 9 - CFD for Slope Block Depths

6. DISCUSSION

6.1 Reliability-Based Predictions

The reliability-based predictions for excavation stability that are represented by the relative frequency histograms and cumulative frequency distributions for individual block volumes are plausible upon consideration of the distributions for joint spacing of the joint sets for each of the case study rock excavations. As the mean joint spacings of the joint sets increased from that for the tunnel excavation to that for the rock slope cutting, so has the mean individual block volume as would be expected.

The reliability-based predictions for excavation support requirements that are represented by the relative frequency histograms and cumulative frequency distributions for block depths are also plausible upon consideration of the distributions for joint spacing for the joint sets of each of the case study rock excavations. The relatively smaller spacings of the sub-vertical joint sets corresponding to the tunnel excavation have been shown to yield overall shorter block depths for the crown section in comparison to the wider spacings of the joint sets corresponding to the chamber excavation that have yielded greater block depths for the crown section. Thus, as would be expected, wider joint spacings of sub-vertical joint sets have produced greater block depths along a horizontal excavation surface. Furthermore, the spacing of bedding structure present within a rock mass can be considered to significantly

Table I - Comparison of Predicted/Practical and Recommended/Installed Rock Bolt Lengths

	Rock Tunnel		Underground Chamber		Rock Slope	
	Crown	Sidewall	Crown	Walls	40 degrees	60 degrees
Predicted (95%)	2.9m	4.2m	2.7-3.2m	4.4m	4.1m	4.5m
Recommended/Installed	3.0m	3.0m	4.0m	3.0m	4.0m	4.0m

Table II - Comparison of Predicted/Practical and Recommended/Applied Rock Bolt Spacings

	Rock Tunnel		Underground Chamber		Rock Slope	
	Crown	Sidewall	Crown	Walls	40 degrees	60 degrees
Predicted (95%)	2.5m	2.0m	6.0m	6.5m	8.0m	6.5m
Recommended/Applied	1.5m	1.5m	1.5m	1.0m	2.5m*	2.5m*

influence the depths of potentially unstable blocks in the sidewall of an excavation. The relatively similar spacings of bedding structure for the tunnel excavation and the rock slope cutting have not surprisingly yielded similar distributions for block depths for the respective sidewall and slope face sections.

6.2 Comparison of Reliability-Based Predictions and In Situ Observations

Comparisons of the reliability-based predictions for support requirements in terms of both rock bolt lengths and spacings are made with actual and/or recommended support requirements for each of the case study rock excavations and a summary of this information is presented in Table I and II. The recommended support requirements have been generally based on consideration of rock mass classifications and rock engineering industry standards.

The predicted rock bolt lengths at the 95% confidence level are in reasonable agreement with the recommended /installed rock bolt lengths for all three case study rock excavations. The recommended rock bolt lengths for the sidewall of the tunnel and the walls of the chamber excavation correspond to rock bolt lengths at lower confidence levels as may be justified in terms of overall safety. It is noted that no major instabilities resulted in any of the case study rock excavations with the installation of the prescribed rock reinforcement.

The predicted rock bolt spacings at the 95% confidence level are in good agreement for the tunnel excavation. The rock bolt spacings at the same confidence level for the chamber excavation and the rock slope cutting correspond to spot bolting requirements in the practical sense and are not in agreement with the recommended or installed rock bolt spacings.

7. CONCLUSIONS

The good agreement seen upon the comparison of reliability-based predictions of rock bolt lengths with recommended/installed lengths for three case study rock excavations reflects the applicability of this approach, in particular for the preliminary design stage of a rock engineering project. In addition, on the basis of the assessment for the three case study rock excavations it is concluded that this approach may be more applicable for assessing rock support requirements for rock masses that are characterized with closely spacing jointing where there is a strong likelihood of instability.

The reliability-based approach has recently been adopted for the assessment of preliminary rock support requirements for a twin arrangement of 17 metre span tunnels to be sited in closely jointed granitic rock in Hong Kong.

8. ACKNOWLEDGEMENT

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