When is a rock engineering design acceptable

Introduction

When is a design in rock engineering acceptable? The aim of the following text[1] is to demonstrate that there are no simple universal rules for acceptability nor are there standard factors of safety which can be used to guarantee that a rock structure will be safe and that it will perform adequately. Each design is unique and the acceptability of the structure has to be considered in terms of the particular set of circumstances, rock types, design loads and end uses for which it is intended. The responsibility of the geotechnical engineer is to find a safe and economical solution which is compatible with all the constraints which apply to the project. Such a solution should be based upon engineering judgement guided by practical and theoretical studies such as stability or deformation analyses, if and when these analyses are applicable.

Tables 1 to 4 summarise some of the typical problems, critical parameters, analysis methods and acceptability criteria which apply to a number of different rock engineering structures. These examples have been drawn from my own consulting experience and I make no claims that this is a complete list nor do I expect readers to agree with all of the items which I have included under the various headings. The purpose of presenting these tables is to demonstrate the diversity of problems and criteria which have to be considered and to emphasise the dangers of attempting to use standard factors of safety or other acceptability criteria.

In order to amplify some of the items included in Tables 1 to 4, several case histories will be discussed in terms of the factors which were considered and the acceptability criteria which were used.

Landslides in reservoirs

The presence of unstable slopes in reservoirs is a major concern for the designers of dams for hydroelectric and irrigation projects. The Vajont failure in 1963 alerted the engineering community of the danger of underestimating the potential for the mobilisation of existing landslides as a result of submergence of the slide toe during impounding of the reservoir.

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[1]Based upon the text of the Müller lecture presented at the 7th Congress of the International Society for Rock Mechanics held in Aachen, Germany, in September 1991.

Table 1 : Typical problems, critical parameters, methods of analysis and acceptability criteria for slopes.
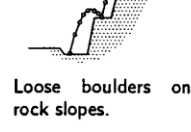
STRUCTURE	TYPICAL PROBLEMS	CRITICAL PARAMETERS	ANALYSIS METHODS	ACCEPTABILITY CRITERIA
Landslides.	Complex failure along a circular or near circular failure surface involving sliding on faults and other structural features as well as failure of intact materials.	• Presence of regional faults. • Shear strength of materials along failure surface. • Groundwater distribution in slope, particularly in response to rainfall or to submergence of slope toe. • Potential earthquake loading.	Limit equilibrium methods which allow for non-circular failure surfaces can be used to estimate changes in factor of safety as a result of drainage or slope profile changes. Numerical methods such as finite element or discrete element analysis can be used to investigate failure mechanisms and history of slope displacement.	Absolute value of factor of safety has little meaning but rate of change of factor of safety can be used to judge effectiveness of remedial measures. Long term monitoring of surface and subsurface displacements in slope is the only practical means of evaluating slope behaviour and effectiveness of remedial action.
Soil or heavily jointed rock slopes.	Circular failure along a spoon-shaped surface through soil or heavily jointed rock masses.	• Height and angle of slope face. • Shear strength of materials along failure surface. • Groundwater distribution in slope. • Potential surcharge or earthquake loading.	Two-dimensional limit equilibrium methods which include automatic searching for the critical failure surface are used for parametric studies of factor of safety. Probability analyses, three-dimensional limit equilibrium analyses or numerical stress analyses are occasionally used to investigate unusual slope problems.	Factor of safety > 1.3 for "temporary" slopes with minimal risk of damage. Factor of safety > 1.5 for "permanent" slopes with significant risk of damage. Where displacements are critical, numerical analyses of slope deformation may be required and higher factors of safety will generally apply in these cases.
Jointed rock slopes.	Planar or wedge sliding on one structural feature or along the line of intersection of two structural features.	• Slope height, angle and orientation. • Dip and strike of structural features. • Groundwater distribution in slope. • Potential earthquake loading. • Sequence of excavation and support installation.	Limit equilibrium analyses which determine three-dimensional sliding modes are used for parametric studies on factor of safety. Failure probability analyses, based upon distribution of structural orientations and shear strengths, are useful for some applications.	Factor of safety > 1.3 for "temporary" slopes with minimal risk of damage. Factor of safety > 1.5 for "permanent" slopes with significant risk of damage. Probability of failure of 10 to 15% may be acceptable for open pit mine slopes where cost of clean up is less than cost of stabilization.
Vertically jointed rock slopes .	Toppling of columns separated from the rock mass by steeply dipping structural features which are parallel or nearly parallel to the slope face.	• Slope height, angle and orientation. • Dip and strike of structural features. • Groundwater distribution in slope. • Potential earthquake loading.	Crude limit equilibrium analyses of simplified block models are useful for estimating potential for toppling and sliding. Discrete element models of simplified slope geometry can be used for exploring toppling failure mechanisms.	No generally acceptable criterion for toppling failure is available although potential for toppling is usually obvious. Monitoring of slope displacements is the only practical means of determining slope behaviour and effectiveness of remedial measures.
Loose boulders on rock slopes.	Sliding, rolling, falling and bouncing of loose rocks and boulders on the slope.	• Geometry of slope. • Presence of loose boulders. • Coefficients of restitution of materials forming slope. • Presence of structures to arrest falling and bouncing rocks.	Calculation of trajectories of falling or bouncing rocks based upon velocity changes at each impact is generally adequate. Monte Carlo analyses of many trajectories based upon variation of slope geometry and surface properties give useful information on distribution of fallen rocks.	Location of fallen rock or distribution of a large number of fallen rocks will give an indication of the magnitude of the potential rockfall problem and of the effectiveness of remedial measures such as draped mesh, catch fences and ditches at the toe of the slope.

Table 2 : Typical problems, critical parameters, methods of analysis and acceptability criteria for dams and foundations.
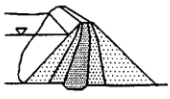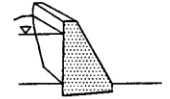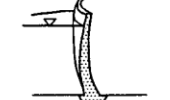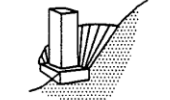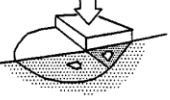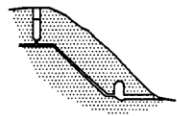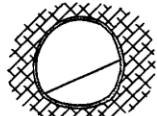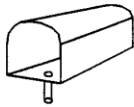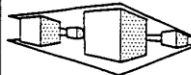
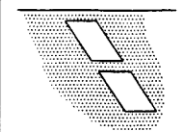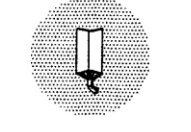| STRUCTURE	TYPICAL PROBLEMS	CRITICAL PARAMETERS	ANALYSIS METHODS	ACCEPTABILITY CRITERIA
Zoned fill dams.	Circular or near-circular failure of dam, particularly during rapid drawdown. Foundation failure on weak seams. Piping and erosion of core.	• Presence of weak or permeable zones in foundation. • Shear strength, durability, gradation and placement of dam construction materials, particularly filters. • Effectiveness of grout curtain and drainage system. • Stability of reservoir slopes.	Seepage analyses are required to determine water pressure and velocity distribution through dam and abutments. Limit equilibrium methods should be used for parametric studies of stability. Numerical methods can be used to investigate dynamic response of dam during earthquakes.	Safety factor >1.5 for full pool with steady state seepage; >1.3 for end of construction with no reservoir loading and undissipated foundation porewater pressures; >1.2 for probable maximum flood with steady state seepage and >1.0 for full pool with steady state seepage and maximum credible horizontal psuedo-static seismic loading.
Gravity dams.	Shear failure of interface between concrete and rock or of foundation rock. Tension crack formation at heel of dam. Leakage through foundation and abutments.	• Presence of weak or permeable zones in rock mass. • Shear strength of interface between concrete and rock. • Shear strength of rock mass. • Effectiveness of grout curtain and drainage system. • Stability of reservoir slopes.	Parametric studies using limit equilibrium methods should be used to investigate sliding on the interface between concrete and rock and sliding on weak seams in the foundation. A large number of trial failure surfaces are required unless a non-circular failure analysis with automatic detection of critical failure surfaces is available.	Safety factor against foundation failure should exceed 1.5 for normal full pool operating conditions provided that conservative shear strength values are used ($c' \approx 0$). Safety factor > 1.3 for probable mximum food (PMF). Safety factor > 1 for extreme loading - maximum credible earthquake and PMF.
Arch dams.	Shear failure in foundation or abutments. Cracking of arch due to differential settlements of foundation. Leakage through foundations or abutments.	• Presence of weak, deformable or permeable zones in rock mass. • Orientation, inclination and shear strength of structural features. • Effectiveness of grout curtain and drainage system. • Stability of reservoir slopes.	Limit equilibrium methods are used for parametric studies of three-dimensional sliding modes in the foundation and abutments, including the influence of water pressures and reinforcement. Three-dimensional numerical analyses are required to determine stresses and displacements in the concrete arch.	Safety factor against foundation failure >1.5 for normal full pool operating conditions and >1.3 for probable maximum flood conditions provided that conservative shear strength values are used ($c' \approx 0$). Stresses and deformations in concrete arch should be within allowable working levels defined in concrete specifications.
Foundations on rock slopes.	Slope failure resulting from excessive foundation loading. Differential settlement due to anisotropic deformation properties of foundation rocks.	• Orientation, inclination and shear strength of structural features in rock mass forming foundation. • Presence of inclined layers with significantly different deformation properties. • Groundwater distribution in slope.	Limit equilibrium analyses of potential planar or wedge failures in the foundation or in adjacent slopes are used for parametric studies of factor of safety. Numerical analyses can be used to determine foundation deformation, particularly for anisotropic rock masses.	Factor of safety against sliding of any potential foundation wedges or blocks should exceed 1.5 for normal operating conditions. Differential settlement should be within limits specified by structural engineers.
Foundations on soft rock or soil.	Bearing capacity failure resulting from shear failure of soils or weak rocks underlying foundation slab.	• Shear strength of soil or jointed rock materials. • Groundwater distribution in soil or rock foundation. • Foundation loading conditions and potential for earthquake loading.	Limit equilibrium analyses using inclined slices and non-circular failure surfaces are used for parametric studies of factor of safety. Numerical analyses may be required to determine deformations, particularly for anisotropic foundation materials.	Bearing capacity failure should not be permitted for normal loading conditions. Differential settlement should be within limits specified by structural engineers.

Table 3 : Typical problems, critical parameters, methods of analysis and acceptability criteria for underground civil engineering excavations.

STRUCTURE	TYPICAL PROBLEMS	CRITICAL PARAMETERS	ANALYSIS METHODS	ACCEPTABILITY CRITERIA
Pressure tunnels in hydro-power projects.	Excessive leakage from unlined or concrete lined tunnels. Rupture or buckling of steel lining due to rock deformation or external pressure.	• Ratio of maximum hydraulic pressure in tunnel to minimum principal stress in the surrounding rock. • Length of steel lining and effectiveness of grouting. • Groundwater levels in the rock mass.	Determination of minimum cover depths along pressure tunnel route from accurate topographic maps. Stress analyses of sections along and across tunnel axis. Comparison between minimum principal stresses and maximum dynamic hydraulic pressure to determine steel lining lengths.	Steel lining is required where the minimum principal stress in the rock is less than 1.3 times the maximum static head for typical hydroelectric operations or 1.15 for operations with very low dynamic pressures. Hydraulic pressure testing in boreholes at the calculated ends of the steel lining is essential to check the design assumptions.
Soft rock tunnels.	Rock failure where strength is exceeded by induced stresses. Swelling, squeezing or excessive closure if support is inadequate.	• Strength of rock mass and of individual structural features. • Swelling potential, particularly of sedimentary rocks. • Excavation method and sequence. • Capacity and installation sequence of support systems.	Stress analyses using numerical methods to determine extent of failure zones and probable displacements in the rock mass. Rock-support interaction analyses using closed-form or numerical methods to determine capacity and installation sequence for support and to estimate displacements in the rock mass.	Capacity of installed support should be sufficient to stabilize the rock mass and to limit closure to an acceptable level. Tunnelling machines and internal structures must be designed for closure of the tunnel as a result of swelling or time-dependent deformation. Monitoring of deformations is an important aspect of construction control.
Shallow tunnels in jointed rock.	Gravity driven falling or sliding wedges or blocks defined by intersecting structural features. Unravelling of inadequately supported surface material.	• Orientation, inclination and shear strength of structural features in the rock mass. • Shape and orientation of excavation. • Quality of drilling and blasting during excavation. • Capacity and installation sequence of support systems.	Spherical projection techniques or analytical methods are used for the determination and visualization of all potential wedges in the rock mass surrounding the tunnel. Limit equilibrium analyses of critical wedges are used for parametric studies on the mode of failure, factor of safety and support requirements.	Factor of safety, including the effects of reinforcement, should exceed 1.5 for sliding and 2.0 for falling wedges and blocks. Support installation sequence is critical and wedges or blocks should be identified and supported before they are fully exposed by excavation. Displacement monitoring is of little value.
Large caverns in jointed rock.	Gravity driven falling or sliding wedges or tensile and shear failure of rock mass, depending upon spacing of structural features and magnitude of in situ stresses.	• Shape and orientation of cavern in relation to orientation, inclination and shear strength of structural features in the rock mass. • In situ stresses in the rock mass. • Excavation and support sequence and quality of drilling and blasting.	Spherical projection techniques or analytical methods are used for the determination and visualization of all potential wedges in the rock mass. Stresses and displacements induced by each stage of cavern excavation are determined by numerical analyses and are used to estimate support requirements for the cavern roof and walls.	An acceptable design is achieved when numerical models indicate that the extent of failure has been controlled by installed support, that the support is not overstressed and that the displacements in the rock mass stabilize. Monitoring of displacements is essential to confirm design predictions.
Underground nuclear waste disposal.	Stress and/or thermally induced spalling of the rock surrounding the excavations resulting in increased permeability and higher probability of radioactive leakage.	• Orientation, inclination, permeability and shear strength of structural features in the rock mass. • In situ and thermal stresses in the rock surrounding the excavations. • Groundwater distribution in the rock mass.	Numerical analyses are used to calculate stresses and displacements induced by excavation and by thermal loading from waste canisters. Groundwater flow patterns and velocities, particularly through blast damaged zones, fissures in the rock and shaft seals are calculated using numerical methods.	An acceptable design requires extremely low rates of groundwater movement through the waste canister containment area in order to limit transport of radioactive material. Shafts, tunnels and canister holes must remain stable for approximately 50 years to permit retrieval of waste if necessary.

Table 4 : Typical problems, critical parameters, methods of analysis and acceptability criteria for underground hard rock mining excavations.

STRUCTURE	TYPICAL PROBLEMS	CRITICAL PARAMETERS	ANALYSIS METHODS	ACCEPTABILITY CRITERIA
Pillars.	Progressive spalling and slabbing of the rock mass leading to eventual pillar collapse or rockbursting.	• Strength of the rock mass forming the pillars. • Presence of unfavourably oriented structural features. • Pillar geometry, particularly width to height ratio. • Overall mine geometry including extraction ratio.	For horizontally bedded deposits, pillar strength from empirical relationships based upon width to height ratios and average pillar stress based on tributary area calculations are compared to give a factor of safety. For more complex mining geometry, numerical analyses including progressive pillar failure may be required.	Factor of safety for simple pillar layouts in horizontally bedded deposits should exceed 1.6 for "permanent" pillars. In cases where progressive failure of complex pillar layouts is modelled, individual pillar failures can be tolerated provided that they do not initiate "domino" failure of adjacent pillars.
Crown pillars.	Caving of surface crown pillars for which the ratio of pillar depth to stope span is inadequate. Rockbursting or gradual spalling of overstressed internal crown pillars.	• Strength of the rock mass forming the pillars. • Depth of weathering and presence of steeply dipping structural features in the case of surface crown pillars. • In situ stress levels and geometry of internal crown pillars.	Rock mass classification and limit equilibrium analyses can give useful guidance on surface crown pillar dimensions for different rock masses. Numerical analyses, including discrete element studies, can give approximate stress levels and indications of zones of potential failure.	Surface crown pillar depth to span ratio should be large enough to ensure very low probability of failure. Internal crown pillars may require extensive support to ensure stability during mining of adjacent stopes. Careful planning of mining sequence may be necessary to avoid high stress levels and rockburst problems.
Cut and fill stopes.	Falls of structurally defined wedges and blocks from stope backs and hanging walls. Stress induced failures and rockbursting in high stress environments.	• Orientation, inclination and shear strength of structural features in the rock mass. • In situ stresses in the rock mass. • Shape and orientation of stope. • Quality, placement and drainage of fill.	Numerical analyses of stresses and displacements for each excavation stage will give some indication of potential problems. Some of the more sophisticated numerical models will permit inclusion of the support provided by fill or the reinforcement of the rock by means of grouted cables.	Local instability should be controlled by the installation of rockbolts or grouted cables to improve safety and to minimize dilution. Overall stability is controlled by the geometry and excavation sequence of the stopes and the quality and sequence of filling. Acceptable mining conditions are achieved when all the ore is recovered safely.
Non-entry stopes.	Ore dilution resulting from rockfalls from stope back and walls. Rockbursting or progressive failure induced by high stresses in pillars between stopes.	• Quality and strength of the rock. • In situ and induced stresses in the rock surrounding the excavations. • Quality of drilling and blasting in excavation of the stope.	Some empirical rules, based on rock mass classification, are available for estimating safe stope dimensions. Numerical analyses of stope layout and mining sequence, using three-dimensional analyses for complex orebody shapes, will provide indications of potential problems and estimates of support requirements.	A design of this type can be considered acceptable when safe and low cost recovery of a large proportion of the orebody has been achieved. Rockfalls in shafts and haulages are an unacceptable safety hazard and pattern support may be required. In high stress environments, local destressing may be used to reduce rockbursting.
Drawpoints and ore-passes.	Local rock mass failure resulting from abrasion and wear of poorly supported drawpoints or orepasses. In extreme cases this may lead to loss of stopes or orepasses.	• Quality and strength of the rock. • In situ and induced stresses and stress changes in the rock surrounding the excavations. • Selection and installation sequence of support.	Limit equilibrium or numerical analyses are not particularly useful since the processes of wear and abrasion are not included in these models. Empirical designs based upon previous experience or trial and error methods are generally used.	The shape of the opening should be maintained for the design life of the drawpoint or orepass. Loss of control can result in serious dilution of the ore or abandonment of the excavation. Wear resistant flexible reinforcement such as grouted cables, installed during excavation of the opening, may be successful in controlling instability.

During the construction of the Mica and Revelstoke dams on the Columbia River in British Columbia, Canada, several potential slides were investigated. Two of these, the Downie Slide, a 1.4 billion cubic metre ancient rock slide, and Dutchman's Ridge, a 115 million cubic metre potential rock slide, were given special attention because of the serious consequences which could have resulted from failure of these slides (Imrie, 1983, Lewis and Moore, 1989, Imrie, Moore and Enegren, 1992).

The Downie Slide and Dutchman's Ridge are located in steep, narrow, V-shaped sections of the Columbia River valley which has been subjected to several episodes of glaciation. The bedrock at these sites consists mainly of Pre-Cambrian para-gneisses and schists within or on the fringe of the Shuswap Metamorphic Complex. In both cases, the potential slide planes, determined by diamond drilling and slope displacement monitoring, are relatively flat-lying outward-dipping tectonic faults or shears which daylight in the base of the river valley.

Based on thorough investigation and monitoring programs, British Columbia Hydro and Power Authority (BC Hydro) decided that remedial measures had to be taken to improve the stability of both the Downie Slide and Dutchman's Ridge. These remedial measures consisted of drainage adits extending within and/or behind the failure surfaces and supplemented by drainholes drilled from chambers excavated along the adits. Work on the Downie Slide was carried out in the period 1977 to 1982 (which included a 3 year observation period) and work on Dutchman's Ridge was carried out from 1986 to 1988.
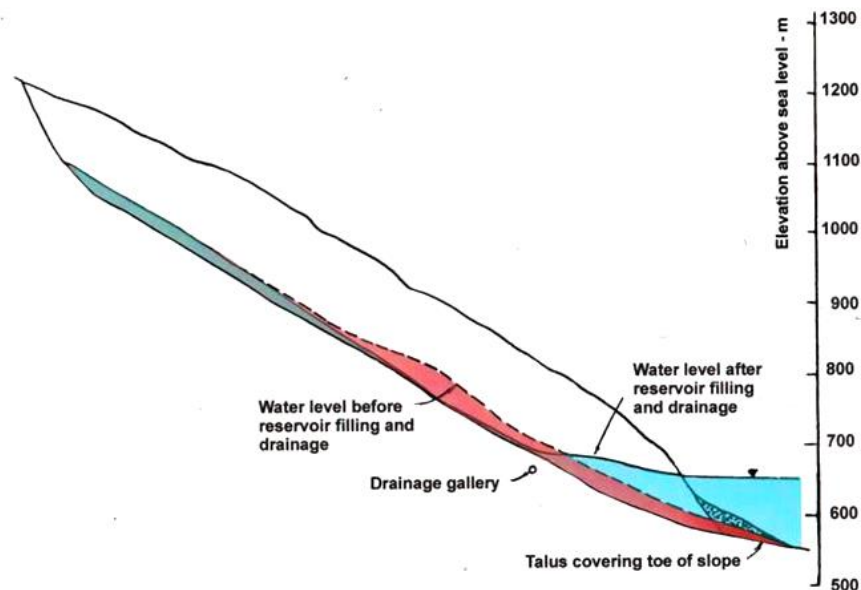

Figure 1: Section through Dutchman's Ridge showing potential slide surface and water levels before and after drainage.

A section through Dutchman's Ridge is given in Figure 1 and this shows the water levels in the slope before reservoir filling and after reservoir filling and the construction of the drainage system. Figure 2 shows contours of reduction in water levels as a result of the installation of the drainage system which consisted of 872 m of adit and 12,000 m of drainhole drilling. Note that the drawdown area on the right hand side of the potential slide was achieved by long boreholes from the end of the drainage adit branch.

Comparative studies of the stability of the slope section shown in Figure 1, based upon a factor of safety of 1.00 for the slope after reservoir filling but before implementation of the drainage system, gave a factor of safety of 1.06 for the drained slope. This 6% improvement in factor of safety may not seem very significant to the designer of small scale rock and soil slopes but it was considered acceptable in this case for a number of reasons:

1. The factor of safety of 1.00 calculated for the undrained slope is based upon a 'back-analysis' of observed slope behaviour. Provided that the same method of analysis and shear strength parameters are used for the stability analysis of the same slope with different groundwater conditions, the ratio of the factors of safety is a very reliable indicator of the change in slope stability, even if the absolute values of the factor of safety are not accurate. Consequently, the degree of uncertainty, which has to be allowed for in slope designs where no back-analyses have been performed, can be eliminated and a lower factor of safety accepted.
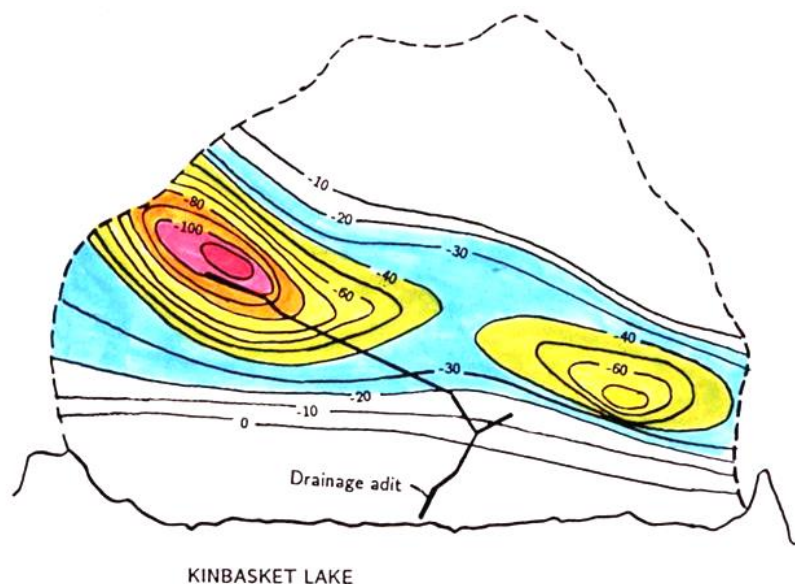

Figure 2: Contours of water level reduction (in metres) as a result of the implementation of drainage in Dutchman's Ridge.

2. The groundwater levels in the slope were reduced by drainage to lower than the pre-reservoir conditions and the stability of the slope is at least as good if not better than these pre-reservoir conditions. This particular slope is considered to have withstood several significant earthquakes during the 10,000 years since the last episode of glaciation which is responsible for the present valley shape.

3. Possibly the most significant indicator of an improvement in stability, for both the Downie Slide and Dutchman's Ridge, has been a significant reduction in the rate of down-slope movement which has been monitored for the past 25 years. In the case of the Downie Slide, this movement has practically ceased. At Dutchman's Ridge, the movements are significantly slower and it is anticipated that they will stabilize when the drainage system has been in operation for a few more years.

Deformation of rock slopes

In a slope in which the rock is jointed but where there are no significant discontinuities dipping out of the slope which could cause sliding, deformation and failure of the slope is controlled by a complex process of block rotation, tilting and sliding. In an extreme case, where the rock mass consists of near vertical joints separating columns of massive rock, toppling movement and failure may occur.
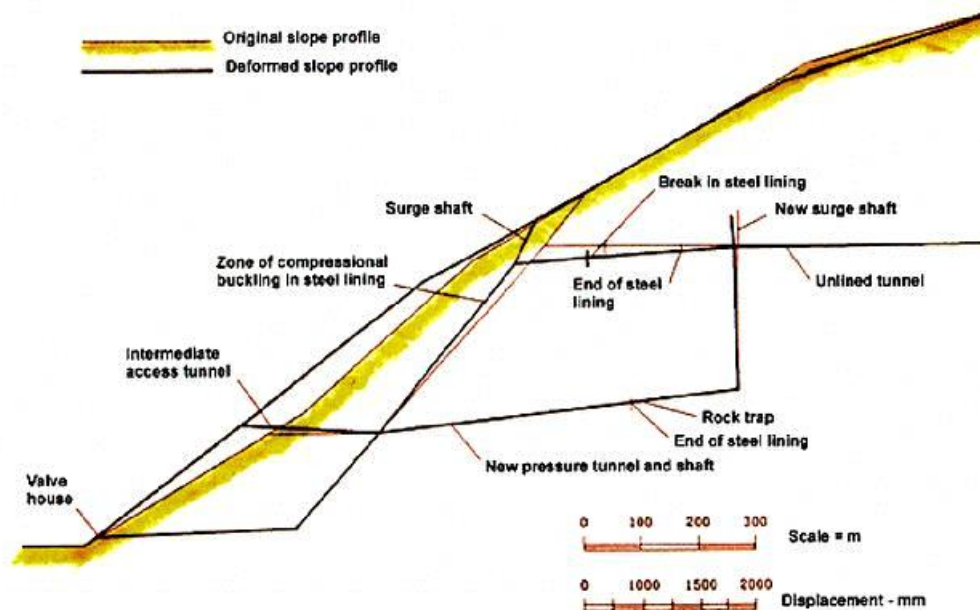

Figure 3: Cross-section through a section of the Wahleach power tunnel showing the original tunnel alignment and the location of the replacement conduit. The dashed line is the approximate location of a gradational boundary between loosened, fractured and weathered rock and more intact rock. Down-slope movement currently being monitored is well above this boundary.

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Figure 3 is a section through part of the power tunnel for the Wahleach hydroelectric project in British Columbia, Canada. A break in the steel lining in this power tunnel occurred in January 1989 and it is thought this break was caused by a slow down-slope gravitational movement caused by block rotations within a near-surface zone of loosened jointed rock.

The Wahleach project is located 120 km east of Vancouver and power is generated from 620 m of head between Wahleach Lake and a surface powerhouse located adjacent to the Fraser River. Water flows through a 3500 m long three metre diameter unlined upper tunnel, a rock trap, a 600 m two metre diameter concrete encased steel lined shaft inclined at 48° to the horizontal, a 300 m long lower tunnel and a 485 m long surface penstock to the powerhouse.

The tunnels were excavated mainly in granodiorite which varies from highly fractured and moderately weathered in the upper portions of the slope to moderately fractured and fresh in both the lower portions of the slope and below the highly fractured mass. Two main joint sets occur in the rock mass, one set striking parallel to the slope and the other perpendicular to it. Both dip very steeply. Average joint spacings range from 0.5 to 1 m. A few joints occur sub-parallel to the ground surface and these joints are most well developed in the ground surface adjacent to the inclined shaft. Thorough investigations failed to reveal any significant shear zones or faults conducive to sliding.

The toe of the slope is buried beneath colluvial and fan deposits from two creeks which have incised the Fraser Valley slope to form the prominence in which the inclined shaft was excavated. This prominence is crossed by several linear troughs which trend along the ground surface contours and are evidence of previous down-slope movement of the prominence. Mature trees growing in these troughs indicate a history of movement of at least several hundred years (Moore, Imrie and Baker, 1991).

The water conduit operated without incident between the initial filling in 1952 and May 1981 when leakage was first noted from the upper access adit located near the intersection of the inclined shaft and the upper tunnel (see Figure 3). This leakage stopped when two drain pipes embedded in the concrete backfill beneath the steel lining were plugged at their upstream ends. Large holes had been eroded in these drainage pipes where they were not encased in concrete and it was concluded that this corrosion was responsible for the leakage. This conclusion appeared to be valid until 25 January, 1989 when a much larger water flow occurred.

Investigations in the dewatered tunnel revealed a 150 mm wide circumferential tension crack in the steel lining of the upper tunnel, about 55 m from its intersection with the inclined shaft. In addition, eight compressional buckle zones were found in the upper portion of the inclined shaft. Subsequent investigations revealed that approximately 20 million cubic metres of rock are involved in down-slope creep which, during 1989-90, amounted to several centimetres per year and which appears to be ongoing. This down-

slope creep appears to be related to a process of block rotation rather than to any deep seated sliding as was the case at both the Downie Slide and Dutchman's Ridge.

While discrete element models may give some indication of the overall mechanics of this type of slope deformation, there is no way in which a factor of safety, equivalent to that for sliding failure, can be calculated. Consequently, in deciding upon the remedial measures to be implemented, other factors have to be taken into consideration.

After thorough study by the BC Hydro and their consultants, it was decided to construct a replacement conduit consisting of an unlined shaft and tunnel section and a steel lined section where the rock cover is insufficient to contain the internal pressure in the tunnel. This replacement conduit, illustrated in Figure 3, will remove the steel lined portions of the system from zones in which large displacements are likely to occur in the future. This in turn will minimise the risk of a rupture of the steel lining which would inject high pressure water into the slope. It was agreed that such high pressure water leakage could be a cause for instability of the overall slope. Further studies are being undertaken to determine whether additional drainage is required in order to provide further safeguards.

Careful measurements of the displacements in the inclined shaft, the length of the steel lining cans as compared with the original specified lengths and the opening of the tensile crack in the upper portion of the steel lined tunnel, provided an overall picture of the displacements in the rock mass. These observed displacements were compared with displacement patterns computed by means of a number of numerical studies using both continuum and discrete element models and the results of these studies were used in deciding upon the location of the replacement conduit.

In addition to the construction of this replacement conduit to re-route the water away from the upper and potentially unstable part of the slope, a comprehensive displacement and water pressure monitoring system has been installed and is being monitored by BC Hydro (Baker, 1991, Tatchell, 1991).

Structural failures in rock masses

In slopes, foundations and shallow underground excavations in hard rock, failure is frequently controlled by the presence of discontinuities such as faults, shear zones, bedding planes and joints. The intersection of these structural features can release blocks or wedges which can fall or slide from the surface of the excavation. Failure of the intact rock is seldom a problem in these cases where deformation and failure are caused by sliding along individual discontinuity surfaces or along lines of intersection of surfaces. Separation of planes and rotation of blocks and wedges can also play a role in the deformation and failure process.

An analysis of the stability of these excavations depends primarily upon a correct interpretation of the structural geological conditions in the rock mass followed by a study

of the blocks and wedges which can be released by the creation of the excavation. Identification and visualisation of these blocks and wedges is by far the most important part of this analysis. Analysis of the stability of the blocks and wedges, and of the reinforcing forces required to stabilize them, is a relatively simple process once this identification has been carried out.

The Río Grande Pumped Storage Project is located in the Province of Córdoba in the Republic of Argentina. Four reversible pump-turbines operating at an average head of 170 m give the project a total installed capacity of 750 MW. These turbines are installed in a 25 m span, 50 m high, 105 m long cavern at an average depth of 160 m .

The rock in which the underground excavations are situated is a massive tonalitic gneiss of excellent quality (Amos et al, 1981). The gneiss has an average uniaxial compressive strength of 140 MPa. The maximum principal stress, determined by overcoring tests, is 9.4 MPa and is almost horizontal and oriented approximately normal to the cavern axis. In massive rocks, this 15:1 ratio of uniaxial strength to maximum principal stress is unlikely to result in any significant failure in the rock and this was confirmed by numerical stress analyses (Moretto, 1982). The principal type of instability which had to be dealt with in the underground excavations was that of potentially unstable blocks and wedges defined by intersecting structural features (Hammett and Hoek, 1981).   In one section of the cavern, the axis of which is oriented in the direction 158-338, four joint sets were mapped and were found to have the following dip/dip direction values:

N.	Dip	Dip dir.	Comments
		Table 5. Dip and dip direction values for joints in one location in the Río Grande cavern	
1	50	131	infrequently occurring joints
2	85	264	shear joint set
3	70	226	shear joint set
4	50	345	tension joint set

Figure 4 is a perspective view of the Río Grande power cavern showing typical wedges which can be formed in the roof, sidewalls, bench and floor by joint sets 2, 3 and 4. These figures represent the maximum possible sizes of wedges which can be formed and, during construction, the sizes of the wedges were scaled down in accordance with average joint trace lengths measured in the excavation faces. In Figure 4 it is evident that the roof and the two sidewall wedges were potentially unstable and that they needed to be stabilised. This stabilisation was achieved by the placement of tensioned and grouted rockbolts which were installed at each stage of the cavern excavation. Decisions on the number, length and capacity of the rockbolts were made by on-site geotechnical staff using limit equilibrium calculations based upon the volume of the wedges defined by the measured trace lengths. For those wedges which involved sliding on one plane or along the line of intersection of two planes, rockbolts were installed across these planes to bring the sliding factor of safety of the wedge up to 1.5. For wedges which were free to fall from the roof, a factor of safety of 2 was used. This factor was calculated as the ratio of the total capacity of the bolts to

the weight of the wedge and was intended to account for uncertainties associated with the bolt installation.

The floor wedge was of no significance while the wedges in the bench at the base of the upstream wall were stabilised by dowels placed in grout-filled vertical holes before excavation of the lower benches.
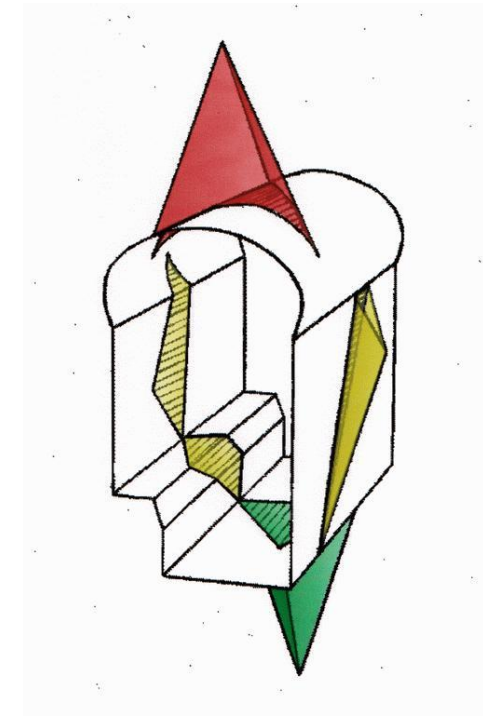

Figure 4: Perspective view of Río Grande power cavern showing potentially unstable wedges in the roof, sidewalls, bench and floor.

Early recognition of the potential instability problems, identification and visualization of the wedges which could be released and the installation of support at each stage of excavation, before the wedge bases were fully exposed, resulted in a very effective stabilisation program. Apart from a minimal amount of mesh and shotcrete applied to areas of intense jointing, no other support was used in the power cavern which has operated without any signs of instability since its completion in 1982.

Excavations in weak rock

In contrast to the structurally controlled failures in strong rock discussed in the previous section, there are many cases where tunnels and caverns are excavated in rock masses which are weak as a result of intense jointing or because the rock material itself has a low strength. Rocks such as shales, mudstones, siltstones, phyllites and tuffs are typical weak rocks in which even moderate in situ stresses are likely to induce failure in the rock surrounding underground excavations.

Progressive failure of this type, which can occur in the rock surrounding an underground excavation in a weak rock mass, is a difficult analytical problem and there are no simple numerical models nor factor of safety calculations which can be used to define acceptable limits to this failure process. Judgement on the adequacy of a support design has to be based upon an evaluation of a number of factors such as the magnitude and distribution of deformations in the rock and the stresses induced in support elements such as grouted cables, steel sets or concrete linings. This design process is illustrated by means of an example.

The Mingtan pumped storage project is located in the central region of the island of Taiwan and utilizes the 400 m head difference between the Sun Moon Lake and the Shuili River to generate up to 1600 MW at times of peak demand. The power cavern is 22 m wide, 46 m high and 158 m long and a parallel transformer hall is 13 m wide, 20 m high and 17 m long. The caverns are 45 m apart and are located at a depth of 30 m below surface in the steep left bank of the Shuili river (Liu, Cheng and Chang, 1988).

The rock mass consists of weathered, interbedded sandstones, siltstones and shales dipping at about 35° to the horizontal. The Rock Mass Ratings (RMR) (Bieniawski, 1974) and Tunnelling Quality Index Q (Barton, Lien and Lunde, 1974) and approximate shear strength values for the various components of the rock mass are given in Table 6 below.

Table 6. Rock mass classifications and approximate friction angles ϕ and cohesive strengths c for the rock mass in which the Mingtan power cavern is excavated

Rock type	RMR	Q	ϕ' degrees	c' MPa
Jointed sandstone	63-75	12-39	50	1.0
Bedded sandstone	56-60	7-31	45	0.8
Faults or shears	10-33	0.1-1.1	30-40	0.15-0.3

Weak beds of siltstone, up to 2 m thick, appear to have caused a concentration of shear movements during tectonic activity so that fault zones have developed parallel to the bedding. The common feature observed for all these faults is the presence of continuous clay filling with a thickness varying from a few mm to 200 mm. The cavern axis is intentionally oriented at right angles to the strike of these faults.

The measured in situ stresses in the rock mass surrounding the cavern are approximately

Maximum principal stress (horizontal) $\sigma_{max} = 10.9$ MPa

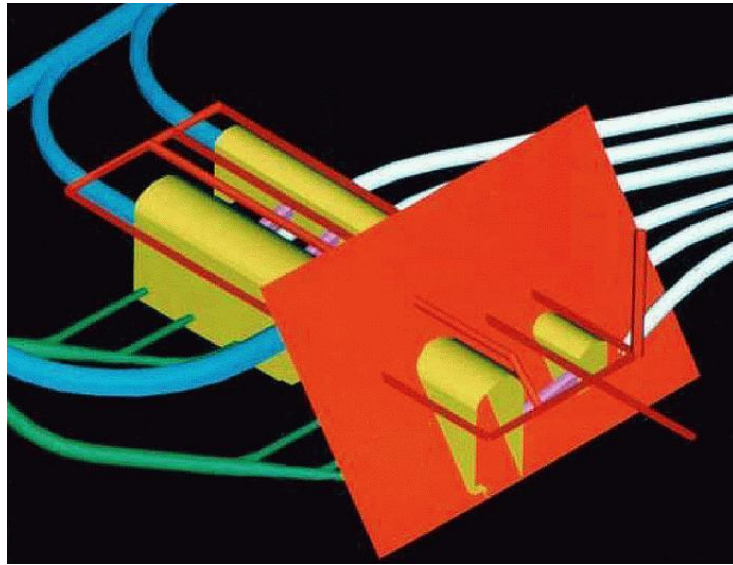Minimum principal stress (vertical) $\sigma_{min} = 7.5$ MPa

Figure 5: Orientation of the underground excavations in relation to the faults in the bedded sandstone surrounding the power cavern and transformer hall of the Mingtan Project. The red plane indicates the dip and strike of the faults.

Bedding faults of significant thickness which were intersected in the roof of the cavern were treated by using high pressure water jets to remove the clay and then filling the cavities with non shrink cementitious mortar (Cheng, 1987, Moy and Hoek, 1989). This was followed by the installation of 50 tonne capacity untensioned grouted cables from a drainage gallery 10 m above the cavern roof in order to create a pre-reinforced rock mass above the cavern. All of this work was carried out from construction adits before the main contract for the cavern excavation commenced.

The initial design of the reinforcing cables was based upon experience and precedent practice. Figures 6 and 7 give the lengths of rockbolts and cables in the roof and sidewalls of some typical large powerhouse caverns in weak rock masses. Plotted on the same graphs are empirical relationships suggested by Barton (1989) for bolt and cable lengths for underground powerhouses.

During benching down in the cavern, 112 tonne capacity tensioned and grouted cables were installed on a 3 m x 3 m grid in the sidewalls. The final layout of the cables in the rock surrounding the power cavern and the transformer hall is illustrated in Figure 8. Five metre long grouted rockbolts were installed as required at the centre of the squares formed by the cable face plates A 50 mm layer of steel fibre reinforced microsilica shotcrete was applied within 5 to 10 m of the face. This shotcrete was later built up to a thickness of 150 mm on the roof and upper sidewalls and 50 mm on the lower sidewalls where it would eventually be incorporated into the concrete foundations.
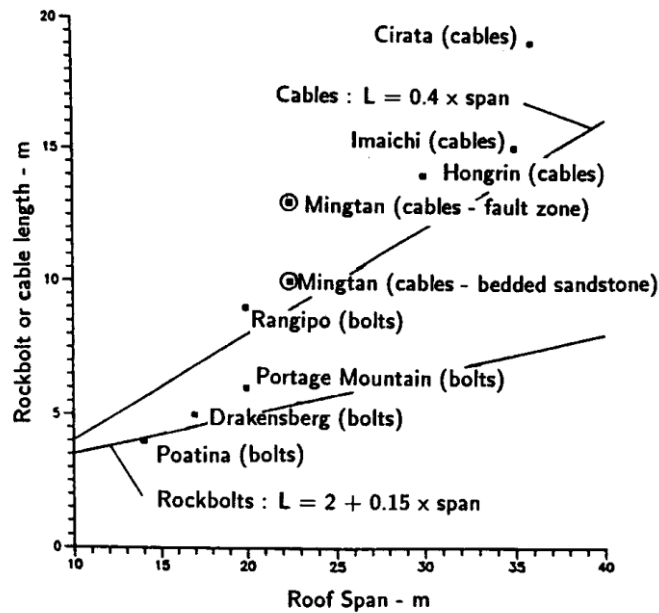
Figure 6: Lengths of rockbolts and cables used for roof support in some large caverns in weak rock. Equations defining trend lines were suggested by Barton (1989).
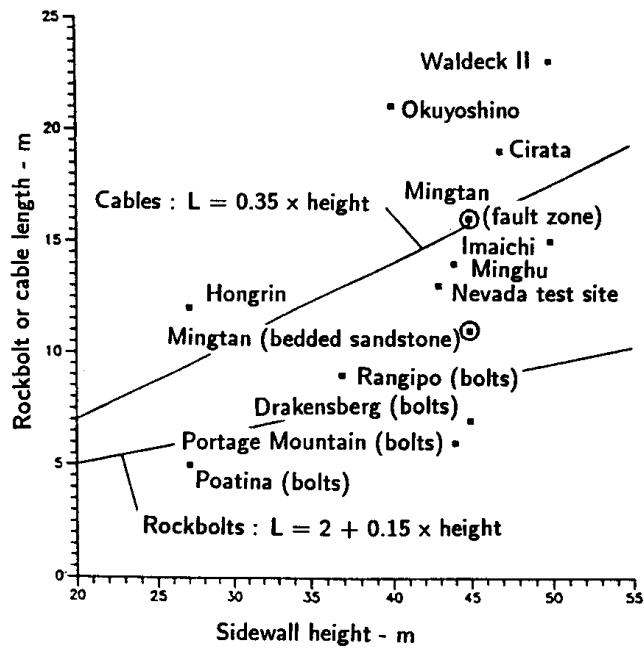

Figure 7: Lengths of rockbolts and cables used for sidewall support in some large caverns in weak rock. Equations defining trend lines were suggested by Barton (1989).

A key element in the decision making process on the adequacy of the support system was a monitoring and analysis process which involved the following steps :

1. Displacements in the rock surrounding the excavations monitored by means of convergence arrays and extensometers, some of which had been installed from construction galleries before excavation of the caverns commenced.

2. Numerical modelling of each excavation stage using non-linear multiple-material models. The material properties used in the models of the early excavation stages were adjusted to obtain the best match between predicted and measured displacements.

3. Prediction of displacements and support loads during future excavation stages and adjustment of support capacity, installation and pre-tensioning to control displacements and cable loads.

4. Measurement of displacements and cable loads (using load cells on selected cables which had been de-bonded) and comparison between measured and predicted displacements and cable loads.

5. Installation of additional cables or adjustment of cable loads to control unusual displacements or support loads.

The aim of this program was to maintain as uniform a displacement pattern around the excavations as possible and to keep the loads on the cables at less than 45% of their yield load. The intermediate rockbolts and the shotcrete were not accounted for in the numerical modelling since it was assumed that their role was confined to supporting the rock immediately adjacent to the excavations and that the overall stability was controlled by the 10 to 15 m long grouted cables.

Figure 8 shows the combination of materials used in analysing one section of the cavern, assuming that the bedding faults could be represented by horizontal layers in the two-dimensional model. In order to match the measured and predicted displacements in the rock mass, it was found that a 2.5 m thick zone of softened and weakened material had to be wrapped around the excavations to account for blast damaged material (achieving good blasting results was difficult in this interbedded rock).

In Figure 9, the predicted and measured displacements along six extensometers installed in the power cavern sidewalls are compared. The overall agreement is considered to be acceptable. Maximum sidewall displacements were of the order of 100 mm at the mid-height of the upstream wall, adjacent to one of the major faults. Elsewhere, displacements were of the order to 25 to 46 mm.

Figure 10 shows the results of monitoring at seven stations along the axis of the power cavern. Before excavation of the cavern commenced, extensometers were installed at each of these stations from a drainage gallery above the roof arch and from construction

galleries as shown in the upper part of Figure 10. In addition, load cells were installed on cables adjacent to some of the extensometers.

Rapid responses were recorded in all extensometers and load cells as the top heading passed underneath them. Further responses occurred as the haunches of the cavern arch were excavated and as the first bench was removed. As can be seen from the plots, after this rapid response to the initial excavation stages, the displacements and cable loads became stable and showed very little tendency to increase with time. The difference in the magnitudes of the displacements and cable loads at different stations can be related to the proximity of the monitoring instruments to faults in the rock above the cavern arch.

The rapid load acceptance and the modest loading of the cables together with the control of the displacements in the rock mass were the goals of the support design. Measurements obtained from the extensometers and cable load cells indicate that these goals have been met.
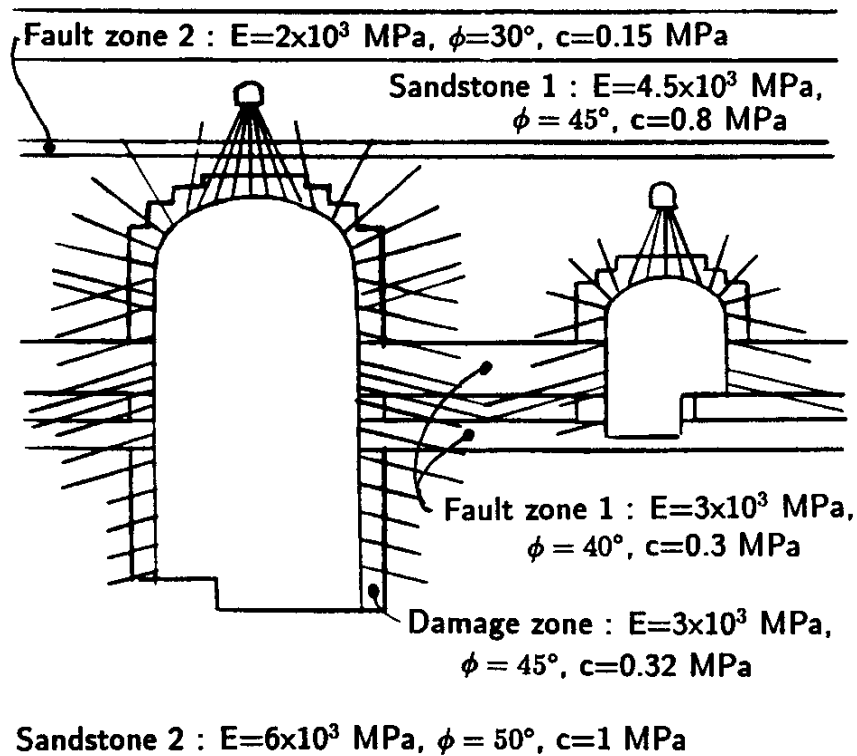
Figure 8: Layout of cables used to support the rock surrounding the power cavern and the transformer hall in the Mingtan pumped storage project. The location and properties of the rock units represent those used in the numerical analysis of failure, deformation and cable loading in a typical vertical section.

Figure 9: Comparison between calculated and measured displacements along six extensometers installed in the sidewalls of the Mingtan power cavern.

Figure 10: Surface displacements and cable loads measured at seven stations along the power cavern axis.

Factor of safety

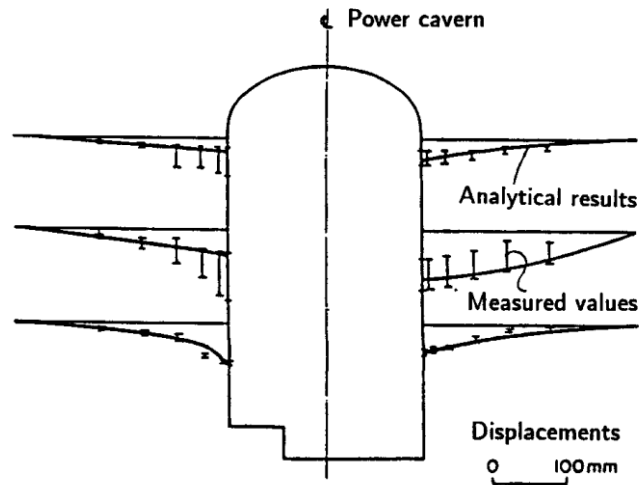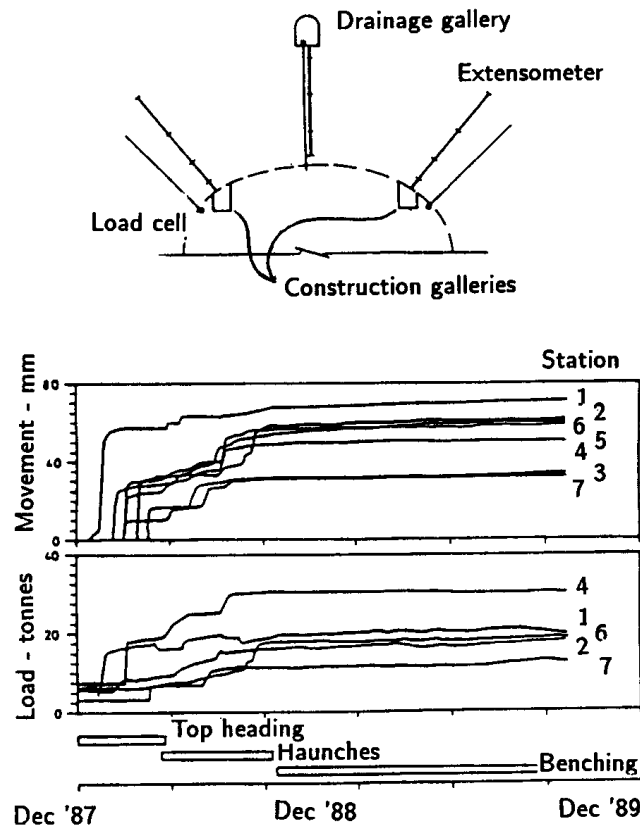The four case histories, discussed in previous sections, have been presented to demonstrate that a variety of criteria have to be considered in deciding upon the adequacy of a rock structure to perform its design objectives. This is true for any design in rock since the performance of each structure will be uniquely dependent upon the particular set of rock conditions, design loads and intended end use.

In one group of structures, traditional designs have been based upon a `factor of safety' against sliding. These structures, which include gravity and fill dams as well as rock and soil slopes, all involve the potential for sliding along well defined failure surfaces. The factor of safety is defined as the factor by which the shear strength parameters may be reduced in order to bring the slope (or dam foundation) into a state of limiting equilibrium (Morgenstern, 1991). The numerical value of the factor of safety chosen for a particular design depends upon the level of confidence which the designer has in the shear strength parameters, the groundwater pressures, the location of the critical failure surface and the magnitude of the external driving forces acting upon the structure.

Figure 11: Hypothetical distribution curves representing the degree of uncertainty associated with information on driving stresses and shear strengths at different stages in the design of a structure such as a dam foundation.

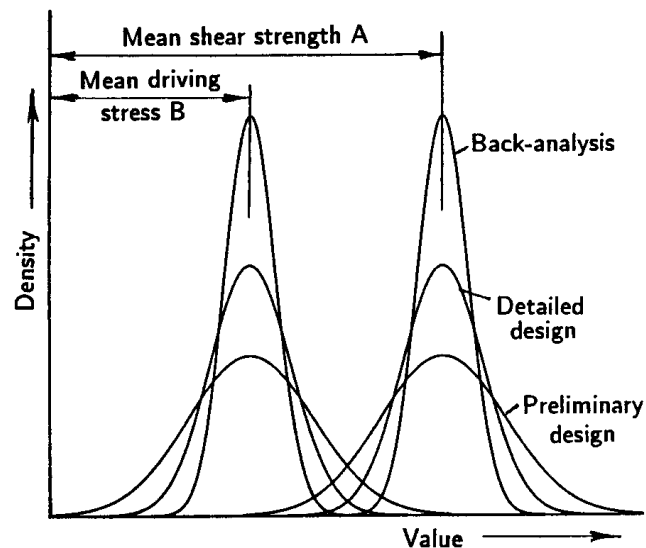Figure 11 illustrates a set of hypothetical distribution curves representing the degree of uncertainty associated with available information on shear strength parameters and disturbing stresses for different stages in the design of a rock or soil structure. The factor of safety is defined as A/B where A is the mean of the distribution of shear strength values

and B is the mean of the distribution of driving stresses. For the purpose of this discussion, the same factor of safety has been assumed for all three cases illustrated.

During preliminary design studies, the amount of information available is usually very limited. Estimates of the shear strength of the rock or soil are generally based upon the judgement of an experienced engineer or geologist which may be supplemented, in some cases, by estimates based upon rock mass classifications or simple index tests. Similarly, the disturbing forces are not known with very much certainty since the location of the critical failure surface will not have been well defined and the magnitude of externally applied loads may not have been established. In the case of dam design, the magnitude of the probable maximum flood, which is usually based upon probabilistic analysis, frequently remains ill defined until very late in the design process.

For this case, the range of both available shear strength and disturbing stresses, which have to be considered, is large. If too low a factor of safety is used, there may be a significant probability of failure, represented by the section where the distribution curves overlap in Figure 11. In order to minimise this failure probability, a high value for the factor of safety is sometimes used. For example, in the 1977 edition of the US Bureau of Reclamation Engineering Monograph on Design Criteria for Concrete Arch and Gravity Dams, a factor of safety of 3.0 is recommended for normal loading conditions when 'only limited information is available on the strength parameters'. This value can be reduced to 2.0 when the strength parameters are 'determined by testing of core samples from a field investigation program or by past experience'.

During detailed design studies, the amount of information available is usually significantly greater than in the preliminary design stage discussed above. A comprehensive program of site investigations and laboratory or in situ shear strength tests will normally have been carried out and the external loads acting on the structure will have been better defined. In addition, studies of the groundwater flow and pressure distributions in the rock mass, together with modifications of these distributions by grouting and drainage, will usually have been carried out. Consequently, the ranges of shear strength and driving stress values, which have to be considered in the design, are smaller and the distribution curves are more tightly constrained.

The case histories of the Downie Slide and Dutchman's Ridge, discussed earlier, are good examples of designs based upon back-analyses. In both of these cases, very extensive site investigations and displacement monitoring had established the location of the critical failure surfaces with a high degree of certainty. Careful monitoring of the groundwater in the slopes (256 piezometer measuring points were installed in Dutchman's Ridge) had defined the water pressures in the slopes and their fluctuations over several years. Some shear testing on fault material recovered from cores was carried out but, more importantly, the mobilized shear strength along the potential failure surfaces was calculated by back-analysis, assuming a factor of safety of 1.00 for existing conditions.

Figure 11 illustrates the hypothetical distribution curves for the range of values for shear strength and driving stresses for the case of a structure in which an existing failure has been carefully back-analyzed. Depending upon the degree of care which has been taken with this back-analysis, these curves will be very tightly constrained and a low factor of safety can be used for the design of the remedial works.

This discussion illustrates the point that different factors of safety may be appropriate for different stages in the design of a rock structure. This difference is primarily dependent upon the level of confidence which the designer has in the values of shear strength to be included in the analysis. Hence, a critical question which arises in all of these cases is the determination or estimation of the shear strength along the potential sliding surface. In a paper on the strength of rockfill materials, Marachi, Chan and Seed (1972) summarize this problem as follows: 'No stability analysis, regardless of how intricate and theoretically exact it may be, can be useful for design if an incorrect estimation of the shearing strength of the construction material has been made'.

Except in simple cases involving homogeneous soils or planar continuous weak seams, determination of the shear strength along potential sliding surfaces is a notoriously difficult problem. This is particularly true of the determination of the cohesive component, c', of the commonly used Mohr-Coulomb failure criterion. Laboratory test specimens tend to be too small to give representative results while in situ tests are difficult and expensive and, unless carried out with very great care, are liable to give unreliable results.

Table 7: Factors of safety for different loading in the design of earth and rockfill dams.

Loading condition	S.F.	Remarks
End of construction porewater pressures in the dam and undissipated porewater pressures in the foundation. No reservoir loading.	1.3	
Reservoir at full supply level with steady state seepage in the dam and undissipated end-of-construction porewater pressures in the foundation.	1.3	Possibly the most critical (even if rare) condition.
Reservoir at full supply level with steady state seepage.	1.5	Critical to design.
Reservoir at probable maximum flood level with steady state seepage conditions.	1.2	
Rapid reservoir drawdown from full supply level to minimum supply level	1.3	Not significant in design. Failures very rare and, if they occur, usually shallow.

For failure surfaces which involve sliding on rough or undulating rock surfaces such as joints or bedding planes, the methodology proposed by Barton (1976) is appropriate for estimating the overall shear strength of the potential sliding surface. This involves adding a measured or estimated roughness component to the basic frictional strength which can be determined on sawn and polished laboratory shear test specimens.

For heavily jointed rock masses in which there are no dominant weakness zones such as faults or shear zones, a crude estimate of the shear strength of the rock mass can be obtained by means of the use of rock mass classification systems as proposed by Hoek and Brown (1988).

In all cases, a greater reliance can be placed upon the frictional component, ϕ, of the Mohr-Coulomb shear strength equation and extreme care has to be taken in the estimation of the cohesive strength, c'. Where no reliable estimates of this value are available from carefully conducted shear tests or from back-analysis of existing failures, it is prudent to assume a cohesive strength of zero for any stability analysis involving structures such as dam foundations.

In the design of fill and gravity dams there is a tendency to move away from the high factors of safety of 2 or 3 which have been used in the past, provided that care is taken in choosing sensible conservative shear strength parameters, particularly for continuous weak seams in the foundations. An example of the range of factors of safety which can be used in the design of earth or rockfill dams is given in Table 7.

Probabilistic analyses

The uncertainty associated with the properties of geotechnical materials and the great care which has to be taken in selecting appropriate values for analyses has prompted several authors to suggest that the traditional deterministic methods of slope stability analyses should be replaced by probabilistic methods (Priest and Brown, 1983, McMahon, 1975, Vanmarcke, 1980, Morriss and Stoter, 1983, Read and Lye, 1983).

One branch of rock mechanics in which probabilistic analyses have been accepted for many years is that of the design of open pit mine slopes. This is because open pit planners are familiar with the concepts of risk analysis applied to ore grade and metal price fluctuations. Probabilistic methods are used in estimating the economic viability of various options in developing an open pit mine and so it is a small step to incorporate the probability of a geotechnical failure into the overall risk assessment of the mine. The mine planner has the choice of reducing the probability of failure by the installation of reinforcement, reducing the angle of the slope or accepting that failure will occur and providing for extra equipment which may be needed to clean up the failure. Since the mine is usually owned and operated by a single company and access to the mine benches is restricted to trained personnel, accepting a risk of failure and dealing with the consequences on a routine basis is a viable option.

On the other hand, the emotional impact of suggesting to the public that there is a finite risk of failure attached to a dam design is such that it is difficult to suggest the replacement of the standard factor of safety design approach with one which explicitly states a probability of failure or a coefficient of reliability. The current perception is that the factor of safety is more meaningful than the probability of failure. Even if this were not so, there is still the problem of deciding what probability of failure is acceptable for a rock structure to which the general public has access.

In spite of these difficulties, there does appear to be a slow but steady trend in society to accept the concepts of risk analysis more readily than has been the case in the past. The geotechnical community has an obligation to take note of these developments and to encourage the teaching and practical use of probabilistic as well as deterministic techniques with the aim of removing the cloak of mystery which surrounds the use of these methods.

Fortunately, there is a compromise solution which is a form of risk analysis used intuitively by most experienced engineers. This is a parametric analysis in which a wide range of possibilities are considered in a conventional deterministic analysis in order to gain a 'feel' for the sensitivity of the design. Hence, the factor of safety for a slope would be calculated for both fully drained and fully saturated groundwater conditions, for a range of friction angles and cohesive strengths covering the full spectrum which could be anticipated for the geological conditions existing on the site, for external forces ranging from zero to the maximum possible for that slope. The availability of user-friendly microcomputer software for most forms of limit equilibrium analysis means that these parametric studies can be carried out quickly and easily for most designs.

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