HARMONIZING ENGINEERING GEOLOGY WITH ROCK ENGINEERING FOR ASSESSING ROCK SLOPE STABILITY: A REVIEW OF CURRENT PRACTICE

Summary:

Progresses in understanding, analysis and control of rock slope movements have been the result of interdisciplinary efforts involving engineering geologists and rock engineers. In addition to rock engineering methodologies, the inputs from engineering geology are absolutely a fundamental to any rock slope design. This paper aims to emphasize the importance of harmonizing engineering geology with rock engineering on stability of natural and engineered rock slopes. The main engineering geological factors featured in the design and construction of rock slopes, role of engineering geological and hydrogeological conceptual models and their combination with the stability analysis methods used in rock slope engineering, input parameter selection, current back-analysis techniques and movement monitoring methods are briefly discussed through some real cases selected from practice and on hypothetical examples.

Key words:

Rock slope stability, engineering geology, rock engineering, back-analysis, movement monitoring, slope design, engineering geological model, hydrogeological conceptual model

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1 Prof. Dr. Reşat Ulusay, Geological Engineer, Hacettepe University, Faculty of Engineering, Department of Geological Engineering, Applied Geology Division, 06800 Beytepe, Ankara, Turkey; resat@hacettepe.edu.tr
1. INTRODUCTION

The construction, design, remediation and maintenance of rock slopes have always been an important area in geo-engineering. Particularly, in the last two decades, increasing demand for ultra-deep open pits reaching up to depths greater than 1000 m and large civil engineering constructions in rocks such as expressways, highways, railways and dams, and the effects of earthquake-triggered landslides on settlements located in mountainous regions resulted in more attention to be paid to rock slope stability. Progresses in understanding, analysis and control of rock slope movements have been the result of interdisciplinary efforts mainly involving engineering geologists and rock engineers. Engineering geology is the discipline of applying geologic data, techniques and principles to the study of rock and soil materials, surface and subsurface fluids, and interactions of introduced materials and processes with the environment so that geologic factors are adequately recognized, interpreted and presented for use in engineering and related practice [1]. The engineering geologist, as a predictor, translates the scientific facts, observed or measured, into engineering data to identify areas of significant physical constraint that will adversely affect the design, construction and maintenance of any intended engineering project [2] (Figure 1).

![Figure 1. Views and perspective of engineering geologist (modified from [2] by [1])]({})

Most of the major practitioners in applied geology in the 19th and 20th centuries have contributed significantly to understanding of slope movement types and processes. Particularly, the experiences gained from the slope failures during the construction of Panama Canal (Figure 2a), and at Malpasset (France) and Vaiont (Italy) (Figure 2b) dams are the important milestones in terms of engineering geology [3]. These cases showed that engineering geological models mainly including effect of discontinuities and identification of accurate failure mode have vital importance in rock slope stability assessments. At the end of 19th and in the first half of 20th centuries, contributions from engineering geology mainly were directed toward developing unified theories of slope formation and evolution [4, 5] and preliminary slope movement classifications [e.g. 6-8]. Then the more detailed slope classification, which is the most popular one in geo-engineering, have been developed by Varnes[9]. Development of Schmidt net [10] and stereographic projection technique [11], and description of discontinuity properties and their quantitative determination [12] are the other contributions provided by engineering geology.

![Figure 2. Views from (a) Culebra slide in the construction of Panama Canal (www.pbs.org), (b) Vaiont dam slide (landslidesblog.org)]({})
In the 1960s, with the beginning of world-wide use of “rock mechanics” as a young science and engineering discipline, existing uncertainties associated with rock slopes have been clarified and some significant developments on theoretical, experimental and numerical aspects for behaviour, analysis and stabilization of rock slopes have been achieved. Method of kinematic analysis of rock slopes and 2-D limit equilibrium methods of analysis and assessment of remedial measures for structurally-controlled rock slope failures [e.g. 13,14] large scale field and laboratory shear testing of rock discontinuities [e.g. 15, 11], the use of computerised test data collection [e.g. 16], development of rock mass classification systems, such as RMR and Q [17-19], and the empirical shear failure criteria for rough-undulating discontinuities [20, 21] and for jointed rock masses [22, 23] are the main contributions of rock mechanics and rock engineering to rock slope engineering. In addition, monitoring the performance of rock slopes had been carried out for many years and had become an integral part of rock slope engineering. More recently, integration of the experimental and theoretical issues in rock mechanics and rock engineering with the computer technology well developed in the second half of 20th century and the use of numerical methods became very popular in rock engineering. Numerical methods have also been one of the major methods applied to rock slopes.

This paper aims to emphasize the importance of harmonizing engineering geology with rock engineering for the assessment of stability of engineered and natural rock slopes. In the paper, the importance of engineering geological model and its harmonization with the main methods of rock slope engineering methods, input parameter selection for stability analyses and back-analysis techniques with their advantages and limitations and importance of movement monitoring are briefly presented and discussed through some real cases selected from practice and on some hypothetical examples.

2. ENGINEERING GEOLOGICAL INPUTS IN ROCK SLOPE STABILITY ASSESSMENTS

The input from engineering geology is a pre-requisite in all stages of rock slope engineering. Failure to take into account engineering geological factors or inadequate inputs or considerations with respect to geological features with a particular slope, can lead to slope failure with accompanying serious consequences. A comprehensive engineering geological model, based on four main factors/inputs such as lithology (rock type), structure (discontinuities), state of degradation and hydrogeological conditions prevailing in rock slopes, is fundamental in rock slope stability assessments. In addition, external forces, such as dynamic loading due to earthquakes are also important for slopes located at earthquake-prone regions. Correct selection of geomechanical parameters of discontinuities and rock masses is the other issue having vital importance on the stability assessments.

2.1. Lithology

Due to different nature and origin of the rock types, their inherent geological features are also different. In softer or weaker rocks such as schist and shale, the material itself can be predominant controlling factor in the slope instability, whereas in the hard rocks such as granites and limestones, the major discontinuities control the stability. In addition, limestones have solution features, which are called karst features originated from solution along discontinuities, may also contribute to trigger failures, particularly in sub-vertical or overhanging cliff slopes. On the other hand, another important rock slope instability problem associated with rock type is commonly experienced in block-in-matrix rocks (BIMROCKS), which are mixture of rocks composed of geotechnically significant blocks within bonded matrix of finer texture such as melanges, fault rocks and other complex geological mixtures [24]. Bimrocks may be sometimes mischaracterized due to their considerable spatial, lithological and mechanical variability, and therefore, their correct characterization is necessary to reduce expansive and inconvenient surprises during slope construction.

2.2. Discontinuities

The most important factor controlling the stability of slopes in jointed rock masses is discontinuities such as bedding, fault, joint, schistosity surfaces etc. and the adverse interaction of their orientations with those of slopes is the greatest contributing factor to rock slope instability. Therefore, the success of rock slope stability assessments depends on the level of understanding of the characteristics of discontinuities, such as orientation, spacing, persistence, roughness, infilling etc., as described by ISRM [25, 26]. Today geological and geotechnical data collection techniques are well developed. However, good quality data collection from discontinuities and the efficient use of the data are important. From practical point of view, it is the best to measure the most of the characteristics of critical geologic structures from surface exposures using different techniques such as scan-line survey, window mapping, photogrammetric method or laser scanning technique (e.g. [26, 27]). However, in cases where there is usually limited
surface area that is exposed and accessible for surface structure, orientation of discontinuities should be determined from orientated cores and/or telewiever logs in conjunction, if possible.

Depending on the control of discontinuities on the slope, two main types of rock slope failure may occur. One of them is the kinematically possible structurally controlled failures occurring along discontinuities or line of intersection of two discontinuities or along columnar structures adversely dipping to the slope face (Figure 3a-c). The second type occurs when the rock mass involves closely spaced discontinuities, which fracture intact rock over small distances, and linear failure planes approaching a circular surface along the whole failure surface, as observed in soil slopes, develop partly through intact rock and partly following discontinuities, resulting in non-structurally controlled rock mass failure (Figure 3d).

Limited or unrealistic assessment of discontinuity characteristics, particularly of orientation, spacing and persistence may result in misconceptions on the modes of failure and on block sizes used in analyses generally different than that of expected in reality, and consequently unrealistic engineering geological models be established for the rock slopes. Depending on increase in slope height, rock slope failures may involve several composite mechanisms. As illustrated in Figure 4, sliding may occur along relatively short adversely dipping discontinuities accompanied by dilation of the mass as the failure path steps up on other sub-vertical discontinuity sets within the rock mass and by shearing through and/or tensile failure of the intact rock and/or rock mass bridges between the discontinuities.

Figure 3. Structurally-controlled rock slope failures: (a) planar failure, (b) wedge failure, (c) toppling, and (d) non-structurally rock mass (circular) failure (Photos: R. Ulusay)

Figure 4. Composite failure path through a rock slope (modified from [28]).
2.3. Effect of Degradation

It is well known that rocks may undergo degradation when they are exposed to atmospheric conditions and/or hydrothermal fluids through rock mass. The degradation process is commonly known as deterioration in the forms of weathering or alteration depending upon the physical and chemical processes involved. Due to weathering hard rocks transform into soft rocks which maintain the structure of the intact rocks, but are characterized by higher void ratios and reduced bond strengths. Soft rocks are transformed into thick granular soil mantles generally called residual soils. The degradation process also influences the joint spacing and discontinuity filling material in the form of clay.

Significant progressive deterioration of rock slopes may occur in engineering time, often giving rise to the need for unplanned maintenance and constituting safety hazard. Superimposed on the lithology and structures, the physical and chemical weathering effects can be predominant in controlling the modes of rock slope failure. For example, open sheet/exfoliation joints, which are formed by stress relief as a result of physical weathering of granitic rocks, tend to run sub-parallel to the rock head with a tendency to daylight in the cut slopes and may result in a structurally controlled instability such as planar failure (Figure 5a). On the contrary to this, if the slope forming rock mass has fully transformed into a soil due to intense chemical weathering (weathering grades V or VI), the slope will fail in the form of circular sliding as commonly observed in soil slopes (Figure 5b).

Another type of weathering, which is a common source of rock slope instability particularly along steep slopes, is differential weathering, which generally occurs in cases where slopes consist of inter-layered hard (e.g. sandstones, limestones) and soft rock units (e.g. shales, claystones and mudstones or tuffs), and may not be recognized sometimes. As differential weathering of softer rocks (highly prone to weathering) progresses, undercutting can lead to loss of support and collapse (falls or topples) of the overlying material (Figure 5c). Large-scale, sub-vertical discontinuities and over-break fractures may promote failure by intersecting undercut surfaces and/or reducing block size within the rock mass.

Deterioration, however, is rarely given much attention at the design stage; emphasis is on the avoidance of deep-seated failures as seen in Figure 5d. But the above given examples emphasize the importance of type, degree, product and depth of the deterioration, which are the main inputs provided by engineering geology, on the mode of slope failure anticipated and on drastic changes in strength and deformability characteristics of the slope forming rock mass.

![Figure 5](image_url)
2.4. Hydrogeological Conditions and Conceptual Model

Water pressure in the discontinuities in a rock mass reduces the effective stresses on such discontinuities with a consequent reduction in shear strength. Therefore, the presence of groundwater in a rock slope is an important factor affecting its stability. The procedure to define an engineering geological modelling involves the definition of a conceptual hydrogeological model that must be very close to the reality and reproduce groundwater flow patterns (fitting computed and measured groundwater heads) with consistent hydraulic parameters.

In a hydrogeological conceptual model, three major phases to be accomplished can be defined: a baseline study as a first phase, conceptualization and characterization phase and prediction as the final phase. Following the baseline hydrogeological study, performed to assess the occurrence and potential of groundwater at and around the slope site, a detailed study program is developed and achieved to construct a hydrogeological conceptual model of the site. The conceptual model construction is essentially based on delineation and characterization of the hydrostratigraphic units in the area and acquisition and analysis of hydrometeorological data. This phase requires extensive field work such as surface and subsurface mapping, drilling, in-situ groundwater level and quality observations and field tests. Upon development a conceptual hydrogeological model of the site, the third main phase is accomplished to predict the spatio-temporal groundwater head and flow rate distribution. This enables to assess not only the type and extent of the threat of groundwater on excavation activities in terms of inflow to excavation and slope-stability but also the impacts of excavation on groundwater resources at and around the site. A flowchart of hydrogeological studies integrated to rock slopes is given in Figure 6 as an example.

![Flowchart of hydrogeological conceptual model](image)

2.5. Effect of Earthquakes on Rock Slope Stability

In natural and engineered slopes, earthquake motions can generate significant horizontal and vertical dynamic forces and increase the shear stress on potential failure surfaces. Unlike gravity and other static forces, earthquake forces are not likely to influence a slope during its design lifetime. However, depending on the characteristics of the slopes and strong ground motion, slope instabilities in different forms may occur. The first step in assessing the influence of seismic forces on slopes is to obtain geologic inputs including the prehistoric seismicity, anticipated magnitude based on the active faults in the vicinity and seismic hazard analysis and choosing suitable attenuation function for rock to estimate the peak ground acceleration to be used in slope stability analyses.
2.6. The Role of Engineering Geological Model

The engineering geological model of a rock slope is comprehensive expression of the various factors which affect the slope stability, and in general, includes the following principal contents: (i) the basic geologic conditions of the slope, (ii) mechanical properties of rock mass and discontinuities, (iii) principal artificial and natural dynamic factors affecting the stability (groundwater, earthquake etc.), (iv) the developing process and characteristics of the rock mass deformation on the slope, and (v) the failure pattern of the slope. By combining geomorphic evidence with the above given conditions based on detailed assessments both in field and laboratory, engineering geological model of the slope focusing the attention on primary and secondary failure modes and triggering mechanisms should be established. Real cases and hypothetical examples showing the importance of engineering geological model in rock slope assessments are briefly discussed in the following paragraphs.

The first example is from a lignite open pit[31, 32]. Engineering geological conditions in this pit with movement monitoring data are illustrated in Figure 7a. In this figure, directions and magnitudes of the movement vectors are generally consistent with the bedding in the overburden and Fault 6. Shear strength of bedding in the overburden was smaller than that of the clay underlying the coal seam. All these engineering geological information indicated that the most possible critical mode of failure should be multi-planar. A quick overburden stripping between points A and B at the toe of the slope (Figure 7a) resulted in shortening of the length of the bedding oppositely dipping to the slope and caused a movement. The uppermost benches in the alluvial soils also moved downward to fill the gap resulted from the movement of the lower benches. This movement confirmed the engineering geological model.

The second example is a simple hypothetical rock slope with the conditions illustrated in Figure 7b. In the construction of engineering geological model for this slope, ignoring both the weak tuff layer and the fault, which may cause a bi-planar failure affecting the entire slope, the mode of failure may be anticipated to occur only throughout the highly weathered portion of the volcanic rock in the form of a circular sliding surface. Contrary to this, if the degree, depth, extent and nature of the weathering in the slope are not well identified, one of the two possible failures (circular failure) can also be missed.

The most important requirement for correct characterization is for investigators to recognize bimrocks and not approach them as stratigraphically orderly (layer-cake) rocks. Once the words “interlayered” or “interbedded” are used in boring logs, there is a tendency to imagine continuous “layer-cake” stratigraphic contacts between borings[33]. Otherwise, misinterpretation of the geologic data by non-geologists may result in unexpected costs and construction difficulties. Upper sketch in Figure 8a shows interpretation of geology based intersections by borings (terminated about 2 m depth) of an assumed continuous and homogeneous sandstone bedrock surface in the Franciscan melange (Bimrock) in US[34]. This model called that the slope instability was shallow, being composed of clay and boulder
colluviums sliding along the contact with the underlying sandstone bedrock and it was decided to remove the shallow failed material. But during excavation, the bedrock couldn’t be found and it was recognized that the instability was actually deep-seated in sheared melange, rather than the shallow soil mass. Bedrock was the result of the misconception by connecting straight lines (the red boundary in Figure 8a) between the interpreted soil/rock contacts as intersected by borings. The excavation was deepened below the design depth of a few meters to tens of meters. The repair finally cost more than a million dollars. Lower sketch in Figure 8b shows a more realistic model to assess bedrock conditions, in which deeper borings intersect discrete blocks in a bimrock containing blocks within a matrix.

In terms of engineering geological model, the other conditions, regarding the orientation and distribution of the blocks, that should be considered when analysing slope stability problems in bimrocks are illustrated in Figure 9. When the bimrock includes a few blocks, it can be analysed as conventional soil (Figure 9a); failure surface is influenced by the orientation and nature of matrix shearing and it cannot be readily analysed as soil or rock (Figure 9b); blocks are oriented at high angles to the slope, which increases stability due to the increased tortuosity at the failure surface forced around the blocks (Figure 9c); and block-poor zones within generally block-rich bimrock are weaker and more likely to fail (Figure 9d). Medley and Sanz Rehermann[35] developed a simple model to investigate the slope stability of idealized bimrocks. They compared their findings to those of Irfan and Tang[36], who performed stability analyses of theoretical slopes modeled bouldery soil, and found that there is a good relationship between the normalized factors of safety and the volumetric block proportions, despite the significant differences in the model geometries, orientation of blocks, geology of the modelled materials, and analytical methods used. Although considerably more analyses should be performed to define the statistical variations, it appears that up to about 25% to 30% volumetric block proportion, the presence of blocks provides relatively little geomechanical advantage. From this lower limit to greater than 55%, there is marked increase in slope stability. However, more future studies are necessary, as recommended by Medley and Sanz Rehermann[35], perhaps by performing Monte-Carlo type simulations using 3-D geomechanical models, to understand the statistical viability of using simple analytical approaches for complex geological conditions.

Figure 8. (a) Interpretation of geology based intersections by borings of an assumed bedrock surface, (b) more realistic bedrock conditions in which borings intersect discrete blocks in bimrock containing blocks within a matrix[34]

Figure 9. Influence of the orientation and distribution of the blocks on stability of slopes in Bimrocks[35]
3. SELECTION OF STRENGTH PARAMETERS

Reliable estimates of mechanical properties of discontinuities and rock masses are required for almost any analysis used to design rock slopes, and contributions to their selection both from engineering geology and rock mechanics are necessary. The evaluation of engineering properties, which are the quantities directly measured or empirically estimated, should consider the identified failure modes and mechanisms.

Shear strength is the main parameter for rock slope stability analysis and can be described by either linear or non-linear failure criterion. This is also valid for a differentiation between the peak and residual strength. Based on the scale effect, which is an important issue in rock slopes [28], as illustrated in Figure 10a, and engineering geological conditions discussed in the previous sections, it is necessary to use the shear strength properties of either the discontinuities or of the rock mass. In the analysis of structurally controlled rock slope failures, the general trend is to use the Mohr-Coulomb and empirical Barton’s failure criteria for smooth and rough discontinuities, respectively. If a discontinuity is filled by a soft/weak material, its shear strength is governed by the strength of the filling (Figure 10b).

Because it is extremely difficult to measure rock mass strength directly using full scale field tests due to their high costs and practical difficulties, back-analysis of previous failures may be used for its estimation, however, it should be kept in mind that it has also some limitations. As an alternative, strength of rock masses usually is estimated from empirical relations and/or rock mass classification systems. However, there are some legacies inherited from these systems developed for other purposes than rock slope stability as discussed by Hack [37], in detail. One of the weaknesses of these systems includes the fact that they are not based on mechanics and that they combine all characteristics into a single number. Since the 1980’s the empirical Hoek-Brown failure criterion [22, 23], which is based on rock mass rating and then Geological Strength Index (GSI; [38]), is being commonly used for estimating shear strength of rock masses. Since the Hoek-Brown criterion represents a curved failure envelope, a transition to the linear Coulomb criterion is often conducted. This is because a linear failure envelope is easier to handle both analytical and numerical methods. However, by considering that the failure envelope of a Jointed rock mass in reality is curved, no transition to a linear envelope should be preferred. In addition, the empirical relationships based on RMR [39] and Q values [40], are also the other alternatives to be used for the same purpose. A most recently developed new rock mass rating system, RMQR, is used to estimate the geomechanical properties of rock masses [41]. As an example on the selection of shear strength parameters, the complex failure surface illustrated in Figure 4 can be considered. In this figure, Zone A, the upper part of the failure surface consisting of a fault, is represented by Mohr-Coulomb criterion; for Zone B, heavily jointed central part, empirical rock mass failure criterion can be used; and for Zone C, depending on surface characteristics of the step-path discontinuities, Mohr-Coulomb or Barton’s criterion is preferred.

Determination of strength of bimrocks is one of the difficult issues in rock engineering. Mechanical properties of the matrix, the volumetric block proportion, shape and size distribution of blocks and their orientation relative to failure surfaces are the main factors affecting their overall mechanical properties. Neglecting the contributions of blocks to overall bimrock strength, choosing instead to design on the basis of the strength of the weak matrix may result in too conservative for many bimrocks in terms of slope design [34]. Based on the study on a physical model melange by Lindquist [42], when the block proportions are between about 25% and 70%, the increase in the overall mechanical properties of bimrocks are mainly related to the volumetric proportion of the blocks in the rock mass (Figure 10c). Although some efforts have been performed to assess the strength of bimrocks based on physical models (e.g. [43-45]), in-situ tests (e.g. [46]) and equivalent material techniques (e.g. [47]), further studies to develop more efficient methods and a database for bimrock strength are still needed.
4. CRITICAL REVIEW ON THE METHODS OF ROCK SLOPE STABILITY ASSESSMENT

4.1. Conventional and Numerical Methods

The rock slope stability analysis may be mainly divided into two groups of methods: (i) the conventional methods (kinematic analyses based on stereographic projection technique, 2D limit equilibrium methods (LEM) and rock fall simulations), and (ii) numerical methods. The main inputs, advantages and limitations of the conventional and numerical methods, which have been discussed in literature in detail (e.g.[50-52]), are summarized in Table 1.

The method of kinematic analysis only helps engineers to recognize potential structurally controlled rock slope instabilities, such as planar, wedge and toppling failures and is more relevant to low height rock slopes. The LEM is used to determine factor of safety, which is an indicator of the stability of slopes. LEMs are highly relevant to simple block failures along discontinuities or rock slopes that are behaving like a soil due to their heavily fractured or weathered nature. Due to the common acceptance of the safety factor approach as the main criterion of slope stability, LEM have been used more often and seems still to remain the most common adopted method. In the case of rockfalls, it is generally impossible to secure all blocks, and therefore, consideration is given to the design of protective measures around structures endangered by the falling blocks. Rockfall protection works, therefore, largely involves the determination of travel paths and trajectories of unstable blocks that have detached from a rock slope face [50]. For the purpose, 2D and 3D computer-based programs, called “rockfall simulators”, analyse the trajectory of falling blocks and also help to determine remedial measures. However, 3D simulations are more reliable and realistic than 2D simulations in case of complex morphology and then trajectories not parallel to each other along the slope. In addition, unexpected bounces, due to local obstacles and not intersected by 2D sections, can be seen only with 3D simulations.
<table>
<thead>
<tr>
<th>Method</th>
<th>Input parameters</th>
<th>Advantages</th>
<th>Limitations</th>
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<tbody>
<tr>
<td>Kinematic analysis</td>
<td>Critical orientation of slope and discontinuity, friction angle.</td>
<td>Simple to use and shows structurally-controlled failure potential.</td>
<td>Suitable only for preliminary assessments. Identification of critical discontinuities requires engineering judgment. Ignores slope geometry, cohesion, groundwater and external loading conditions. No Factor of Safety (FS) is calculated.</td>
</tr>
<tr>
<td>LEM</td>
<td>Slope geometry, shear strength of material /discontinuity/rock mass, groundwater and external loading conditions.</td>
<td>Easy to use. Software available for different failure modes with multiple materials. Mostly deterministic, but be used for probabilistic analysis. Calculates FS in short time and suitable for sensitivity analysis.</td>
<td>In-situ stress, strains and intact material failure not considered. Pre-defined slip surface needed. Probabilistic analysis requires well-defined input data.</td>
</tr>
<tr>
<td>Rockfall simulation</td>
<td>Slope geometry and surface condition. Block sizes, shapes, unit weights and coefficients of restitution.</td>
<td>Practical for protective measures. Can utilize probabilistic analysis. 2-D and 3-D codes are available.</td>
<td>Limited experience in use relative to empirical design charts.</td>
</tr>
<tr>
<td>Continuum modelling (e.g. FEM, FDM)</td>
<td>Slope geometry, constitutive criteria, groundwater conditions, shear strength, in-situ stress state.</td>
<td>Without statical assumptions, can model complex behavior and mechanism and slip surfaces of any shape in 2-D and 3-D with coupled modeling of groundwater. Incorporate creep deformation and dynamic analysis.</td>
<td>Not easy to use and require well-trained and experienced users. Some inputs are not routinely measured. Software requires longer run times when compared to LEM.</td>
</tr>
<tr>
<td>Discontinuum modelling (e.g. DEM, DDA)</td>
<td>Geometry of slope and discontinuity, intact constitutive criteria, discontinuity stiffness and strength, groundwater and stress.</td>
<td>Allow for block deformation and movement of blocks relative to each other and model complex behavior and mechanisms. Assess effects of parameter variation on instability.</td>
<td>Not easy to use and require well-trained and experienced users. Limited data on discontinuity properties available and need to simulate representative discontinuity geometry.</td>
</tr>
<tr>
<td>Hybrid models (e.g. ELFEN)</td>
<td>Combination of continuum and discontinuum modeling, damping factors, stiffness, shear strength, fracture energy</td>
<td>Model the transition from continuous to discontinuous behavior considering fracture and fragmentation processes. Combine advantages of both discontinuum and continuum modeling. Allow dynamic analysis using constitutive models.</td>
<td>More memory required for complex models. Comparatively little practical experience in use. Yet to be coupled with groundwater.</td>
</tr>
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This is because at such scales stability is affected by the strength and deformation properties of both intact rock and discontinuities, the geometry and distribution of discontinuities throughout the rock mass and complexities related to material anisotropy, non-linear behaviour, in-situ stresses and the presence of coupled processes such as pressures, seismic loading etc. These types of failure cannot be modelled by means of conventional methods of analysis. The numerical modelling techniques have been developed to generate a range of possible solutions for such rock slope stability problems. Numerical methods can be considered in three groups, namely continuum modelling, discontinuum modelling and hybrid modelling. Continuum modelling is applicable to rock slopes composed of intact rock, weak rocks and heavily jointed rocks and include the finite element (FEM) and finite difference (FDM) methods. Discontinuum modelling is suitable for rock slopes of which stability is controlled by discontinuity behaviour, and is referred to as discrete element method (DEM). Most recently hybrid approaches, including coupling of continuum and discontinuum approaches, which often fail to realistically simulate the progressive failure of rock slopes, particularly the dynamics of kinetic release accompanying complex internal distortion, dilation and fracture [51]), are increasingly being adopted rock slope stability analysis as an advanced numerical modelling (Figure 11).
4.2. Methods for Earthquake-Induced Slope Failures

The use of empirical bounds by Keefer [53] and an equation proposed by Aydan [54] for the maximum distance of disrupted and coherent earthquake-triggered slope instabilities are the approaches for preliminary assessments. Three basic inertial methods: (i) pseudo-static analysis, (ii)Newmark’s displacement method [55], and (iii) dynamic analysis, for assessing the earthquake-induced rock slope failures are available in the literature.

The greatest difficulty with the first method, based on a static LEM analysis treating earthquake motion as a static force, involves conservative results, the selection of an appropriate seismic coefficient and the value of an acceptable safety factor and no information on the magnitude of displacement. Newmark’s method is the extension of the simple pseudo-static method by directly considering the accelerogram of the sliding mass within the slope to determine permanent displacements. However, it involves many assumptions and limitations [56, 57]. Dynamic analysis typically incorporates a FDM (e.g. FLAC3D code; [58]), DEM approximation (e.g. 3DEC code; [58]) or a FEM approximation, which compute slope stresses and strains using earthquake accelerograms as input and obtain permanent deformation of the slope. The estimation of travel distance of natural slopes upon dynamic failure is also of great importance. Although this issue is well known and some methods are available, the present numerical methods are still insufficient to model earthquake-induced post-failure motions [59].

4.3. Back-Analysis Approach

By considering that the best laboratory is the nature itself, back-analysis of previous slope failures is an attractive method to estimate operative shear strength along the sliding surface at the time of failure. For back-analysis, the failure mode should be well established and accurate information on the failure geometry, groundwater conditions and other factors, which are considered to have contributed to the failure, should be available. Often LEM is used to back-calculate the strength, assuming safety factor is equal to unity. In practice, LEM is used to estimate cohesion (c) and friction angle (φ) by assuming one of these parameters and back-calculating the other for a safety factor of unity. The most efficient solution, in the back-analysis of failure surfaces of which shear strength is expressed by Mohr-Coulomb linear failure criterion, is to analyse more than one cross-section obtained from the instability and then to estimate the operative shear strength parameters from the intersection of the c-φ-curves satisfying a factor of safety of unity, called multiple solution, or to compare these intersections with the range of peak and residual strengths of the sliding surface experimentally determined in laboratory or field, if available[60, 61] (Figure 12a).

In the case of failures along rough discontinuities or through closely jointed rock masses, which obey to non-linear failure criteria (i.e. different c and φ values are operative along the sliding surface depending on normal stress), the above mentioned approach is not valid. As shown in Figure 12b-c, in such cases, shear stresses estimated from different points selected along the sliding surface under different normal stresses depending on the suitable failure criterion (Hoek-Brown criterion for the heavily jointed rock mass in Figure 12b and Barton’s criterion for the rough discontinuity surfaces in Fig. 12c) are plotted onto the failure envelope of the sliding surface and if there is a good match between the failure envelope and shear stress plots, it can be concluded that the failure geometry analysed...
represents the actual failure and the failure criterion used is correctly chosen [62, 63]. Descamps and Yankey[64], who discussed the limitations in the back-analysis in detail, indicate that back-analysis is reliable only when the model and all assumptions are reasonable and accurate representations of the real system. Therefore, in practice, both LEM and numerical modelling tools, and if possible, movement monitoring data should be used together to generate a more accurate solution in back-analysis of the failed rock slopes.

Figure 12. Back-analysis approaches using LEM for (a) planar sliding surfaces based on multiple solution [61], (b) failure in a heavily jointed rock mass [62], (c) planar failure along rough-undulating surface [63].

4.4. Role of Movement Monitoring

Slope failures, such as occurred in the Vaiont Dam (Figure 2b), result in loss of life and damage to or destruction of engineering structures. In open pit mines, where access is generally restricted, slope failures may cause loss of life or not, but they interrupt the mining operation (Figure 5d) and cause damage to property. Therefore, threats due to gradual deformations in slopes should be well managed and for this purpose slope monitoring is necessary. Slope monitoring, which forms an integral part of slope management, is mainly conducted to detect potential unstable ground, to assess the performance of slope design and to apply suitable remedial measures. If this procedure is properly carried out, the risk can be eliminated or reduced. For long years slope monitoring were practiced through visual observations including manual inspection and mapping of tension cracks along the slope face. Although these methods are useful, they are time consuming and have limited accuracy. In addition, some basic instrumentations such as borehole extensometer, inclinometer etc., which are installed in boreholes, are commonly used as subsurface monitoring techniques for slopes particularly in civil engineering practice and it seems that their use will also keep their popularity in the future.

In the beginning of the 21st century, the depth of open pits considerably increased reaching up more than 1000 m and this progress resulted in an increasing frequency of large pit slope failures. Advance warning of these slope instability problems became more important, and therefore, importance of slope monitoring in open pit mining increased. This situation necessitated the development of various new slope monitoring systems, which allow continuous monitoring for routine inspection of the rock and their deformation. The recent technologies used to monitor slopes are Automated Total Station Networks, Light Detection and Ranging (LIDAR) scanning, Slope Stability Radar (SSR), Global Positioning System (GPS), Time Domain Reflectometry (TDR), high resolution micro-seismic...
monitoring. Particularly in the last decade, active monitoring of pit slopes with radar, which is suited for predicting movements on larger surface area, become a standard practice. In the recent years, microseismic monitoring technique also gained considerable attention for early forecast of slope failures and in estimating deformations with considerable accuracy. From the literature it appears that although the microseismic monitoring method in the monitoring of natural and cut slopes for civil excavations is not regarded as having reach maturity [64], while its popularity and use in the monitoring of pit slopes are increasing since 2002 [65]. Researches over the last two decades has led to development of a system for slope monitoring based on the concept of measuring Acoustic Emission (AE). AE rates generated by active waveguides are proportional to the velocity of slope movement, and can therefore, be used to detect changes in rates of movement in response to destabilizing and stabilizing effects, such as rainfall and remediation, respectively (e.g. [66, 67]). However, for its more common use further studies seem necessary.

5. CONCLUSIONS

Due to more complex nature of rock masses when compared to soils, rock slope engineering is a complex field that requires rock engineers and engineering geologists to work together for developing a site specific model. Engineering geology provides direct inputs to rock engineering design approaches and harmonization of an accurate engineering geological model, which includes geology, structure, degree of deterioration, groundwater conditions and triggering static and dynamic factors, with material behaviour and rock engineering methodologies essential for rock slope stability assessments.

A number of studies are mainly concentrated on small and moderate size slope instabilities. Depending on increase in the depth of slopes, due to complex failure mechanism in which the slip surface of composite shape can be formed, the current trend is integrating conventional and numerical methods together in conjunction with the data from conventional and advanced movement monitoring techniques. It is also noted that particularly experiences on failure mechanisms associated with large-scale slopes in hard, brittle and jointed rock masses are limited and new approaches for the estimation of the strength of large-scale rock masses are needed. Compilation of rock mass strength data from the back-analysis results particularly of large scale slope failures may form a very useful database for the assessment of slopes which have not subjected to any failure. Development of more powerful numerical methods which can better model the propagation of failure along discontinuities and through intact rock is one of the expectations. Further studies are also necessary for better understanding of characterization, strength and mechanical behaviour of bimrocksand faulted-fractured zones in the assessment of rock slope stability. In addition, the consideration of failures of natural rock slopes has received very little attention in earthquake engineering, and therefore, much attention should be given to possibility of such slope failures and to assessment methodologies in terms of rock engineering.

6. REFERENCES


[63] Tanyas H, Ulusay R: Assessment of structurally-controlled slope failure mechanisms and remedial design considerations at a feldspar open pit mine, Western Turkey, Engineering Geology, 155, 2013, 54-68.


