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# ROCK SLOPES PROCESSES AND RECOMMENDED METHODS FOR ANALYSIS

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**ABSTRACT:** The stability of rock slopes is of great concern in many engineering projects, for example; road cuts, foundations, retaining walls, and dam excavation. Generally, rock slopes susceptible to instability could be divided into two main categories, the structurally controlled slopes, and the complex rock slopes. In the later, rock slope instability would involve intact material fracturing to allow for rupture surface to be formed. Kinematics of a landslide or the type of movement or instability associated with the landsides is one of the principal criteria for classifying a landslide. In this paper, the modes of instabilities that have been observed in the field are discussed and examples are given. The most widely used methods of analysis of rock slopes such as, limit equilibrium, finite element, discrete element; boundary elements are discussed to highlight their advantages and disadvantages. In this paper, the classification and the methods of analysis are discussed to provide researches as well as engineers with the tools required to analyze and design rock slopes. In complex rock slopes, in civil or mining engineering, hybrid numerical methods must be adapted to better understand the behavior of rock slope. In the case of simple sliding slopes, Limit Equilibrium Techniques can be easily utilized.

*Keywords: Slope Stability, Processes, Rock, Numerical, Hybrid, Methods*

## 1. INTRODUCTION

Geomaterial slopes are susceptible to movement due natural or man-made causes, earthquakes, rain, temperature are among those causes. To analyze or design a rock slope prone to movement; engineers need to understand: the cause and effect of the slope movement, types of slopes movement, and the available methods and techniques to analyze a slope.

Rock slopes susceptible to instability could be divided into two main categories, the structurally controlled slopes, and the complex rock slopes. In the later, rock slope instability would involve intact material fracturing to allow for rupture surface to be formed. Kinematics of a landslide or the type of movement or instability associated with the landsides is one of the principal criteria for classifying a landslide. The height of the slope also plays a role in the mode of failure; as the height increases the complex failures may occur.

In this paper, the modes of instabilities that have been observed in the field are discussed and examples are given. The factors affecting rock slope movement can attributed to many causes, and in rare cases movement of a slope can be attributed to one single reason. The slope movement process involves series of events from cause to effect; these causes and effects are discussed shortly in this paper to further our understanding of this issue. Different modes of failures requires different solving approaches to be adapted; simple sliding slopes can be easily analyzed by Limit Equilibrium techniques where as the most complex slopes require multiple approaches to be fully understood. This paper presents the most popular methods that can be used by engineer to analyze of design rock slopes. In this paper, different analysis and design tools are presented.

## 2. MODES OF ROCK SLOPES INSTABILITIES

The stability of rock slopes is of great concern in many projects such as, mining, roadway cuts, foundations, and dam excavation. Hajiabdolmajid and Kaiser [1] pointed out that rock slopes susceptible to instability can be divided into two main categories, the structurally controlled slopes and the complex rock slopes. Kinematics of a landslide or the type of movement or instability associated with the landslides is a key and important characteristic of the landslide itself and one of the principal criteria for classifying any landslide [2, 4].

Cruden and Varnes [2] recognized five kinematically distinct modes of landslide movement, fall, slide, topple, flow and spread. Cruden and Varnes [2] and [3] showed landslide classification system based on the slope kinematics and the type of the material involved in the instability. These types of landslide movement are described briefly in the following sections.

### 2.1 Falls

Rock fall starts with the detachment of rock from slope along a surface on which little or no shear displacement occurs. Masses and single fragments descend down the slope at a very rapid to extremely rapid gravitational movement. The material may fall freely through the air, bouncing, or rolling depending on the slope angle below the unstable section [5, 2]

Due to the high kinetic energy involve in the fall, the masses can be moved a relatively great distance compared to its original position. Cruden and Varnes [2] also differentiated between the styles of falling depending on the slope below the masses, above 76°free falling occurs while at less than that bouncing and rolling occurs.

### 2.2 Topples

Toppling is the forward overturning out of slope of a mass of rock about of axis below the center of gravity of the displaced mass [2]. Toppling sometimes triggered by the forces of gravity, the forces exerted by adjacent units, fluids, or ice within discontinuities that stresses the rock mass.
Goodman and Bray [6] described three main categories of toppling in nature; the flexural toppling, the block toppling and the block-flexure toppling, Benko [7] and [8] discussed a deep seated large scale toppling failure which is an extension of simple flexural toppling, they identified these cases in mining environments. Large scale toppling failure involves tensile failure of toppling columns as a result of shearing between the in-dipping discontinuities. These topples are common toppling at which the main discontinuity dip into the slope.

Later, Cruden [9] described the underdip topples at which the main set of discontinuities dip outside of the slope. Secondary toppling may form in nature and it is generally initiated by undercutting of the slope toe by natural events such as erosion or weathering or by human activity. Deep seated Complex rock topple may develop in nature or open pit mines, [7], [8], and [10] discussed the formation of deep seated topple in open pit mines.

### 2.3 Slides

Varnes [4] differentiated between two main categories of slides; the rotational slides and the transitional slides. The rotational rock slide move along curved rupture of surface, this curved and concave rupture surface may evolve as the stresses inside the rock mass exceed the strength and rupture surface formed in progressive manner. In the case of transitional slides, the mass move along a planar or undulating rupture surface. The sliding occur on one or more surface of rupture, joint, fault, weathered interfaces or weak layers, step path failure which involve intact rock destruction, may also occur. Wedge slide may occur on two intersectional joints; this failure is three dimensional slides, see Figure 1.

According to [2], the sliding type of movement is further divided into rotational, translational, compound, and complex and composite slides. Compound slides are intermediate between rotational and transitional and usually have steep main scarps, flattening with depth.

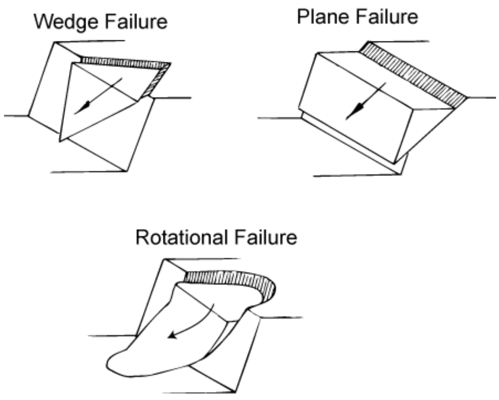


Fig. 2 Examples of Rock Slides, after [11]

### 2.4 Spreads

Terzaghi and Peck [12] introduced the term spread to the geotechnical community to describe sudden movements on water-bearing seams of silt or sand overlain by homogenous clays or loaded by fills. Cruden and Varnes [2] characterized as “extension of a cohesive soil or rock mass combined with a general subsidence of the fractured mass of cohesive material into softer underlying material”. The movement does not occur on a well-defined rupture surface. They distinguished three types of spreads; block spreads and Liquefaction spreads and complex spreads as identified in England by [13].

### 2.5 Flows

A spatially continuous movement, in which the surfaces of shear are short-lived, closely spaced and usually not preserved. The bottom boundary of the sliding mass may be a distinct surface at which differential movement has occurred or a relatively thick zone. In some cases, debris slid may develop into debris flows or debris avalanches as the moving material gains water, move into steeper slopes or loses cohesion.

Keefer and Johnson [14] differentiated between earth slide and earth flow based on the amount of internal deformation of the displaced mass. If the displaced mass is strongly internally deformed the landslide is most likely an earth flow. McRoberts and Morgenstern [15] define the term skin flow to describe a type of flows in the permafrost; the slope movement in the skin flow is rapid movement of a thin thawed soil and vegetation flows or slides over the permafrost table. A special category within this type of movement is represented by the slow flow in bedrock, known as deep-seated creep. In this type of movement large-scale gravitational deformation takes place without the formation of a pervasive failure surface.

Hungr et al. [3] proposed a classification system based on genetic and morphological aspects. They divide the flows to the following categories; slow gravel or non-liquefy sand flow, extremely rapid sand, silt or debris flow slides accompanied by liquefaction, extra-sensitive clay flow slides, peat flows, slow to rapid earth flows in non-sensitive plastic clays, mud flows, debris flow in steep established gullies, debris floods, debris avalanches, the flows formed by large scale failures in bedrock as rock avalanches. An example of rock avalanche is the Frank slide of Alberta.

## 3. FACTORS AFFECTING ROCK SLOPES STABILITY AND PROCESSES

Slope movement, can attributed to many causes, and in rare cases movement of a slope can be specified to one single reason. According to [4] the slope movement process involves a continuous series of events from cause to effect.

In general, rock slopes in mining environment, road way cut or natural rock slopes for instance, can be connected through cause and effect relation, Stacey [16] summarized these factors as; ground water conditions, the rock mass geological units, in situ and induced stress state, rock mass strength, rock mass structures and orientations, the geometry of the rock slope, the seismic environment, in the case of the open pit mine, the time frame of the mine operation may influence the mine. Eberhardt [17] presented the instability of rock slopes processes as a cause and effect. He examined many physical processes that might affect rock slopes numerically and showed that numerical methods are capable of modeling complex problems relating to geometry, non-linearity or the presence of coupled processes.

## 4. METHODS OF ANALYSIS AND DESIGN OF ROCK SLOPES

Many methods for rock slopes analysis and design are available and can be used by engineers ranging empirical methods to complex numerical modeling approaches.

The method of analysis has to be chosen based on the problem to be analyzed. For example, simple sliding analysis can be performed for a sliding block along persistent surface, while complex calculations required to analyze heavily jointed rock mass. It is also important to realize that more than one method might be needed for slope stability analysis so that to cover the shortcomings inherited in any individual method. Stead et al [18] discussed different approaches for rock slope stability and presented the advantages and disadvantages of each method. In the following I discuss briefly the most important analysis approaches.

### 4.1 Empirical Methods

Due to the fact that experience is essential in geotechnical engineering, Empirical methods are based on previous experiences. The controlling factors influencing instability of slopes, such as slope height, slope angle, geological structure, material type, groundwater conditions, and other parameters, are gathered and applied to the problem in hand. For example, Hoek and Bray [11] used the slope height and face angle to plot the stable and unstable slopes of previous case histories, this graph can be used as preliminary estimation of the stability of a slope. The process of empirical design is concentrated on learning from past successes and failures, [19].

The rock mass classification systems can serve as an empirical method, these methods incorporate many factors affecting the slope, examples of the classification systems are the Q and the RMR systems. The later has been modified by [20] to classify rock slopes and called the SMR (Slope Mass Rating). In this system, new factors for slope geometry and joints were added to the RMR system proposed by [21]. The stability of a slope is then assisted for stability and suggested support.

The empirical methods is relatively easy to apply and give the engineer a chance to learn from the past experience, but cannot be applied to design large slopes involve very complex geometry, coupled problems and/or complex network of discontinuities, especially if these slopes associated with high risk.

### 4.2 Kinematic Methods

These methods of analysis involve utilizing the stereographic projections to evaluate if a block of rock mass has a potential to move along fully developed discontinuities surround that block. Goodman [22] defined the kinematics as, “kinematics refers to the motion of bodies without reference to the forces that cause them to move”.

Hoek and Bray [11] explored the potential of the kinematic methods to explore simple rock slopes such as; Planar, wedge, circular and toppling movements. The use of the kinematic methods ignores the strength parameters and the acting forces on the slope, but it identify failure potential. The orientations of the discontinuities and the slope face such as dip and dip direction, are projected to lower hemisphere stereonet, the potential of forming block that is free to move is then evaluated. The strength of the discontinuities and of the rock mass is essentially ignored, as they are considered not to have an effect on the potential for failure Computer programs such as, DIPS developed by the Rocscince group can be used to plot the discontinuities; Figure 2 kinematic shows an example of such kinematic analysis using the stereonet projections.

### 4.3 Physical Modelling

Rock mass behavior can be tested through physical modelling which involves three methods; down scaling model material, larger loads on stronger material and the centrifugal forces modeling. Physical modelling can be applied in rock slopes modeling to predict the failure mode in a rock slope. In the first method, the down-scaling method involves the use of weak material such as plaster to model a rock mass which might fail under its own gravity weight,

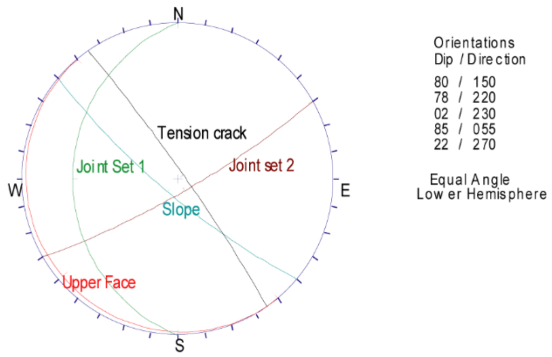


Fig. 2 Kinematic analysis for wedge failure

Barton [23] used very weak material to model rock slopes. In the second method, using of large loads in the laboratory such as, Uniaxial, biaxial or triaxial loading, can be applied to strong material such as concrete to simulate rock slopes behavior, [24] used this method to examine the failure of modeled material. The third method involves the using of centrifugal forces to model the increase of height of a slope, [25] and [26] used centrifugal acceleration to model toppling in rock slopes.

Benko [6] described the limitations of physical modelling, which include accounting for the effect of scaling from field conditions to the laboratory, and in deriving reliable quantitative results from the experiments. Physical modeling and especially the use of centrifugal acceleration is relatively expensive and limited in terms of the force required to cause failure of strong rock material in large rock slopes. For example, to simulate a 400 m rock slope with real rock mass properties, a 0.5 m modeled slope needs an acceleration of 800 g, which is hard and expensive to achieve. The advantage of this modelling approach is that the major rock structures can be incorporated in the modeled slope as seen in [25] and [26] models

### 4.4 Probabilistic Methods

The rock slopes involve many variable factors, in addition to the common variable factors between the soil and rock slopes such as the strength parameters, rock slopes have discontinuities that are naturally variable and not a constant value, example of these, the length of the gab and the orientation angles. Incorporating parameters variation in rock slopes design might shed a light on the important factor controlling the slope under study. In addition to estimate the probability of failure, risk analysis associated with some parameters might give confidence in the decision-making process for rock slope design, [27].

The parameters controlling rock slope behavior is more likely to have a distribution such as normal or beta distribution, rather than an exact deterministic value, which is more realistic. Although, the probabilistic approach is more powerful than a deterministic approach, but it has important limitations such as, the distributions of the parameters such as, the cohesion, friction and/or tensile strength must be identified, which is difficult to have due to the high cost associated with collecting of these data [7]. In numerical methods, incorporating probability theory required huge computational power; this might be achieved in the future as the computer power increase, the discrete element code UDEC [28] can accommodate some probability variables such as, the joint’s spacing, gap and length. Limitations to deterministic methods lie in the accuracy of defining the various input parameters; in addition, failure mode must be identified in most cases.

In probabilistic methods the parameter controlling the slope behavior are distributions and not a single value, so, to evaluate the probability of failure all of these distributions must be incorporated in the solution to obtain a performance function of the factor of safety.

One of the methods is to combine all the statistical distribution by using the Monte Carlo simulation. In the Monte Carlo simulation, each controlling parameter distribution is sampled randomly and the performance function is evaluated, if this procedure is repeated a large number of times a statistical distribution of the performance function can be built up, the probability of failure is then calculated as the ratio between the cases of failure to the total number of simulations.

### 4.5 Limit Equilibrium Analysis

Many limit equilibrium methods have been developed in geotechnical engineering to analyze and design slopes in both soil and rock mechanics. It is based on the concept of the factor of safety. In its simplest form the factor of safety is the ratio between the sum of the forces resisting failure, and the sum of the forces driving failure; a factor of safety greater than unity implies stability.

In general, to solve a geomechanical problem and achieve an exact solution, the differential equations of equilibrium, the strain compatibility equations, the constitutive equations for the material, and the boundary conditions of the problem must be solved. The methods of limit equilibrium analysis attempt to achieve this by a number of assumptions to simplify the problem, [29]. Computer programs have been developed that rapidly solve limit equilibrium equations, and determine a factor of safety. The programs SLIDE and SWEDGE [30] can be used to conduct limit equilibrium analysis for rock slopes.

The most popular limit equilibrium method is the method of slices, a slip surface is assumed and then the moving mass divided into slices, at each slice the force and/or moment equilibrium equations are solved to determine the inter-slice normal and shear forces. Major difference between various methods of slices techniques depends on which equations of statics are satisfied and the assumption regard the inter-slice forces. For example, while, Ordinary method only satisfies moment equilibrium Morgenstern-Price method [31] satisfies moment and force equilibrium and include both normal and shear inter-slice forces. Fredlund and Krahn [32] developed the General Limit Equilibrium method (GLE); it is based on two factors of safety; one with respect to moment equilibrium and one with respect to the horizontal forces. Understanding the limitations and assumptions included in each method of slices must be recognized prior to use of any method of slices in rock slope analysis. Sarma [33] introduced a method capable of handling internal shears observed in the rock mass.

### 4.6 Numerical modeling approaches

The development of computers and computational speed in the last three decades has resulted in development of numerical methods applications in the geotechnical field, for both surface structures and underground excavations. Large rock slopes are in general complex due to the heterogeneity, stress state, discontinuities, coupled processes, geometry, progressive failure and non-linearity of material behavior.

Due to these complexities, numerical simulation must be used to account for these factors; the numerical methods is capable of handling the boundary conditions, the constitutive equations of the material, the differential equations of equilibrium, and the strain compatibility equations. Many numerical methods have been developed and used in the geotechnical engineering; Stead et al. [18] discussed the advantages and disadvantages of some of the methods mentioned above. A brief description of the most important methods is discussed in the following sections.

#### *4.6.1 Continuum Modeling*

Continuum modelling assumes that the displacement field is continuous and results in a small displacement, shear failure by sliding along the maximum shear strain zone, and the tensile strength plays a minor role. The actual rupture surface does not form in the continuum modelling, so, the after failure analysis is not possible. Also, the discontinuities inside a rock mass cannot be modeled explicitly except for few major ones.

To overcome some of the shortcomings of the continuum modelling, new approaches have been developed such as, introducing new constitutive models and simulating localization of shear bands in the intact material. For example, Adhikary et al. [34] used the Cosserat medium to simulate rock slopes, but the actual rupture surface did not form. This approach according to [35] is a mesh dependent and the shear band has tendency to follow the pattern of the discretized mesh. Some constitutive models such as the ubiquitous joint model can simulate implicitly the behavior of the jointed rock mass.

Two continuum methods have been used in geotechnical engineering, the finite element method and the discrete element method. The difference between the two methods is the method of solution of the differential equations systems. One of the advantages of the finite element over finite difference is that the mesh generation is more flexible. For Example, FLAC [36] is a finite difference code developed by the Itasca group, while PHASE2 is a finite element code developed by the Rocsience group [30]. Figure 3 shows an example of a finite element model using PHASE2.

#### *4.6.2 Discontinuum Modeling*

Rock masses are generally characterized by block nature and a network of discontinuities that in most cases dominate the behavior of a rock mass. Including the structures in the rock slope modelling is essential and must be taken into account. Cundall [37] introduced the distinct element modelling approach at which the discontinuities can be modeled explicitly.

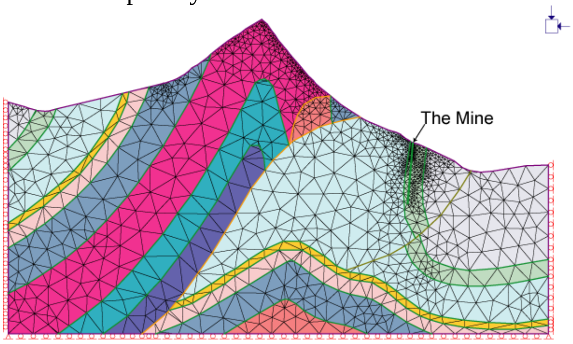


Fig. 3 Finite Element model

The blocks forming the model are free to move and rotate and completely detached from the rock mass body as failure occur. The blocks in this modeling approach are discretized into finite difference mesh at which failure can occur at the same manner of the continuum modelling approach.

Cundall [37] proposed the discrete element numerical modeling method. This logic is used by the Universal Distinct Element Code [28], which is one of the most commercially available distinct element programs by Itasca Consulting group. Rock slopes can be reasonably well modelled by using UDEC code. It can handle complex geometries along with number of material types and complex constitutive models. Any number of discontinuity sets and orientations can be included, all with different strength characteristics. But, the traditional discrete element methods require a defined failure surface and the actual rupture surface is not free to develop inside the rock mass.

*4.6.3 Hybrids Methods*

The previous mentioned methods are not capable of handling fracturing through intact material from the pre-existed structures inside the rock mass. Terzaghi [39] stated that most of the rock masses include non-continuous rock joints and the strength must be derived from both the joints and the intact rock between the joints.

To simulate this effect different methods have been developed based on the fracture mechanics principles; failure between rock joints can occur through tensile failure (MODE I) or as shear failure (MODE II). Lajtai [40] presented experimental study on Plaster of Paris and showed that the tensile strength control the failure of rock bridges especially at a low normal stresses range. Einstein et al. [41] developed an approach to model the fracturing through intact material between two joints, their approach was limited to one parallel joint set. Shen et al. [42] developed an approach to simulate fracturing through intact material by using the displacement discontinuity method; in his approach the fracture toughness for MODE I fracturing is compared to the stress intensity factor at the tip of a joint to allow fracturing from the tip of a pre-existing joint. He also includes the G-Criterion at which MODE II failure occur when the strain energy release rate is larger than the surface energy required to separate the material.

Scavia [43] and used the displacement discontinuity method and fracture mechanics principles to model fracture initiation and development through a network of pre-existing joints. He used the linear elastic stress intensity factor at the joint tip to initiate the fractures. Kaneko et al. [44] used the same principals to simulate a homogeneous rock slope. The failure was initiated at the toe and incorrectly propagated to the upper face of the slope, not to its crest.

The Particle Flow Code (PFC2D) [45] is a discrete element code at which the rock mass is simulated by circular particles that can be bonded together to represent intact rock. However, it is not clear how spheres should be calibrated to represent discontinuities as the spheres create very irregular surfaces. Potyondy and Cundall [46] concluded that extensive numerical calibration is needed to use bonded particles to simulate intact rock. Stead et al. [18] and Stead and Coggan [47] used the ELFEN hybrid method to simulate other rock slopes. This technique is promising, further comparison with deformation pattern might be needed. Figure 4 shows an example from their work. Alzo'ubi [48] also used a hybrid modeling modelling approach to observe the effect of layers thickness on the mode of instability. This hybrid approach is capable of modelling the evolving of non-directional physical rupture surface; it allows the formation, propagation, and coalescence of cracks inside rock slopes.

## 5. CONCLUSION

Understanding modes of movements of rock slopes is detrimental in choosing an effective method or methods of analysis. Failure of man-made and/or natural geomaterial slopes may highly impact societies. Rock slopes susceptible to instability could be divided into two main categories, the structurally controlled slopes, and the complex rock slopes. To classify a landslide or potential one, the type of movement or instability must be identified. This paper introduced the modes of movements that have been observed in the field. Several causes might cause rock instabilities such as freezing/thawing cycles, high water table, and over cutting of slopes; the slope movement process involves series of events from cause to effect. To analyze rock slopes; many numerical modeling approaches are at the disposal of engineers. I recommend to use more than one tool to produce results and compare them with the actual behavior of a rock slope. The classification, the causes, and the methods of analysis available will help researches as well as engineers to establish their method of choice to investigate a rock slope.

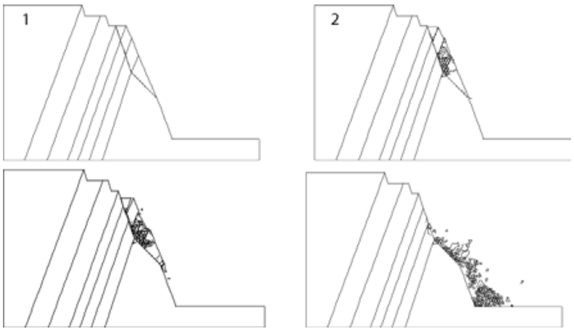


Fig. 4 Rock slope failure simulation by ELFEN

## 6. ACKNOWLEDGEMENTS

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