Influence of material spatial variability on slope stability in soft rock

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Abstract. Impacts of material spatial variability usually are not considered in traditional slope stability analyses. However, most geotechnical materials are not uniform inside a slope. The objective of the present study was to evaluate the impact of material spatial variability on slopes in soft rocks. The random field model was used to obtain the material spatial variability. The numerical analyses of slopes in soft rock mass were performed using a distinct element method based program UDEC. After that, Monte Carlo simulation, UDEC, and random field models were used together to analyze the impact of material variability and the slope stability. A series of numerical experiments were performed to understand the effect of joint spacing, joint angle, and size of scale fluctuation (δ). Also, failure types and mechanisms were synthesized and evaluated. The probability of failure became higher as the slope considered horizontal spatial variability of the material. The influence of the horizontal size of δ on slopes without joints was much higher than the slopes with joints. In addition, the slopes with joints yielded fewer failures as δ increased. The results of the analyses could be used for prevention of slope failure, future support system, and geotechnical exploration.

1. Introduction
The material properties and characteristics of weak or soft rock masses are different from hard rocks and soils. Traditionally, the analyses of excavated slope stability in the soft rock masses were usually similar to the slope analyses in soils. The failure mechanism of the soft rock masses usually includes both failures in intact rock and discontinuities. However, the behavior of the soft rock masses may also depend on the orientation of the discontinuities. Plane sliding on a discontinuity or circular failure may occur in soft rock masses. Besides, the impacts of material spatial variability were usually not considered in traditional slope stability analyses. However, most geotechnical materials are usually not uniform inside a slope. Therefore, the conventional ways of slope analyses are not very representative. The uncertainties and the results of errors will increase if the slope stability analyses consider the material spatial variability. On the other hand, most geological explorations are only conducted in vertical borings. Horizontal and inclined borings were seldom conducted, unless required by a client or under special conditions, such as tunnel excavation. Therefore, the present study examined the influence of the material spatial variability on the slope stability in excavated weak rock masses. Also, we studied the effect of discontinuities on slopes and the interaction between soft rock and discontinuities.
2. Types of slope failures and choice of the numerical method

Slope failure modes will depend on the types of materials and geological features involved, e.g., soils, rocks, or a combination of soils or rocks with joints or bedding. For soils, the typical slope failure modes are translational slides (infinite slope failure) and rotational slides (circular, non-circular, and multiple retrogressive slide failures). A detailed classification of landslides can be found in a report by Skempton and Hutchinson [1]. For rocks, possible slope failure modes include plane sliding, wedge sliding, toppling failure, and failures involved shearing or bending failures in an intact rock. These types of failures can be found in the report by Aydan et al. [2]. These failure modes in soil and rock slopes demonstrate the impact of geological features, especially for rock slopes, on the slope failure modes.

Most analytical methods used for the analysis of soil and rock slopes are based on the limiting equilibrium methods. Several methods, e.g., Bishop's, Janbu's, Morgenstern and Price's, Spencer's, and graphical wedge methods have been developed for the analysis of soil slopes. Details of these methods and procedures can be found in many books on slope stability. Common types of numerical methods used in stability analysis are finite difference method (FDM), finite element method (FEM), boundary element method (BEM), and distinct element method (DEM). The first three methods i.e. FDM, FEM, and BEM, are usually considered as continuum methods, while DEM is considered as a discontinuum method.

Several numerical software had been used in past for analyses of slope stability in rock masses and soils. However, modelling of joints is possible using the finite element method, but this requires some special elements, such as joint elements [3] or thin-layer elements [4], and the use of various constitutive equations to represent the behavior of the rock mass and individual discontinuities. These special elements are related and connected to other elements in the continuum. Separation and movement along the interfaces are limited and in need of a special treatment. Discretization of the entire domain into "joint" elements and continuum elements would be a very slow and expensive way for complicated modelling conditions. The most convenient and versatile method available for simulating discontinuum behavior is the distinct element method. Unlike other numerical modelling techniques, the DEM was designed specially to model discontinuities. The DEM considers assemblies of rigid or deformable blocks that interact through springs acting at the contact points. It models the joints between blocks as interfaces, i.e., a joint is treated as a boundary condition rather than a special element in the whole system. Each block may be free to rotate or to translate with associated slip and/or separation at block interfaces. Rock is distinguished from other engineering materials by the presence of discontinuities. Rock masses usually contain bedding planes, joints, faults, and other structural features that render them discontinuous and often controls their engineering behavior. For rock mass slopes, a discontinuum method would be preferred for analysis. Therefore, the distinct element method (DEM) was chosen in this study to perform numerical analyses and to analyze the failure modes of slopes.

3. Impacts of material variabilities and random field model

Stability of excavated or natural slopes in soils and rock masses is frequently influenced and controlled by the existence of joints and weaker zones or layers. The properties of rocks and soils used in the previous research of slopes had commonly been modelled deterministically with a perfect spatial correlation, i.e., a material property over the whole spatial domain was characterized by one single variable. Thus, the material properties were considered to be the same at all places within the evaluated slope. However, soil and rock properties usually show significant variation from point to point even within technically homogeneous soil or rock stratum as shown in figure 1. Weaker zones or spots could be developed arbitrarily during geological deposition processes since geological materials all have some degree of variability spatially. Slope failures could be initiated from the discontinuities and soft spots and spread progressively throughout a slope. Thus, site exploration and slope stability analyses considering material spatial variability would be very vital.

Most of the studies on rocks have been carried out on modelling spatial variation of the occurrence of joints or fractures in rocks [5-8]. Very little research has been done on spatial variability of rock properties, especially for weak rock properties. In order to understand the impacts of material
variabilities on slope stability, the random field model proposed by Vanmarcke [9] was chosen to characterize the studied material. Vanmarcke [9] had pointed out that there are three major sources of uncertainty in material profile modelling. The random field model suggested by Vanmarcke [9] deals mainly with the first type of uncertainty, i.e., the natural heterogeneity or in-situ variabilities of the material caused by variation in mineral composition and stress history. Nevertheless, this model is valuable in planning investigation schedules during site examination and designing material testing. The method of random field model is to treat the geologic material (soil or rock) profile as a random field, also known as stochastic or random processes. In this model, material properties are not perfectly correlated within a single material layer, but they exhibit some limited spatial correlation. The material property can only be correlated within a certain distance. The two samples taken from similar materials may possess different material properties and are treated as separate random variables if the two samples separate longer than a distance. Thus, the variability of material property can be considered and modelled more suitably using the random field model.

The average material properties are assessed over a depth interval or a lag distance, $L$, to illustrate spatial variability. The spatial variability of material properties tends to be reduced due to this averaging processes. This averaging process leads to a decrease in variance as the length of the averaging distance increases. Thus, Vanmarcke [9-10] introduced the calculation of the scale of fluctuation, $\delta$. For two points that lie within the distance, $\delta$, the corresponding values of the material property are likely to be either both above or both below the mean value. Materials with a large value of $\delta$ will have uniform properties across space, while materials with a small value of $\delta$ will have more random properties across space. The material spatial variability of the studied rock formation, Eagle Ford Shale was assessed based on the test results of Brazilian tensile strength, $\sigma_{t,B}$, tested at close interval. The estimated scale of fluctuation $\delta$ was about 1.22 m. The detailed procedures about the theory of random field model and the equations obtaining the scale of fluctuation for the Eagle Ford Shale can be found in reports by Hsu [11] and Hsu and Nelson [12].

![Figure 1. Random variation of material properties in a geological formation (after Vanmarcke [9]).](image)
4. Numerical analyses using distinct element method

The distinct element method (DEM) was first introduced by Cundall [13] as a means of modelling the progressive failure of rock slopes. Cundall and Hart [14] defined the meaning of a distinct element method. The DEM uses a dynamic relaxation technique to solve Newton's laws of motion to determine the forces between, and the displacements of, units during the progressive and large-scale deformation of discontinua. This method has become a popular method for analyzing the problems associated with the impacts of discontinuities in underground construction, slopes, and foundations. A commercially available program called universal distinct element code (UDEC) [15], a two-dimensional program developed by Dr. Cundall since 1980. The UDEC has the capabilities to model variable rock deformability, plastic behavior of intact rock, nonlinear elastic behavior of the joints, and fluid flow and fluid pressure generation in joints.

In UDEC, the rock mass is comprised of joints and discrete blocks, which could be rigid or deformable. Inside each deformable block, the finite difference method is used to discretize it into a finite difference mesh. Thus, in order to model the influences of material variability on slope stability, the random field elements (REFs) were embedded with finite difference mesh. The size of random field element was chosen as 6.1 m, shown in figure 2, which is 5 times of scale of fluctuation to avoid correlation between two elements.

![Random field element (RFE=6.1m)](image)

**Figure 2.** Configuration of modelled slope and random field elements.

5. Slope stability analyses considering material spatial variabilities

A method integrating distinct element method (DEM), random field model, and Monte Carlo simulation was used to perform a numerical simulation to analyze the impacts of material spatial variation. The material properties, slope height, and slope angle were chosen similar to the studies by Hsu [11] and Hsu and Nelson [12]. The slope height and slope angles were 61 m and 60 degree, respectively. The input material properties and parameters for numerical analyses are shown in table 1.

Persistent discontinuity sets in the weak rock masses are considered in the numerical analyses of slope stability. The groundwater level was assumed to be deep enough not to affect the stability of slopes for the long-term condition. The strain-softening model was used for analysis of weak intact rock. A Coulomb slip model with or without a capability of weakening upon failure was used for the discontinuities. The coefficient of earth pressure at rest, Ko was assumed to be 1.5.

5.1. Probability of failure

Two hundred slope analyses were performed for each combination of joint dip and joint spacing considering material spatial variation, and the probability of failure (Pf) was then calculated. Six different combinations of joint dip and spacing were analysed. These six slope cases were stable if the slopes were analysed with average material properties. However, these slopes may experience a different
5 degree of slope failures due to the impacts of material variation. A stable boundary was obtained by Hsu [11], and the results of analyzed probability of failures (Pf) were plotted in figure 3. The probability of failure was still very high as the joint dip or joint spacing is near the stable boundary. The probability of failure for the steep joint dip was higher than the gentle joint dip under the same joint spacing. Thus, the location of weaker material may cause the failure of a slope. Typical toe failure for a gentle joint dip and angle and deeper base failure for steep joint dip have been shown in figure 4a and 4b, respectively. Slopes without any joints had much lower probability of failure, i.e., the presence of joints did cause slopes to fail much easier.

5.2. Impacts of weaker spot location
To illustrate the impacts of material variation and location of weaker material, the slopes without any joints were chosen to understand the failure mechanisms and paths. The influence of weaker material locations can be seen in figure 5. Contours of material property type 1 to 5 were plotted in the figure 5. Material type 1 represents the material with the lowest shear strength. The locus of the failure zone, dotted lines were also plotted in figure 5. Failure of the slope started from the weaker zones around the slope toe and caused the failure zone to extend up progressively to the slope top.

Table 1. Average material properties for intact Eagle Ford Shale (EFS), bedding and discontinuities.

| Property                        | Intact EFS | Bedding | Joint set |
|--------------------------------|------------|---------|-----------|
| Cohesion, MPa                  | 0.41       | 0.34    | 0         |
| Friction angle, °              | 24         | 17      | 13        |
| Young's modulus, MPa           | 276        | 276     | -         |
| Tensile strength, MPa          | 0          | 0       | 0         |
| Dilation, °                    | 0          | 0       | 0         |
| Poisson's ratio                | 0.25       | -       | -         |
| Unit weight, g/cm²             | 2.24       | 2.24    | -         |
| Joint normal stiffness, kn, MPa/m | -         | 1005    | 1005      |
| Joint shear stiffness, ks, MPa/m | -         | 502     | 502       |

Figure 3. Boundary of stable and unstable zones, and the probability of failures for selected joint dips and joint spacings.
5.3. Influences of the size of a random field element
The influence of the size of the random field element on the slope stability was also assessed. The size of the element at both vertical and horizontal directions was changed to evaluate the probability of failure. For slope with joints, an increase of horizontal random field element size only increases marginal probability of failure. However, the probability of failure for the slopes without joints may increase significantly if the horizontal random field element size increases. The reason for the above results may be due to the presence of the joints interrupting the extent of a failure zone.

Figure 4. (a) Typical toe failure for gentle joint dip and angle; and (b) deeper base failure for a steep joint dip.

Figure 5. Contours of material properties and the locus of failure zone for a slope without joints.

6. Conclusions
The influence of material spatial variation on the slope stability in weak rock masses was studied in this paper. The random field model was used to obtain the material spatial variability. Monte Carlo simulation, UDEC, and random field model were used to analyze the impact of material variability and the slope stability. Some conclusions obtained from this study are as follows:

1. The probabilities of failure were all very high, as the combination of the joint dip and joint spacing is close to the unstable zone boundary.
2. The slopes without joints yielded higher probabilities of failure than the slopes with the joint when considering material spatial variation.

3. As the horizontal size of the random field element increased, the probabilities of failure for the slopes with joints remained unchanged.

4. The slopes without joints, the probability of failure increases as the size of random field element increases.

5. The joints may become a non-uniform boundary for the weak materials to fail or shear as the slope becomes unstable.

References

[1] Skempton A W and Hutchinson J N 1969 Stability of Natural Slopes and Embankment Foundations, State-of-the-art Report In Proc. 7th Int. Conf. SMFE in Mexico City 2 p 291-335

[2] Aydan O, Shimizu Y and Kawamoto T 1992 Rock Mass Characterization System for Rock Slope Stability Analysis In Rock Characterization ISRM Symp. EUROCK’92 in Chester pp 275-280 UK, ed J A Hudson (Thomas Telford Services Ltd.)

[3] Goodman R E, Taylor R L and Brekke T L 1968 A Model for the Mechanics of Jointed Rock J. of Soil Mech. Found. Eng. Div., ASCE 94(3) 637-659

[4] Desai C S, Eitani I M and Haycocks C 1983 An Application of Finite Element Procedure for Underground Structures with Nonlinear Materials and Joints In Proc. 5th Int. Cong. Rock Mechanics p 209-216 (Australia: Melbourne)

[5] White W 1993 Soil variability: characterization and modelling Proc. of the Conf. on Probabilistic Methods in Geotechnical Engineering pp 111-120 Canberra, Australia, ed by K S Li and S-C R Lo (A. A. Balkema)

[6] Einstein H H 1993 Modern Developments in Discontinuity Analysis-The Persistence-Connectivity Problem Comprehensive Rock Engineering 3 193-213

[7] Villaescusa E and Brown E T 1990 Characterizing Joint Spatial Correlation Using Geostatistical Methods Proc. of the Int. Symp. on Rock Joints pp115-122

[8] Mostyn G R and Li K S 1993 Probabilistic slope analysis—state of play In Proc. Conf. on Probabilistic Methods in Geotechnical Engineering pp 89-110 (Canberra, Australia, A A Balkema, Rotterdam)

[9] Vanmarcke E 1977 Probabilistic modeling of soil profiles J. of Geotechnical Engineering Division, Proc. of ASCE 103 GT11 1227-1246

[10] Vanmarcke E 1983 Random Fields: Analysis and Synthesis (The MIT Press) pp 382

[11] Hsu S C 1996 Stability and Failure Mechanisms of Slopes in Weak Rock Masses PhD Dissertation (The University of Texas at Austin)

[12] Hsu S C and Nelson P P 2005 Material Spatial Variability and Slope Stability for Weak Rock Masses J. of Geotechnical and Geoenvironmental Engineering 132(2) 183-193

[13] Cundall P A 1971 A Computer Model for Simulating Progressive Large Scale Movements in Blocky Rock Systems In Proc. of the Symp. of the Int. Society of Rock Mechanics 1 Paper No. II-8 (France: Nancy)

[14] Cundall P A and Hart R D 1993 Numerical Modeling of Discontinua Comprehensive Rock Engineering 2 231-243

[15] Itasca Consulting Group Inc. 2004 Universal Distinct Element Code Version 3.1, Minneapolis, MN