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The distinct element method-based model and its use in slope stability analysis

Tara Nidhi Bhattarai
Department of Geology, Tri-Chandra Campus, Tribhuvan University
Ghantaghar, Kathmandu, Nepal
(E-mail: tnbhattarai@wlink.com.np)

ABSTRACT

With the advent of various modeling programs, it has become possible to build a model of a rock mass and perform extensive analysis to solve related rock engineering problems. However, there has been almost no research effort devoted to the generation of the basic input data that is needed for the faster and better model and for our improved design techniques. In light of these points, this paper describes a step-by-step procedure for the characterization of discontinuity as a means to prepare input data for a model construction based on the distinct element method (DEM). A case study is also presented in which the DEM models were constructed to carry out the slope stability analysis of an excavated rock slope. A rock reinforcement system is also designed based on the results obtained with the DEM analysis.

INTRODUCTION

Stability of natural and excavated slopes is always of great concern in rock engineering. Rock slopes of a site may be composed of single or multiple types of rocks having similar or dissimilar origin. Furthermore, they may be dissected by many discontinuities such as bedding, joints, faults and fractures. Accordingly, rock masses of a slope are not a continuum and their behaviors are largely controlled by these discontinuities.

Stability analysis of a rock slope consisting of a single or of a small number of blocks associated with a single persistent discontinuity or small number of discontinuities may be carried out by the limit equilibrium method as described, for example, by Giani (1992). In the case of jointed rock masses comprised of a finite number of discrete and interacting blocks, the finite element method (FEM) and the distinct element method (DEM) have often been used. The FEM, however, uses a continuum formulation and is, therefore, not suitable to simulate large deformation such as toppling failure of rock masses. On the other hand, the DEM includes a discontinuum approach in analyzing blocky rock masses and it uses a force displacement law, which specifies forces between blocks, and a motion law, which specifies the motion of each block due to unbalanced forces acting on the blocks. In addition, the rock mass is modeled as an assemblage of rigid or deformable blocks, and discontinuities are regarded as distinct boundary interaction between these blocks (Cundall 1971).

With the advent of various modeling programs such as the FEM and the DEM, it has become possible to build a model of a rock mass and perform extensive analysis to solve related rock engineering problems. Theoretically, it is expected that more detailed information will lead to a better model. But practically, more detailed information means more data, leading to more field and laboratory measurements, which demand extra time and resources, but its output, in some occasions, may be counter-productive in making the model too complex. After all, models are built because the real ground condition is too complex for our understanding and thus it does not help if we build models that are also too complex (Starfield and Cundall 1988). Therefore, the art of modeling lies in determining what aspects of the geology are essential in relation to the project being investigated and how to quantify pertinent geological structures as appropirate input data for the model. However, existing literature reveals that there has been no any rigorous method that may be applied to prepare input data for model construction. Hoek (1995) has also mentioned that there has been almost no research effort devoted to the generation of the basic input data that we need for our faster and better model and for our improved designed techniques. In light of these points, this paper describes a step-by-step procedure for the characterization of discontinuity as a means of preparing input data for the DEM model construction.

The field site described in this paper is situated at the Omaru River lower dam site, Kyushu Island, Japan, where a dam is to be constructed to create a lower storage reservoir for a pumping power plant. The DEM models were constructed to carry out stability analysis of the left slope of the river so as to understand the response of the slope after excavation for a dam foundation, and also to analyze a suitable rock reinforcement system to stabilise the excavated slope. This paper discuses how input data are prepared and stability analysis is performed with the DEM models.

GEOLOGY

Characterizing the geology of a site is one of the important tasks in model construction because input data regarding geologic features such as rock types and rock thicknesses, and major geologic structures such as unconformities, faults, fold axes, joints, bedding and their orientations are crucial for discontinuum analysis. Besides understanding the geologic setting of the site, it is equally important to assess whether the location at which the joint data are collected has been subjected to the same geologic history of deformation as the location where extrapolation is to be made. If their histories are found to differ, extrapolation is not valid (Piteau 1973). Thus, as shown in Fig.1, characterizing the geology of a site should be the first task in preparing input data for a model construction.

An extensive surface geologic mapping of the site was carried out followed by observation along test adits and examination of drill cores to characterise the detailed geology of the dam site. Geologically, the area is underlain by the Hyuga Group consisting of an accretionary complex of Middle Eocene to Early Oligocene age (Kimura et al. 1991). As shown in Fig.2, the slope surface consists of colluvium of about 6 to 13 m thick on the left side and less than 3 m thick on the right side of the river. Underlying the loose colluvial zone, there exists a sequence of alternating sandstone and shale zones. The rock types in the sandstone zone are sandstone associated with a few layers of shale, whereas the shale zone is composed of mainly shale intercalated with a few beds of sandstone. The thickness of sandstone zones is generally within the range of 0.5 to 3.0 m on the left slope, but on the right slope they form a band of 6 to 11 m thick. On the other hand, the thinly bedded shale gives rise to zones that vary between less than 1 m to as large as 5 m thick. The beds are persistent all along the hill slope with an average dip of 34° and dip direction towards the river, that is, parallel to the dip direction of natural hill slope whose gradient is 33° on average.

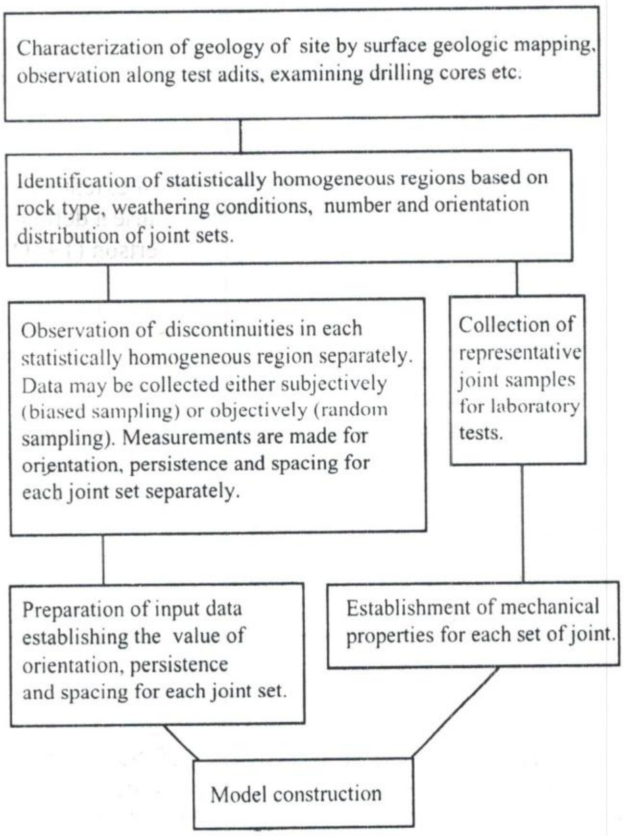


Fig. 1: Steps involved in the process of input data preparation for model construction.

DISCONTINUITIES

Discontinuity measurements are taken for various purposes, which include investigation for geological structure, rock mass classification and generating input data for specific analytical, numerical or empirical models of rock mass stability, deformation, fluid flow, blasting, rock cutting or support design. The data required for each of these purposes will also vary considerably, necessitating different sampling strategies. In subjective (biased) sampling, only those discontinuities that appear to be important are measured. In an objective (random) survey, all discontinuities intersecting a fixed line (scanline sampling) or area of rock exposure (window sampling) are described. A danger of using the objective approach is that the need to make engineering judgments in the field is under-emphasised. Data may be collected with little thought as to their implications and critical, but rare, data may be overshadowed in analysis by vast quantities of statistically correct, but irrelevant data. On the other hand, the subjective approach requires suitably experienced personnel to do the work and most importantly good, relevant exposures. Thus, the most suitable method for collecting data for a particular project will depend upon the nature of the problem, the quality of exposure, the resources available for investigation and the experience and expertise of the personnel involved (Hencher 1987).

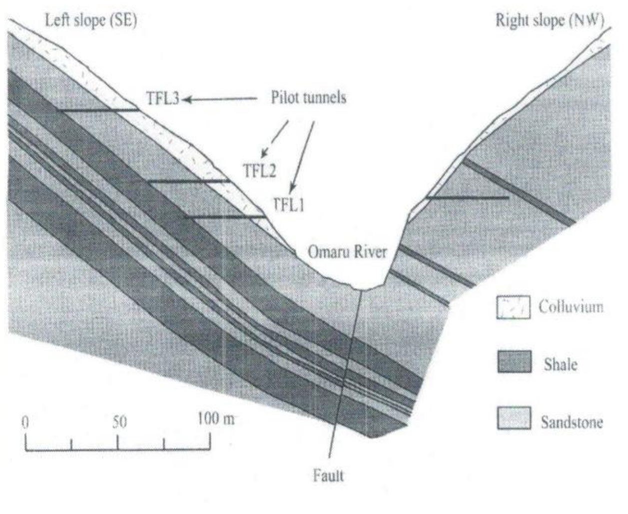


Fig. 2: Geological cross-section of the site.

This section describes a procedure intended to prepare input data regarding discontinuity parameters, namely the orientation, spacing and persistence (trace length) required to construct the DEM model of the investigated site. Since the site covers a considerably large area and also good exposures are available, the subjective approach was employed.

Statistically homogeneous region

Joint geometry pattern may vary not only from one rock type to another, but also within the same rock type. Therefore, the first step in the procedure of joint geometry characterization in a rock mass should be the identification of statistically homogeneous regions (Kulatilake et al. 1993). Such a region includes a portion of a rock mass that contains the same rock type having similar weathering conditions, and the same number of joint sets and with similar orientation distributions. The weathering conditions may be assessed as per British Standard (Anon 1981) or following the method adopted by Central Research Institute of Electric Power Industry (CRIEPI 1992), Japan. Orientation data are measured using borehole cores or examining exposed rock faces and are plotted in the Schmidt contour diagram to identify clusters or sets and their representative orientations (ISRM 1978). the level of detail required in collecting orientation data depends on the nature of the project and the stage at which the information is used. In general, the sample zone should contain between 150 and 350 discontinuities to obtain realistic equal area polar plots (Priest 1993).

As already mentioned above, the rock mass of the site consists of intercalating zones of sandstone and shale underlying colluvium. These three types of materials have made the site heterogeneous as their engineering properties differ greatly. Fig. 3 shows the weathering grade of the rock mass established as per CRIEPI (1992) by observing the rock mass exposed along three horizontal drifts driven in the left slope of the river. Orientation data of discontinuities measured independently for each of these weathering classes show that nearly identical pole clusters prevail in sandstone zones irrespective of weathering classes. An example of such clusters is shown in Fig.4. In contrast to the sandstone zone, only a few joints exist in the shale zones, which implies that the contact between shale and sandstone zones are major discontinuities. Taking these points into account, the sandstone zones and shale zones were treated as two different statistically homogeneous regions and joint characterizations in these regions were also performed separately. It is important to note that the soil cover lying on the slope face should also be treated as a different statistically homogeneous region, but since this layer will be excavated during construction of the dam foundation, it has not been included in further analysis.

Joint sets and their orientations

Besides bedding, three prominent sets of joints were delineated in the sandstone zones based on the clusters or sets that appeared in the Schmidt contour diagrams discussed above. A single orientation value that is representative of the discontinuities in each set was then established on the basis of the "center of gravity" of a cluster of normals on the contoured diagram (Fig. 4). The data shown in Fig. 4 correspond to C_M class of sandstone. Similar analysis was performed with data collected from C_L and C_H classes of sandstone, separately. Their mean values were taken as the final input data for slope stability analysis. The orientation of bedding in shale zone was also established in the same way.

Persistence (trace length)

Persistence refers to the continuity or areal extent of a discontinuity and is particularly important because it defines the potential volume of the failure mass. Robertson (1971)

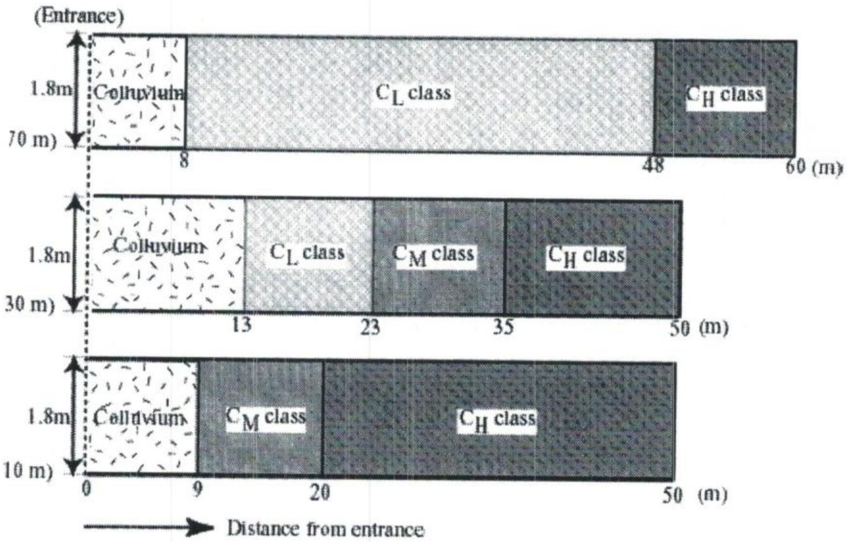


Fig. 3: Weathering grade of the rock mass (sandstone) as per CRIEPI (1992) observed in three pilot tunnels. The locations of the pilot tunnels (TFL1, TFL2 and TFL3) are shown in Fig. 2.

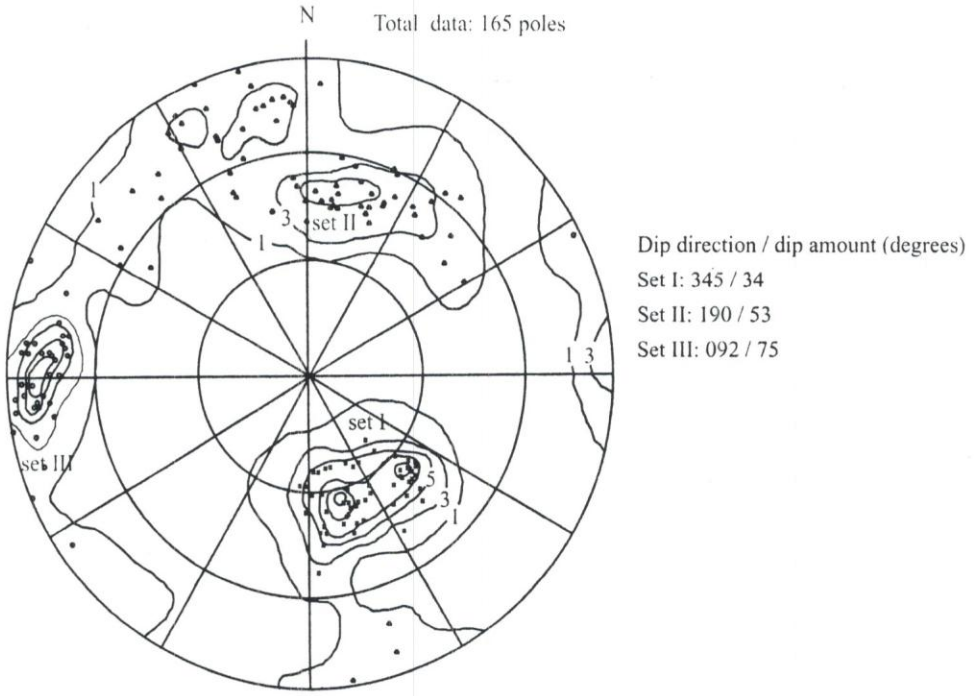


Fig. 4: Schmidt contour diagram representing the orientation of three sets of joints on a polar equal-area net. Data corresponds to C_M class rock exposed in three pilot tunnels shown in Fig. 3.

has mentioned that the dip length and strike length of a joint set are of approximately isotropic dimension. During the field investigation, an attempt was made to determine the most frequently occurring trace lengths of prominent joint sets and measurements were carried out accordingly. The trace length value was then quantified by plotting the field data in a histogram as shown in Fig. 5 from which the modal value and dispersion may be evaluated. The data shown in Fig. 5 correspond to C_L class of sandstone. Similar histograms were constructed with data collected from C_M and C_H classes of sandstone separately. Final input data were then decided considering these three histograms collectively.

Spacing

The distance between two discontinuities of the same set measured normal to the discontinuity surfaces is called spacing. Together, persistence and spacing of discontinuities define the size of blocks. A convenient method of presenting large numbers of spacing data for which statistical treatment may be required is the use of histograms, one for each set of discontinuities as has been described by Priest & Hudson (1981). They concluded that when a sufficiently large sample of these individual spacing values (preferably more than 200 individual measurements) are plotted in histogram form, a negative exponential distribution is often evident. They noted further that the general trend of such histograms is for many small spacing values and a few very large spacing values in the distribution. All these points were kept in mind during the field investigation so that an attempt was made to determine spacings of prominent joint sets. The spacing value was then quantified by plotting the field data in a histogram as shown in Fig.6 from which the modal value and dispersion was evaluated. The data shown in Fig. 6 corresponds to the C_L class of sandstone. Similar histograms were constructed with data collected from the C_M and C_H classes of sandstone separately. Final input data were then determined by collective consideration of the three histograms.

Surface features

Beside the geometrical parameters mentioned above, one also has to assign mechanical properties for these discontinuities as a means to describe their surface features. Mechanical properties include the values of friction angle, cohesion, dilation, and normal stiffness and shear stiffness. Procedures of laboratory tests to establish these properties may be found in Esaki et al (1998).

PREPARATION OF INPUT DATA

Discontinuity parameters, namely, the orientation, spacing and trace length define the geometry of a joint set and thus determine the shape and size of the resulting rock blocks, which are important indicator of rock mass behavior (ISRM 1978). For that reason, values of these parameters were established as per the methods explained above and rock blocks were generated in a DEM model based on these values (Table 1). But as shown in Fig.7, this approach results rectangular rock blocks throughout the model, which was not in agreement with the field situation where interlocked blocks were also observed. To minimize this problem, the concept of joint gap length as mentioned in UDEC (1996)

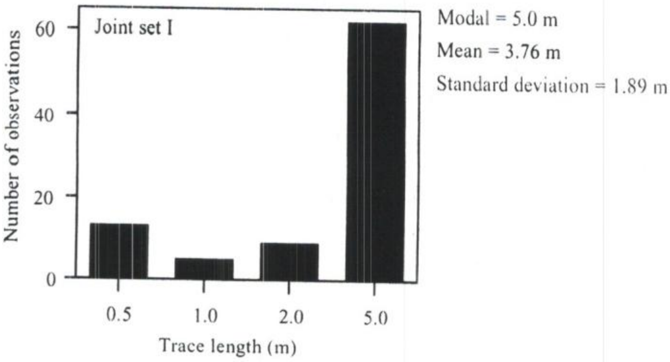


Fig. 5: Distribution of joint persistence (trace length) in sandstone (C_L class).

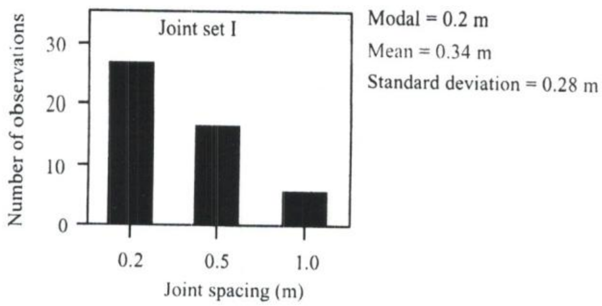


Fig. 6: Distribution of joint spacings in sandstone (C_L class).

was considered and the following method was used to establish its value.

The gap length describes the non-connected length of a joint segment. In other words, it is the space between the end points of two adjacent collinear joint lines. Since the related joint segments within the same set may not be always aligned along a straight line, it is very difficult and time consuming to measure the joint gap length. However, a possible range of its value may be estimated by visualizing the distribution of joint pattern in the field. Afterwards, an attempt may be made to identify the most probable joint gap length by comparative study among various joint patterns generated in DEM models using estimated values of gap length while keeping the values of other geometric parameters as determined by the field investigation. The following example clarifies this approach.

A representative rock mass lying on the left slope of the Omaru River was selected and, by using the method already explained above, the values of orientation, trace length and spacing of beddings and two set of joints (joint-I and joint-II) were established as shown in Table 1. The joint pattern resulted in a DEM model, assuming zero gap length, is already shown in Fig.7.

Based on the field observation, the bedding was considered to be fully persistence (zero gap length). The following two assumptions (Table 2) were made to establish the values of gap lengths (g_m, mean value, and g_d, value of standard deviation from the mean) for each joint set.

Case 1: The value of g_m of the joint set-I was considered to be equal to 5% of the mean value of spacing of the joint set-II and vice versa. The value of g_d was considered to vary among 0%, 10% and 15% of the newly decided value of g_m.

Case 2: The value of g_m of a joint set was considered to be equal to 5% of the mean value of trace length of the same joint set. The value of g_d was considered to be equal to 0% of the newly decided value of g_m.

To identify the appropriate choice of the above-mentioned cases, joints were generated in DEM models and the resulting joint patterns were compared with a representative joint configuration observed in the field. It was observed that the resulting rock blocks (Fig.8) of upper-case 1(with g_d = 10% of g_m) are more or less identical to the field condition and these values (Table 3) were used as input data for model construction.

MODEL CONSTRUCTION

One of the purposes of building a model is to understand the response of a slope after excavation works carried out for a dam foundation. In a similar way, slope stability analysis results in delineating of the likely volume of failed rock mass in order to design countermeasures such as rock bolting. Thus, while constructing a model, one has to consider the entire rock mass in which influence of excavation may be assumed. Such an approach may include thousands of rock blocks in the model, which may pose a problem as to how the long computational time required for a DEM analysis may be reduced. To overcome this problem, a conceptual model was constructed first so as to identify important mechanism, modes of deformation and likely modes of failure of the slope. Based on the output of such analysis, further steps were taken as mentioned below.

Conceptual model

Since the purpose of constructing such a simple model is to understand the basic features of failure within reasonable computational time, only the most critical part of the rock mass (Fig. 9) was selected. By using the data shown in Table 1 and 3, a DEM model was constructed along a representative vertical section of the slope. As shown in Fig. 10, the model exhibits the situation after excavation for a dam foundation. Although the rock mass in the field consists of sandstone interbedded with thin layers of shale, only sandstone is included in the model. However, the mechanical properties of shale were used for the bedding planes of sandstone as a means of accommodation the effect of omitting shale layers in the model. Mechanical properties (Table 4) of discontinuities used during the analysis were established by laboratory tests following the procedures explained in Esaki et al (1998). The model was then run to

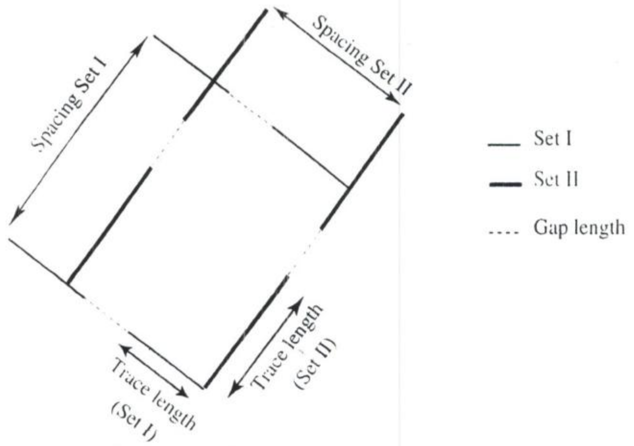
Table 1: Values of joint parameters established by the field investigation.

Parameters	Bedding		Joint set - I		Joint set - II	
	Mean	Standard deviation	Mean	Standard deviation	Mean	Standard deviation
Orientation (degree)	345/34	-	345/34	-	165/53	-
Trace length (m)	600	-	0.915	0.496	0.851	0.458
Spacing (m)	7	-	0.515	0.317	0.353	0.211

Orientation refers to dip direction/dip amount.

Table 2: The assumed values of joint gap length calculated on the basis of mean value of spacing and trace length given in Table 1. Definition of the related parameters are shown in the Sketch 1 below.

	Mean value of gap length (g_m) (m)		Standard deviation (g_d) (m)	
	Case 1	Case 2	Case 1	Case 2
Set I	5 % of mean value of spacing of Set II = 0.01765	5 % of mean value of trace length of Set I = 0.04575	0 % of (g_m) of case 1 = 0.0 10 % of (g_m) of case 1= 0.001765 15 % of (g_m) of case 1 = 0.00264	0.0
Set II	5 % of mean value of spacing of Set I = 0.02575	5 % of mean value of trace length of Set II = 0.04255	0 % of (g_m) = 0.0 10 % of (g_m) =0.002575 15 % of (g_m) =0.00386	0.0



Sketch 1: Definition of joint parameters mentioned in Table 2.

identify the failure mode and also to gain some insight into the location of failure initiation and the growth of failure with time. In other words, the intention was to determine whether the model would reveal some important mechanisms similar to those that were likely to occur in the field, so that the model could serve as a tool for further analysis.

As shown in Fig.11, the model predicts that the plane (shear) failure first initiates at bedding exposed near the bottom of the slope and then follows along the bedding plane of the sandstone. It would be appropriate to examine the field situation in order to verify this result. The excavated slope strikes parallel to the strike of the bedding planes. In addition, the former dips with an angle that ranges between

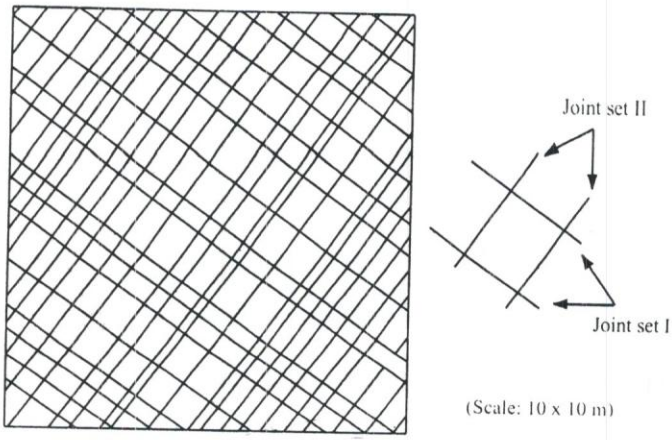


Fig. 7: Joint pattern resulted with the input data of joint orientation, persistence and spacing shown in Table 1.

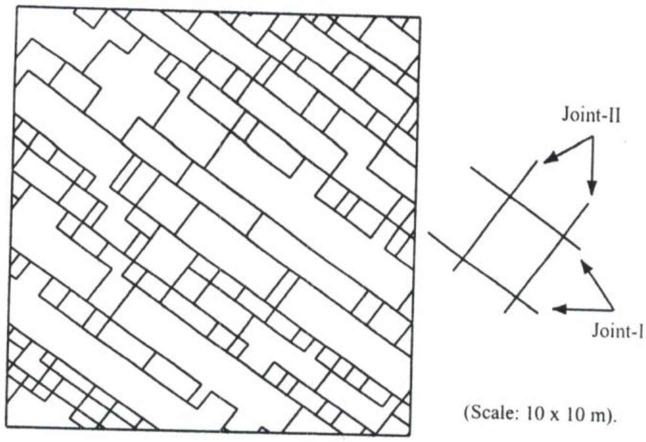


Fig. 8: Joint pattern resulting after assigning the value of joint gap length in addition of the input data of joint orientation, persistence and spacing.

Table 3: Values of joint gap length established by the DEM analysis.

Parameters	Joint set - I		Joint set - II	
	Mean	Standard deviation	Mean	Standard deviation
Gap length (m)	0.01765	0.001765	0.02575	0.002575

45° and 55°; whereas the latter amounts to 34°. The internal friction angle of the bedding plane in sandstone is only 24°. Taking these values into account, the kinematic admissibility analysis clearly indicates "daylight" situation for plane failure along the bedding planes. In addition, the failure plane forms a critical weak discontinuity because of its longer persistency, which considerably reduces the influence of overburden (normal stress) acting on it. Past failures occurring in the field have shown similar features. For these reasons, the failure pattern as depicted in Fig.11 seems to be reasonable and thus the model may be used for further analysis.

It is important to note that the conceptual model does not cover the entire construction site as already mentioned above. Besides, bedding planes in the sandstone are persistent all along the slope and thus there remains a possibility that the whole slope may slide down along these discontinuities making the entire rock mass, lying upslope from the excavation, unstable. In other words, there is a possibility that the failure may extend beyond the left boundary of the model (Fig.10). Hence the conceptual model is not sufficient to predict the full effect of the excavation on the stability of the slope. This situation demands modeling the entire rock mass lying between the river and the slope crest so as to perform an analysis that will define the likely failed portion of the slope with acceptable accuracy.

Analytical model

Since the likely extension of the plane failure, as predicted by the conceptual model discussed above, appears to affect the whole rock mass lying between the river and the slope crest, it becomes evident that the model should be constructed covering the entire rock volume of the slope. But such an approach will result in thousands of rock blocks in the model causing computational run time beyond practical

Table 4: Mean value of joint material properties and intact rock properties established by laboratory tests.

Properties	Beddings	Joints
Normal stiffness (MPa/m)	1870	1140
Shear stiffness (MPa/m)	647	675
Friction angle (degree)	24	26
Cohesion (MPa)	0.119	0.097
Dilation angle (degree)	10	8

Properties(intact rock)	Sandstone	Shale
Density (kg/m^3)	2700	2700
Young's modulus (MPa)	51012	11772
Poisson's ratio	0.17	0.14
Cohesion (MPa)	23.7	7.9
Friction angle (degrees)	60	50
UCS*(MPa)	196	42
Tensile strength (MPa)	13.7	6.6

*: Unconfined compressive strength

Fig. 9: A portion of the rock mass selected from the construction site to build a conceptual model of the excavated slope.

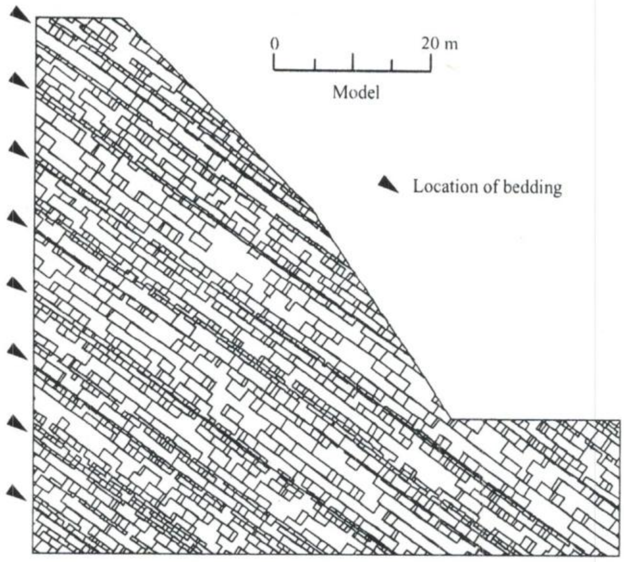


Fig. 10: DEM conceptual model of the slope.

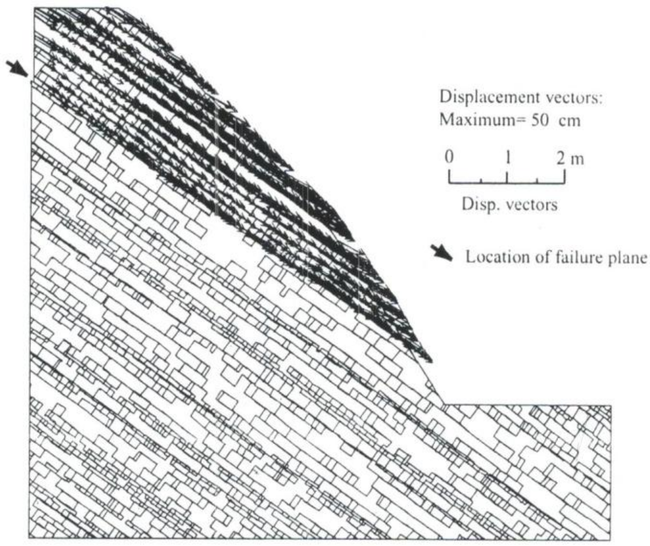


Fig. 11: Displacement vectors developed in a DEM model showing plane failure along a bedding plane.

value. It is necessary to develop some sort of procedure so that the number of rock blocks in the model can be reduced appropriately in order to complete the analysis within reasonable time.

Results obtained with the conceptual model have clearly revealed the plane type of failure. It has also predicted the location of the failure plane convincingly. Therefore, the procedure of reducing the number of rock blocks should be devised in such a way that these two basic features will also be reproduced with the new model. These features can be maintained only if the input data regarding the orientation of discontinuities and geometrical parameters, namely, persistence and spacing of bedding are not changed. But the values of persistence, gap length and spacing of joints (both sets) may be altered systematically as these parameters have no direct influence on mode and location of the failure for this particular case. This approach enlarges the size of the rock blocks and consequently decreases their number in the model.

Based on the above discussion, to avoid the long computational time required for a DEM analysis, the values of joint parameters, namely, persistence, spacing and gap length were increased by ten times their original values. But in the case of bedding planes, the original values were used because, as already mentioned above, they form critical discontinuity planes and changing their values may cause the lose of the essential geologic features, such as location of the failure plane. All the other input data, including the mechanical properties of discontinuities, were used as per their original values. Table 5 shows the discontinuities input data prepared to construct the final analytical model shown in Fig.12.

Regarding the effect of increasing block size on the response of the model, it is noted that weight forces depend on the block volume. Similarly, forces due to water pressure depend on the block side surface area; rock mass shear strength, and failure and deformation mechanisms of block systems depend on block size; engineering properties such as cavability, fragmentation characteristics and rock mass permeability also vary with discontinuity spacing (Giani 1992). These facts indicate that the approach of increasing block size should be pursued with great caution so as to ensure that the model satisfactorily captures those basic features that are important for the problem being investigated. Consequently, an attempt was made to compare the resulting mechanism of failure with that of the conceptual model (with its original block size). Since the analysis was performed in dry conditions, changes in permeability due to an increase in block size were not investigated.

As shown in Fig.13, the result of the analytical model is in agreement with that of the conceptual model shown in Fig.11. Therefore, a detailed analytical model covering the entire slope lying between the river and slope crest may be constructed using the input data shown in Table 5. This model is intended to carry out the stability analysis of the slope under various practical situations, as will be described later.

SLOPE STABILITY ANALYSIS

Geotechnical descriptions

As already mentioned above the site consists of an alternating sequence of sandstone and shale. Bedding planes are the only prominent discontinuities present in the shale. Besides bedding, two prominent sets of joints, one parallel and the other perpendicular to the bedding, occur in the sandstone. The Q-System of rock mass classification shows the Q-value ranging between 0.3 and 3.0. On this basis, the rock mass was assumed to have discontinuum behavior, and consequently, the DEM was employed for numerical analysis, as suggested by Grimstad and Barton (1993). Mechanical properties of discontinuities and intact rock required for the DEM analysis were established by laboratory test methods and the results are reproduced in Table 4. Since the bedding planes of the sandstone are characterised by

Table 5: Final input data for analytical model construction.

Parameters	Bedding		Joint set - I		Joint set - II	
	Mean	Standard deviation	Mean	Standard deviation	Mean	Standard deviation
Orientation (degree)	345/34	-	345/34	-	165/53	-
Trace length (m)	600	-	9.15	4.96	8.51	4.58
Spacing (m)	7	-	5.15	3.17	3.53	2.11
Gap length (m)	0	0	0.1765	0.01765	0.2575	0.02575

Orientation refers to dip direction/dip amount.

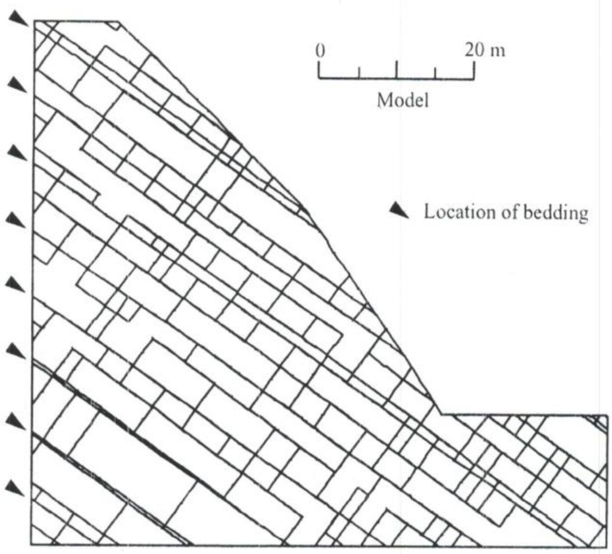


Fig. 12: DEM analytical model constructed by increasing the value of joint parameters (trace length, gap length and spacing) to ten times their original values. Beddings retain their original values.

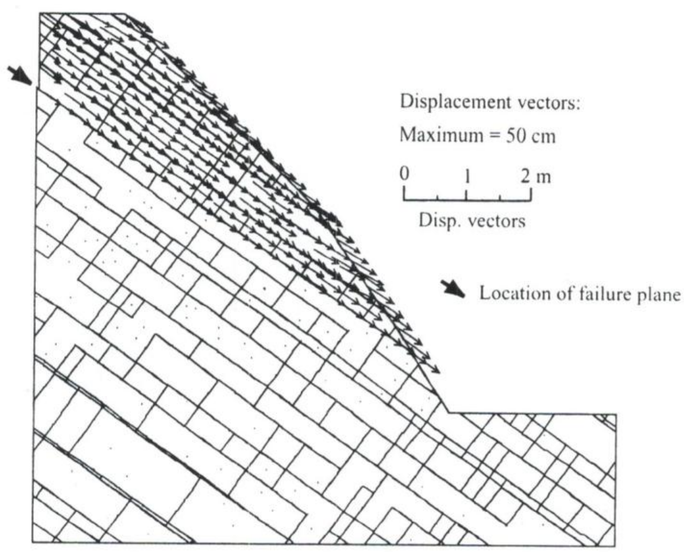


Fig. 13: DEM results showing plane failure along a bedding plane. The failure mechanism is identical with that of the model shown in Fig. 11.

the presence of a thin layer of shale, their mechanical properties were assumed to be the same as that of a bedding plane of shale.

DEM model construction

Table 6 shows the input data prepared as per the procedures discussed above for model construction. A DEM analytical model of the natural slope was then constructed as shown in Fig.14. The uppermost zone, shown in Fig.14, is a sandstone zone, which overlies a shale zone. Farther below is an alternating sequence of sandstone and shale zones of smaller thickness. Besides bedding planes, two sets of discontinuities were also generated in the sandstone zone. In case of the shale zone, only bedding planes were generated. No joints and bedding planes were generated in other underlying rocks because they are not exposed on the slope surface, and thus they do not affect the results of the analysis. In addition to the model of the natural slope, a DEM analytical model of the excavated slope was also constructed as shown in Fig.15. While running these models, the internal deformations of blocks (intact rock) were assumed to be governed by an elastic, isotropic constitutive model, whereas the Coulomb slip constitutive model was adopted for deformations along the joint contacts following the provisions available in UDEC (1996). The bottom of the model was considered to have a fixed boundary, whereas both its sides were modeled with roller boundaries.

Case analysis

Slope stability analysis was performed for three different situations: the existing situation intended to examine the stability of the natural slope, the excavation situation designed to understand the stability after making the excavation of the slope for the dam foundation and a rock reinforcement situation considered to examine the effectiveness of cablebolts proposed to stabilise the excavated slope. The existing situation attempts to model the entire volume of the rock mass lying between the river and the slope crest

Table 6: Input data prepared for DEM analytical model construction.

Parameters	Bedding planes		Joint set - I		Joint set - II	
	Mean	Standard deviation	Mean	Standard deviation	Mean	Standard deviation
Orientation (degree)	345/34	-	345/34	-	165/53	-
Trace length (m)	600	-	9.15	4.96	8.51	4.58
Spacing (m)	7	-	5.15	3.17	3.53	2.11
Gap length (m)	0	0	0.1765	0.01765	0.2575	0.02575

Orientation refers to dip direction/dip amount.

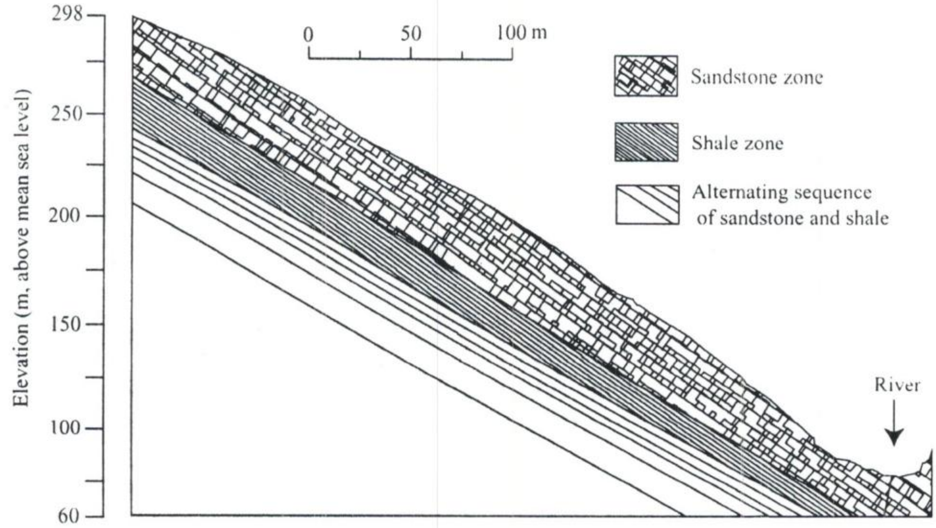


Fig. 14: DEM analytical model of the natural slope.

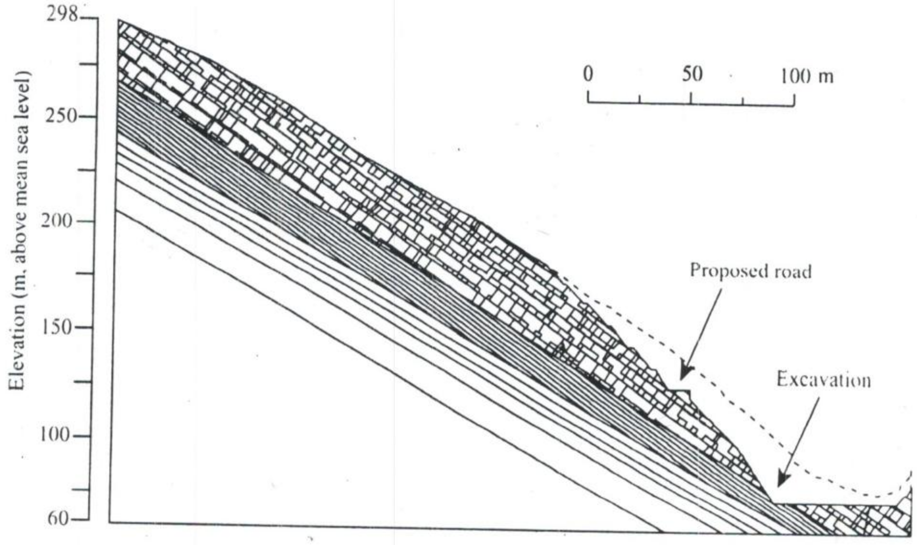


Fig. 15: DEM analytical model of the slope after excavation.

(Fig. 14) along a representative vertical cross-section of the natural slope. The excavation situation simulates step-by-step excavation creating benches with slope height and slope angle of 12 m and 45°, respectively. Near the reservoir, the slope is designed to have higher gradient (55°) with a gentler slope of 45° upslope (Fig. 15). The reinforcement situation is identical to the excavation situation with the addition 10 m long cablebolts with vertical spacing of 2 m in the lower part and 4 m in the upper part of the slope. The length and spacing of the cablebolts were determined by taking into account the field situation and current design techniques of rock anchoring. Table 7 shows the cablebolt and grout properties used during DEM analysis.

RESULTS AND DISCUSSION

After achieving an equilibrium condition for the model of the natural slope as indicated by the constant values of the x- and y- components of displacement (Fig. 16) at various elevations of the slope face, excavations (Fig. 15) were made in stages starting from the uppermost bench and simulations were also performed accordingly. As a response to the last stage of excavation, the model indicates that the plane (shear) failure first initiates at the contact of the sandstone and shale zones exposed near the river and then follows a bedding plane of sandstone nearly up to the middle part of the model as indicated by the shear displacement shown in Fig. 17. Afterwards, it grows along joints oriented perpendicular to the bedding planes of sandstone and finally approaches the slope face. Instabilities at various elevations of the excavated slope are also visible from Fig. 18 in which continuously increasing deformations with increasing calculation steps are clear. In response to the reinforcement situation, the model predicts stable slope conditions as indicated by the constant values of deformation (Fig. 19) predicted at various positions of the slope and also by the constant values of displacement vectors (Fig. 20) developed during the model run.

Such results must be understood as consequences of the assumptions that the major influential discontinuities were adequately represented in the model, and that the mechanical properties of the discontinuities were correctly established by the laboratory tests. On the other hand, engineering judgment is very much needed to justify the predicted location of the failure plane in the upper part of the slope as it may be influenced by the location of the discontinuity.

It is worthy to note that the entire joint network may not be visible in the field and the small exposure available for data collection and for comparison of the analytically generated joint pattern may not be representative of the entire area of interest. Similarly, it should be noted that there is a danger of overlooking important individual or rare discontinuities as a result of contouring orientation data as a means of establishing representative values of joint sets (Hencher, 1985). Besides, the subjective survey of only those discontinuities considered to be important for the project needs good, relevant exposures, which may not be available in some circumstances. In the same way, while reducing the number of rock blocks in a model by enlarging the input values of a joint, the position of discontinuities, as observed in the field, may not be reproduced in the model. This situation may give rise to errors in the predicted location of the failure plane. Therefore, these points should be considered fully before adopting the model for a final analysis. Alternatively, several parametric analyses should be made to verify the response of the model with well-established engineering judgment regarding the site of investigation. Final analysis should be pursued after the model results converge with the field situations satisfactorily.

Table 7: Cablebolt and grout properties employed during DEM analysis.

Cablebolt properties	Diameter	Mass density	Elastic modulus	Tensile yield force
	35.7 mm	7500 kg/m^3	100 GPa	10 MN

Grout properties	Thickness	Shear strength	Shear stiffness
	6.3 mm	4.0 MN/m	1000 MN/m/m

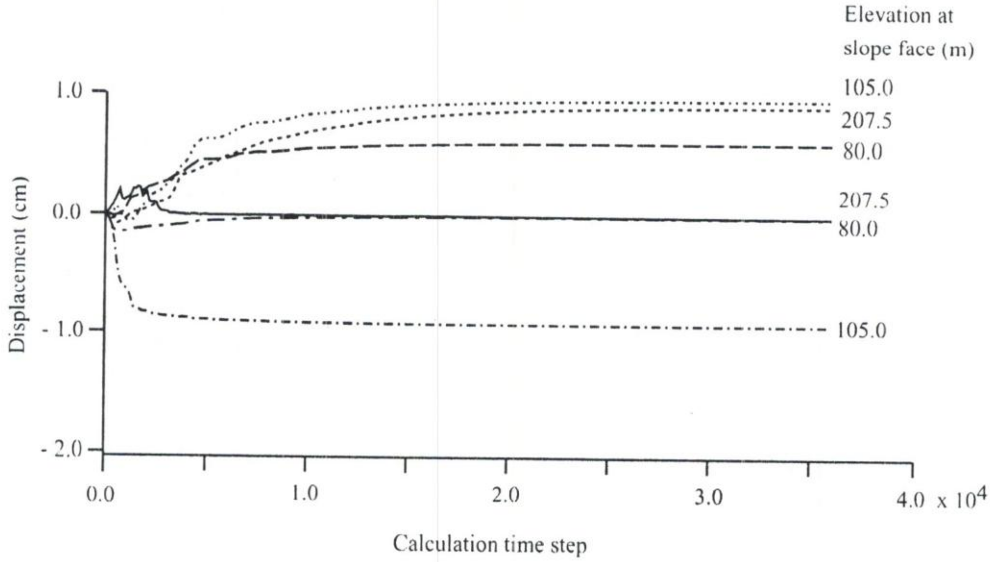


Fig. 16: Predicted x- and y-components of displacement at various elevations of the natural slope face. Positive values refers to x-component, negative values refer to y-component.

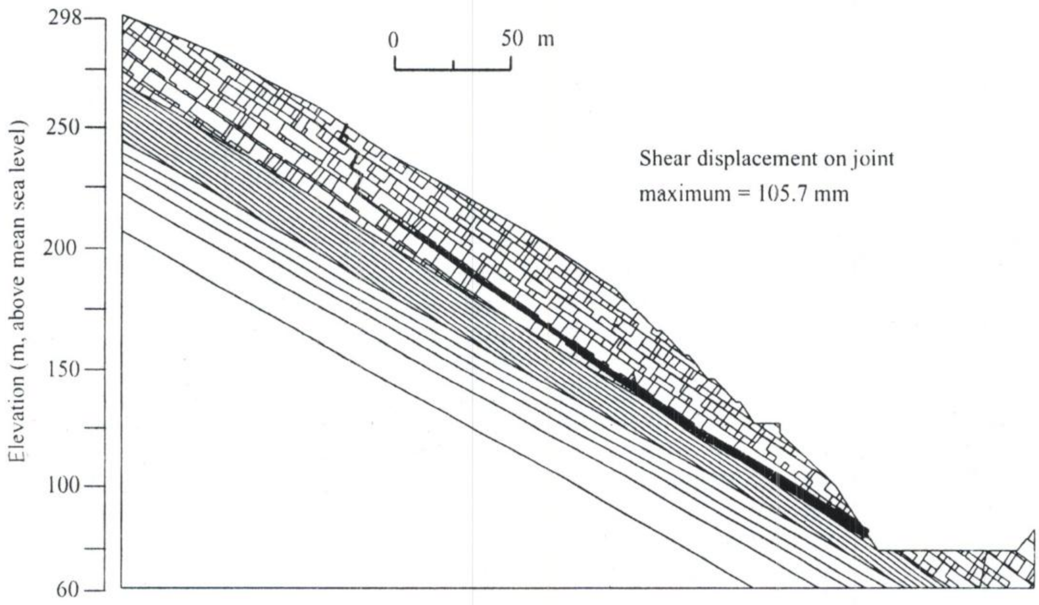


Fig. 17: Predicted shear displacement (thick black line) along the joints.

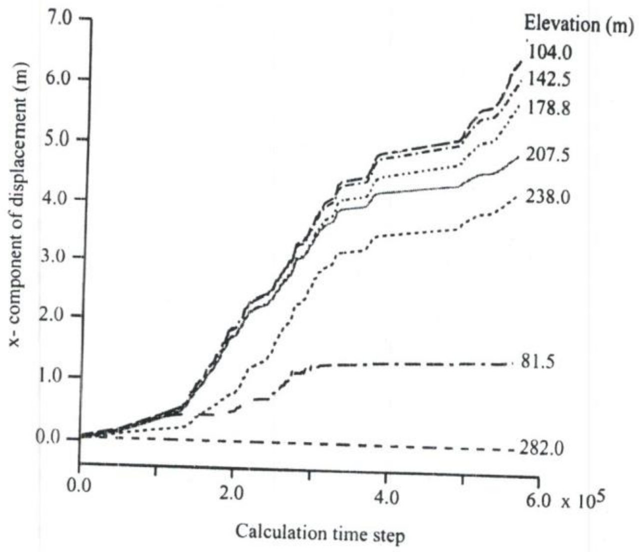


Fig. 18: Predicted x-component of displacement at various elevations of the slope after excavation.

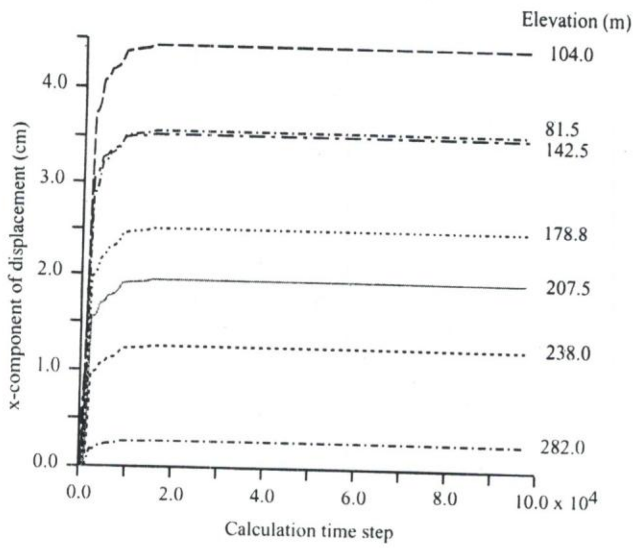


Fig. 19: Predicted x-component of displacement at various elevations of the slope after applying cablebolt reinforcement.

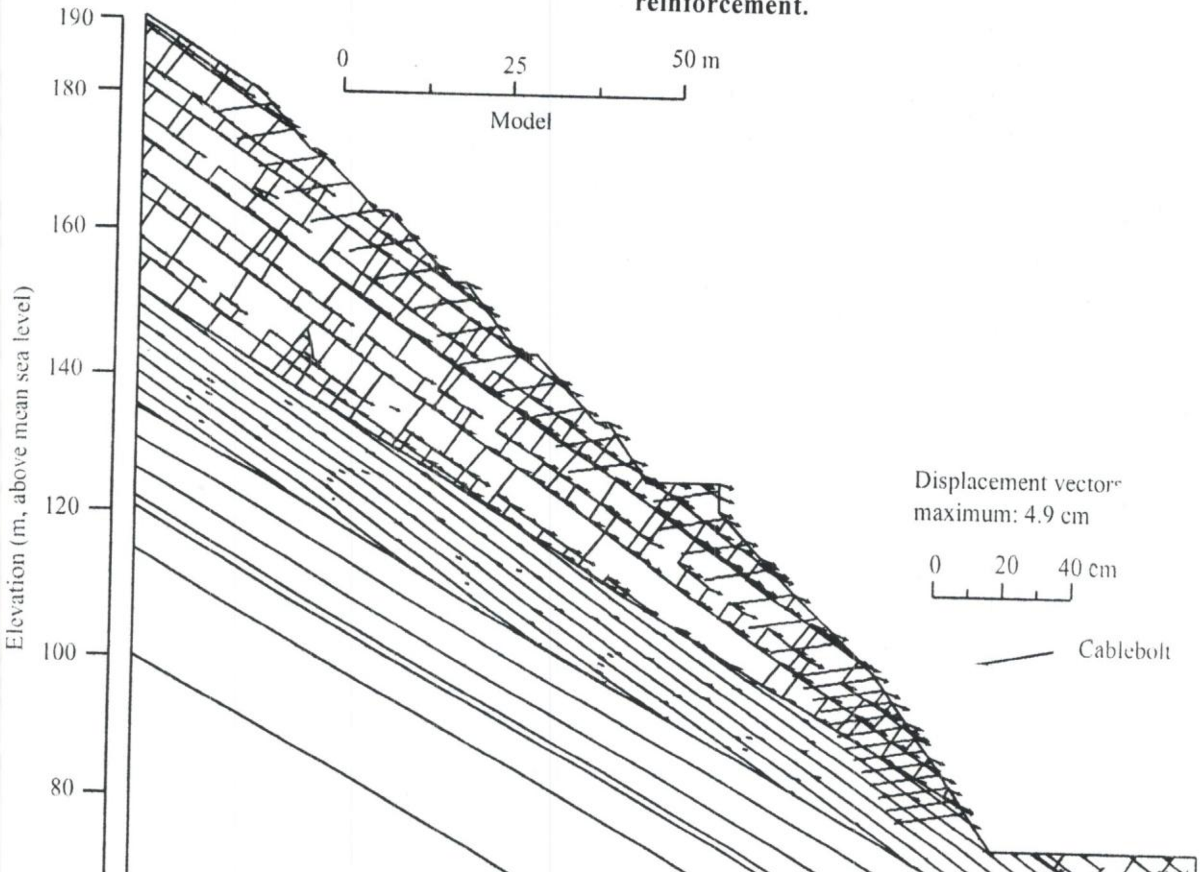


Fig. 20: Displacement vectors developed in a DEM model after applying the cablebolt reinforcement in the excavated slope.

CONCLUSIONS

In this study, a process of DEM model construction is discussed that starts with characterizing the geology of the site followed by identifying statistically homogeneous region for further joint measurements. In each homogeneous region, joints are then observed subjectively in order to quantify orientation, spacing and trace length for each set occurring in the rock mass. In addition, a method of establishing the value of gap length between joint segments is also proposed based on the values of statistically analyzed field data of trace length and spacing of the joint to establish the joint geometry in a rock mass.

The joint geometry predicted by the proposed method closely resembles the field situation. The DEM conceptual model, the input data for which were established by using the proposed methods, has also been found useful in gaining some insight regarding the extent and mode of failure of the slope. Based on parametric studies, the input data for the analytical model, with regard to joint persistence, gap length and spacing, wee increased by ten times their original values. The results of the analytical model convincingly match those of the conceptual model. The results obtained with the analytical model are also comparable with that of the past slope failure cases existing nearby to the investigated site.

The result of the slope stability analysis carried out using the DEM analytical models reveals that the investigated slope is prone to plane failure during an excavation for a dam foundation. The analysis also indicates that the excavated slope may be stabilised by installing 10 m long cablebolts with vertical spacing of 2 m. Based on these results, Kyushu Electric Power Company is carrying out construction.

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