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Numerical probabilistic analysis for slope stability in fractured rock masses using DFN-DEM approach

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ABSTRACT

Due to the uncertainties in input geometrical properties of fractures, there is no unique solution for assessing the stability of slopes in jointed rock masses. Therefore, the necessity of applying probabilistic analysis is inevitable on these cases. In this study, a probabilistic analysis procedure along with relevant algorithms were developed using Discrete Fracture Network-Distinct Element Method (DFN-DEM) approach. In the right abutment of Karun 4 dam and downstream, five joint sets and one major joint were identified. According to the geometrical properties of fractures in the Karun river valley, instability situations seemed applicable on this abutment. In order to evaluate the stability of a rock slope, different combinations of joint set geometrical parameters were selected, and a series of numerical DEM simulations were performed on generated and validated DFN models in DFN-DEM approach to measure minimum required support patterns in dry and saturated conditions. Results indicate that the distribution of required bolt length was well fitted with a lognormal distribution in both circumstances. In dry conditions, the calculated mean value was 1125.3 m, and more than 80 percent of models needed only 1614.99 m of bolts which was equivalent to a bolt pattern of 2 m spacing and 12 m length. However, as for the slopes with saturated condition, the calculated mean value was 1821.8 m, and more than 80 percent of models needed only 2653.49 m of bolts which was equivalent to a bolt pattern of 15 m length and 1.5 m spacing. Comparing the obtained results with that of numerical and empirical methods show that the investigation of a slope stability with different DFN realizations which were conducted in different block patterns was more efficient than the empirical methods.

Keywords : *Rock slope; Discrete fracture network; Probabilistic analysis; Discrete element method; Monte Carlo simulation.*

1. Introduction

The Stability of natural and excavated rock slopes has always been of great concern in rock engineering. What makes these analyses challenging is the existence of some sources of uncertainty. This uncertainty arises from the inherent variability and insufficient information concerning the site conditions and incomplete understanding or simplification of a failure mechanism [1]. Uncertainty in rock slope engineering may occur as scattered values for geometrical properties of discontinuities such as orientation, persistence and spacing as well as laboratory or in situ test results [2]. Einstein (2003) has provided an excellent overview of sources of uncertainty in rock engineering [3]. To tackle the issues pertaining to uncertainty, probabilistic analyses were introduced as efficient tools to quantify and model variability and uncertainty [2]. In this regard, various probabilistic studies of rock slopes in civil and mining engineering applications have been conducted by a number of scholars [2-15].
The approaches used in the modelling process of probabilistic studies include kinematic, experimental, probabilistic, limit equilibrium, key block theory and numerical methods. Numerical methods are capable of modelling the main features such as faults, joints, fractures as well as ground water conditions. Besides, they can offer more realistic approximations on slope behavior compared to analytical models. Indeed, non-numerical approaches including analytical, physical or limit equilibrium methods may not be suitable due to presumed simplifications which can lead to conservative results in most cases.

Numerical methods, which are employed for stability analysis of rock slopes, consist of continuous, discontinuous and hybrid modellings [16]. Among the available discontinuous numerical methods, the distinct element method (DEM) is considered as an efficient tool to model the discontinuous behavior of rock masses. The DEM has been mainly employed in academic studies. Thus, further experience in the application of this method to practical case studies is required so as to specifically understand how to decide upon the input data and modeling parameters for the DEM analysis [18]. Discrete Fracture Network (DFN) method, the forces acting on blocks and the designed support pattern are also estimated using Distinct Element Method (DEM) approach. In DFN modelling, the fracture system geometry is based on stochastic representation by the probabilistic density functions of fracture parameters (e. g. orientation, size, location and aperture). This system is formulated according to the field mapping results, and it reduces the inherent uncertainty of fracture networks in jointed rock mass which is based on Monte Carlo simulations. By this approach, discontinuity systems are generated according to probability density functions of discontinuity parameters such as joint persistence, orientation and aperture. In this regard, Bhasin and Kaynia utilized DEM to estimate the potential volume of detached rocks as a result of a large failure in a rock slope in Norway with 700 m in height [19]. Kveldsvik et al. employed Discrete Element Method for dynamic analysis of a rock slope in Norway with 800m in height [20]. In another

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study, Curtis et al. used Discrete Element Method in order to dynamically analyze an arc Dam [21]. By taking advantage of 3DEC software, Corkum and Martin investigated the effect of slope stabilization on the confining wall adjacent to Revelstoke Dam in Canada [22]. Bonilla-Sierra et al. used photogrammetric data and DFN-DEM method to assess the stability of a slope in France by main joint sets [23]. According to aforementioned subjects, the significance of DFN-DEM methods in slope stability analysis is undeniable in a wide variety of cases.

The main objective of this research is to present an algorithm for probabilistic analysis of rock slope stability and designing sufficient reinforcements based on discontinuity parameters. To fulfil this objective, a solution with high accuracy and low uncertainty needs to be developed for stability analysis of rock slopes in large scale. Therefore, by considering the potential failure in right abutment and downstream of Karun 4 dam, dam site data are used for DFN-DEM. Having carried out the statistical studies on joint properties mapped from Karun 4 Dam structure, the discrete fracture network was created. Subsequently, several different models from a DFN with similar rock mass features were analyzed by UDEC as a two dimensional DEM software. Lastly, the required bolting pattern was acquired in order to evaluate the slope behavior in saturated and dry conditions using probabilistic analysis.

2. Methodology

In this research, after introduction of Karun 4 Dam site and its geological features, a joint study was conducted. Suitable distribution functions of joint persistence, dip angle, orientation and aperture were fitted to generate discrete fracture networks. Afterwards, several models from fracture networks with mentioned characteristics were generated by FracIUT2D code.

2.1. Geological description of the area

Karun 4 double-curvature arch dam is a national Iranian project located 180 kilometer south-west of Shahrekord City and 4 kilometer downstream of Armand and Bazoft Rivers intersection in Charmahal and Bakhtyari Province. The geographical coordinate of dam is 31º 35' 53" N and 50º 24' 05" E (Fig. 1(a)). The dam valley is U-shaped (Fig. 1(b)) and the abutments are mostly striped with approximate dip angles of 70° to 80°. Old and new alluviums are spread out in the area. The new alluvium contains rock debris, slope wash, residual soils as well as new and old alluvium. The size of these aggregates vary from very fine particles (clay) to boulders (mostly limestone). In the right abutment of Karun 4 dam and in the downstream of the dam body, five joint sets and one major joint have been identified. Based on geometrical properties (Dip and Dip Direction) and their influence zones on the Karun river valley, they can create a potentially unstable condition. The height of mentioned abutment is 185 m and it has a dip direction of 132 degrees. Fig. 1(c) provides a clear visual description of the structure. According to permeability test results, the rock mass in this region is in the range of middle to highly permeable, and this condition is extended to the depth of 100 meters.

2.2. The probabilistic method

Variation in geometrical parameters of joints is an abvious fact, and it is very hard to find a representative deterministic value for them. Therefore, in order to overcome the deficiency of deterministic methods, several researchers have used probability theories to quantify uncertainties in rock properties (orientation and fracture persistence). The available probabilistic methods can be divided into two categories: analytical methods and simulation techniques. Analytical methods are based on closed-form expressions of the main descriptors, such as mean and standard deviation of random variables. Since many assumptions need to be made in formulation, these methods are very difficult to be used especially when the function is algebraically complex and nonlinear. Therefore, simulation techniques are more widely preferred [24]. Among them, Monte Carlo simulation in its simplest form is a random number generator that is useful for forecasting, estimation, and risk analysis. A simulation calculates numerous scenarios of a model by repeatedly picking values from a user-predefined probability distribution for the uncertain variables and using those values as input data for a model [25]. Indeed, Monte Carlo simulation is a stochastic process which addresses the 'randomness' of the fracture network geometry by representing the fracture properties including location, size, orientation, and aperture as random variables. These random variables are followed by their own specific PDFs after assuming the shapes of fractures and calculating the densities of all sets of fractures. The aim of the simulation is to generate a large number of realizations of fracture systems, each of which corresponding a particular set of individual random variables for the locations (of fracture geometric centers), orientations, size, and apertures generated according to their specific PDFs [26]. This simulation is attractive in concept because it reduces the uncertainties induced by the largely unknown fracture system geometry and improves the quantification of variability of the properties. However, it also requires much more time and resource consumptions in view of the computational requirements given that a large number of realizations must be generated and used as the geometric models supporting the numerical modeling [26].

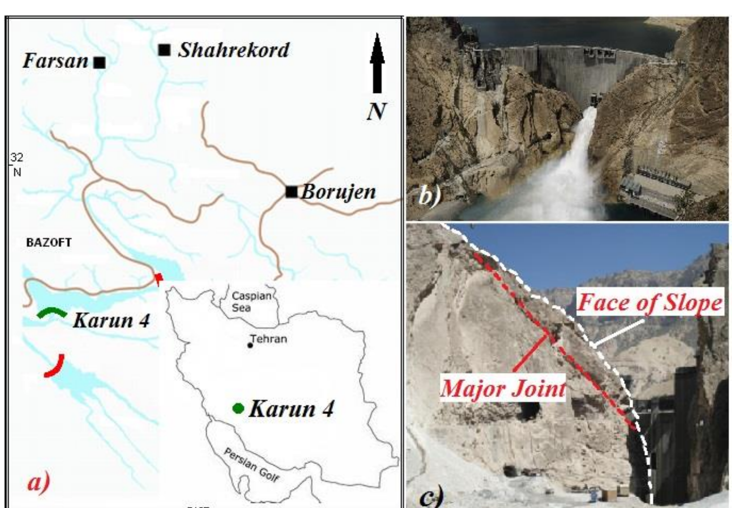

Fig. 1. (a) location of Karun 4 Dam, (b) Karun 4 Dam, (c) The visual description of site in right abutment of Karun 4 dam.

In this study, the joint condition was simulated by DFN method using FracIUT^{2D} code [27]. The imported data for generating the 2D Discrete Fracture Network are as follows:

- Probability Density Function (PDF) of joint orientations (Joints Fisher constant)
- Probability Density Function (PDF) of joint persistence
- 2D Density function of each joint set

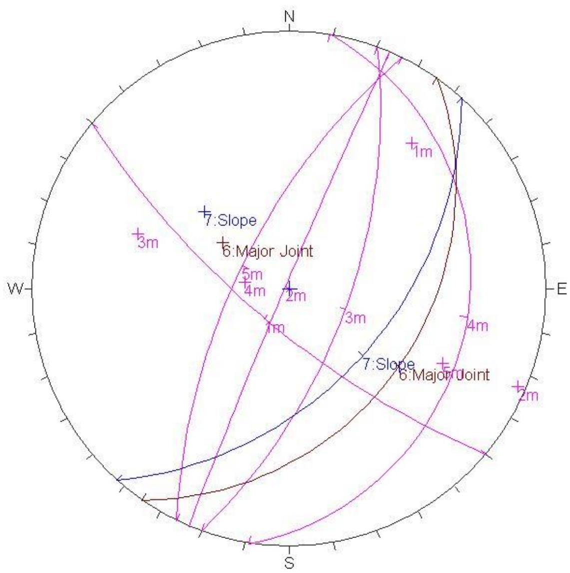

Fig. 2. (a) location of Karun

A total number of 7265 joints were mapped from right abutment of Karun 4 by using 1000 window mappings. Having identified the mapped joints, they were drawn in Stereonet (as shown in Fig. 2), in each case the best distribution was fitted to the existing data. As a result, the corresponding PDFs were determined. Subsequently, the parameters obtained from PDFs were inserted in the DFN. As can be seen in Fig. 2, five main joint sets are discerned according to obtained data. Among them, 4 joint sets are considered as fractures (joint set 2, 3, 4 and 5) and the other one (joint set 1) is regarded as a bedding plane. There is a very large joint in the right abutment of dam which its geometrical properties (dip and dip direction) and its influence zone on the Karun River valley may trigger instability in the mentioned abutment. This joint is displayed in Fig. 2 as plane 6. Plane 7 represents the abutment.

Then, the log-normal function was fitted to joint persistence (σ and μ are the first moment (i.e. Mean) and the second moment (i.e. Variance)), and afterwards, the persistence of 5 main joint sets were estimated which are provided in Table 1. Thereafter, using the data acquired from joint mappings from window mapping method, the distribution function of geometrical parameters of all joint sets were determined as shown in Table 1.

Once the required parameters were determined, FracIUT^{2D} code was used to generate a large number of DFNs (500 DFNs). Then realizations with different sizes were generated, and in order to avoid the boundary effect, sufficiently large parent fracture networks were initially generated and smaller fracture networks were excluded from the large model. Since simulation of a large number of stochastic DFN models in field scales demands heavy computations and is very time consuming, the volume of blocks placed over the major joint is measured and two extreme cases with small and large block sizes are selected for static analysis. In this case, the effect of blocks size on stability can also be investigated.

Fig. 3 shows the DFN models after eliminating the dead-end fractures. DFN1 model contains relatively large sized blocks and DFN2 is consisted of small sized blocks. Each model contains 5000 number of fractures and an approximate number of 7000-10000 rock blocks.

According to Fig. 3, the small and large blocks are the ones positioned on the joint surface. Histogram and cumulative frequency diagrams were used to compare the dimension and area of formed blocks in the two DFNs which are shown in Fig. 4. As can be seen in Fig. 4(a), variation of block area in DFN1 (see Fig. 3(a)) is in a range of 0 to 300 m^2, whereas for DFN2 (see Fig. 3(b)) it is between 0 to 200 m^2. Additionally, the cumulative frequency of blocks area in Fig. 3(a) and Fig. 3(b) are applied for comparing the dimensions of blocks lying on the main joint set as diagrams shown in Fig. 4(c). It can be concluded that the blocks in Fig. 3(a) are generally larger than those in Fig. 3(b).

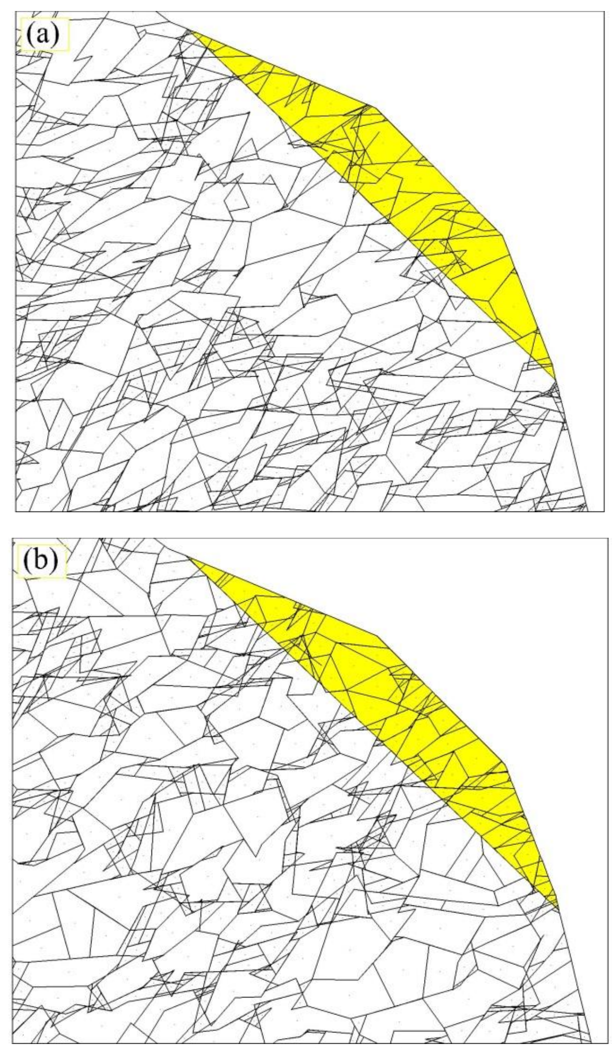

Fig. 3. Two DFN models with small and large block sizes over major joint. (a) DFN1 with relatively large block size, (b) DFN2 with small block size.

Table 1. Input geometrical parameters of fracture sets in this study

Orientation set (Dip/Dip Direction)					
	73/220	88/293	64/110	20/099	67/296
Coefficient, k for Fisher Distribution					
	61.2592	36.243	57.1631	8.77979	81.9405
Intensity (P_{20})					
	0.031074	0.037533	0.036694	0.093719	0.031933
Length Distribution function parameters					
Distribution	Lognormal	Normal	Lognormal	Lognormal	Lognormal
σ	0.65345	3.6174	0.70692	0.65949	0.63016
μ	2.4125	8.0536	1.8732	1.9827	1.8178
L_{min} (m)	1.8	2.18	1.3	0.8	1.6
L_{max} (m)	52.5	24.1	31.6	61.7	16.1
Length statistical parameters					
Mean	13.622	8.0536	8.2326	8.9961	7.3342
Std. Dev (m)	8.9758	3.6174	5.6237	6.7618	3.9483

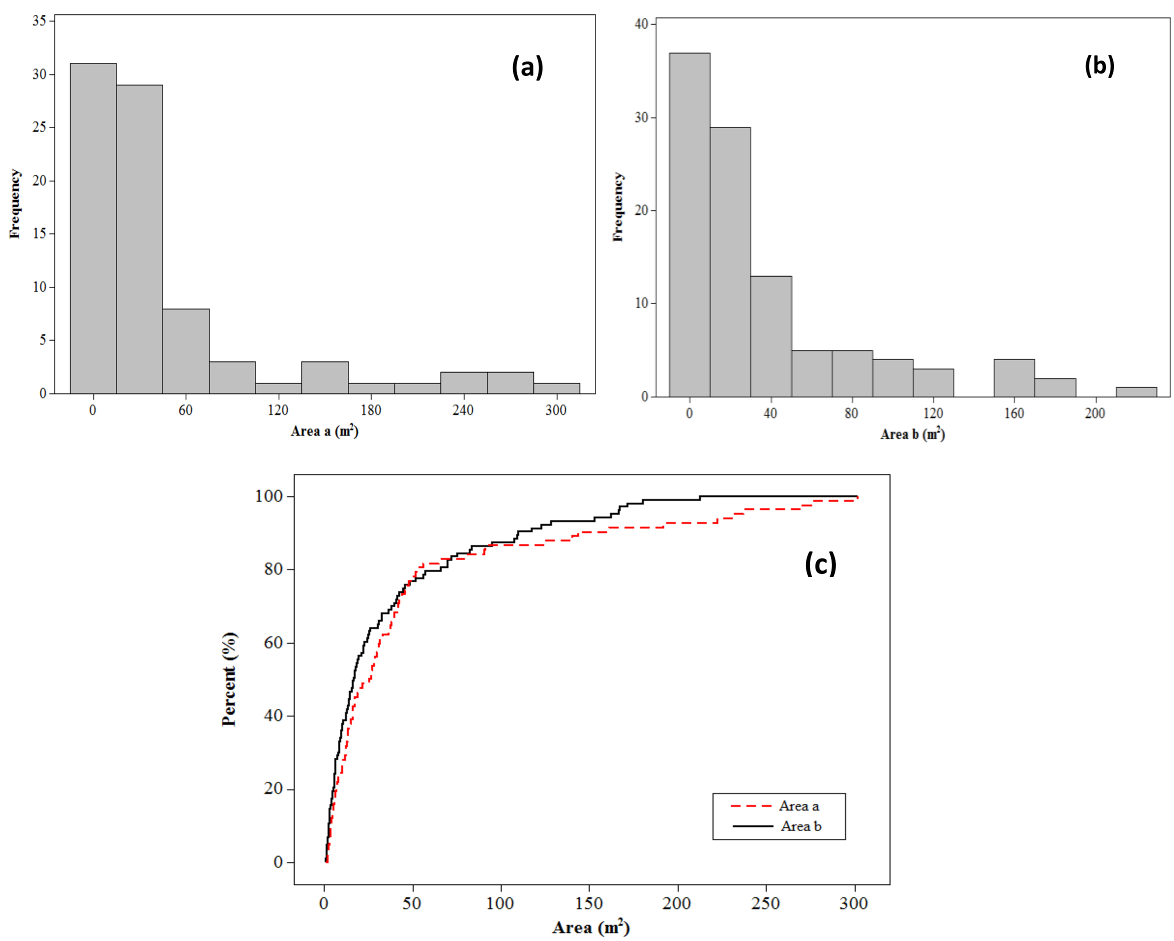

Fig. 4. Distribution of blocks area in DFN1 (a), Distribution of blocks area in DFN2 (b), and Cumulative distribution of block area of both DFN models (c).

2.3. Physical and Mechanical Properties of Rock Mass

According to initial observations, the rock mass lithology is identified as limestone. The input mechanical properties of intact rock and fractures are respectively presented in Tables 2 and 3, and the Mohr-Coulomb criterion was presumed for stability analysis by UDEC. The minimum and maximum apertures are shown in Table 3. Vertical and horizontal gravitational stresses were considered as in-situ stress in the whole model. The ratio of horizontal to vertical stresses (K0) is 0.7.

Table 2. Mechanical properties of intact rock [28].

Intact rock	
Density (Kg/m3)	2500
Poisson's ratio	0.25
Elastic modulus (Gpa)	24
Friction angle (degree)	62
Cohesion (Mpa)	4.4
Tensile strength (Mpa)	10

Table 2. Mechanical properties of intact rock [28].

Parameter	Joint set 1 (bedding)	Joint sets 2,3,4,5	Major joint
Normal stiffness (Mpa/m)	2500	4000	3400
Shear stiffness (Mpa/m)	800	1200	930
Residual friction angle (degree)	35	40	43
Cohesion (Mpa)	0.15	0.35	0.15
Joint permeability (Pa-1 Sec-1)	83	83	83
Residual aperture at high stress (mm)	1.5	0.2	1
Aperture for zero normal stress (mm)	7.5	1	5

Table 3. Mechanical properties of fractures [28].

Parameter	Joint set 1 (bedding)	Joint sets 2,3,4,5	Major joint
Normal stiffness (Mpa/m)	2500	4000	3400
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Joint permeability (Pa^{-1} Sec^{-1})	83	83	83
Residual aperture at high stress (mm)	1.5	0.2	1
Aperture for zero normal stress (mm)	7.5	1	5

3. Numerical Analysis Process

In this section, the 2D numerical static stability analyses from right abutment of Karun 4 dam in dry and saturated conditions are conducted using UDEC. Then according to the displacements recorded from static stability analyses, sufficient support systems were suggested in order to avoid potential failures. To fulfill this objective, shotcrete and rock bolt patterns were applied as support systems to prevent rock block failures. The required bolt length for dry and saturated conditions was investigated through statistical methods, and at the end of the process the optimum length of bolts as well as bolt spacing were introduced through different bolting patterns.

3.1. Model establishment

Since the abutment is extended longitudinally, plane strain presumption is valid in two dimensional modelling for sections being perpendicular to it. UDEC was employed for assessing the stability of right abutment in the discontinuous media. The DFN was assigned to the DEM model as the geometric basis for the block stability analysis using UDEC code. By taking advantage of the defined equation of motion for rigid body analysis, the displacement and movement of blocks were estimated [33]. As illustrated in Fig. 5, the roller boundary conditions were assumed along the lateral sides of the model so that no displacement was allowed in x-direction. Additionally, at the bottom of the model, the boundary was fixed in a way that no movement was allowed in x and y directions.

In order to model the dry conditions as well as conditions indicating presence of ground water and suggest appropriate support systems, 15 models were selected randomly among 500 generated DFNs. Consequently, the analysis was conducted on 15 DFNs.

In an analysis on various DFNs, it was deduced that some blocks with large volumes which are usually placed on major joints may potentially lead to instability and consequently trigger risks in the long term. In this regard, 10 points are defined in Fig. 6 that record displacements in x and y directions on the slope face and the major joint.

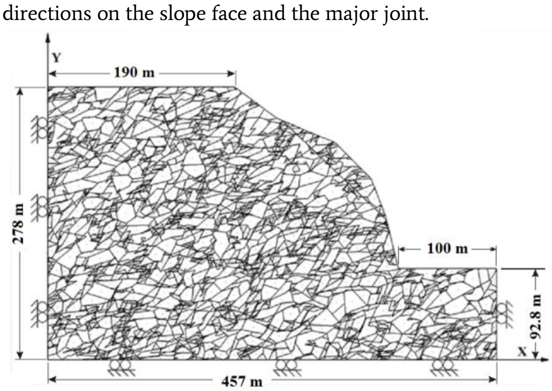


Fig. 5 Boundary condition in static analysis.

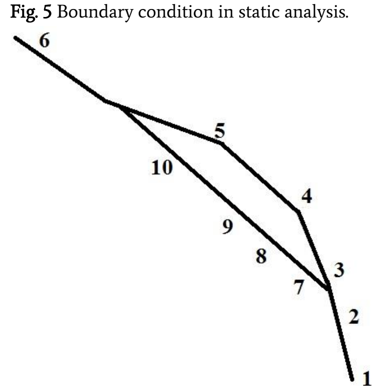


Fig. 6 Monitoring points on slope.

3.2. DFN-DEM analysis in dry conditions

In both DEM models, the formed blocks on major joint (6) are potentially unstable. Figs 7 and 8 show blocks movements in DEM models for DFN1 and DFN2, respectively. The red arrows show the displacement vectors, and the boundaries of blocks are indicated by black lines. In these figures, the maximum displacement of blocks on major joints are 4.91 cm and 4.7 cm for DFN1 and DFN2, respectively. These values are obviously greater than the measured displacements of other blocks in both models.

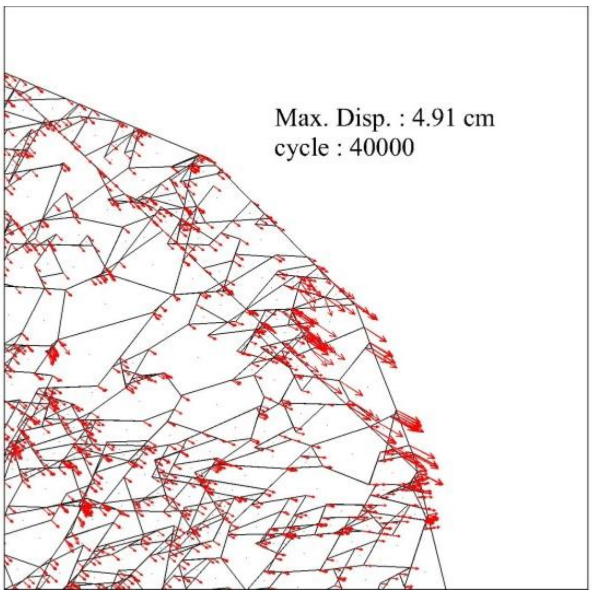


Fig. 7. Displacement vectors of DFN1 model.

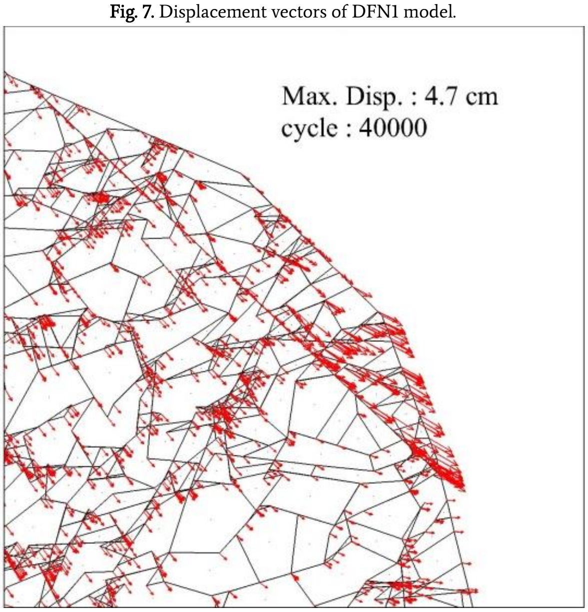


Fig. 8. Displacement vectors of DFN2 model.

Fig. 9 shows displacements pertaining to control points defined in Fig. 6 for DFN1. It can be clearly seen that some regions of major joint (points 3, 4, 5 on the slope surface and points 7 and 8 on the major joint) still have potential to move (gradient of displacement line is positive), while other regions have a constant movement. Furthermore, the local slides of the blocks have been recorded during construction by monitoring the displacements.

The measured displacements are about 5 cm, which clearly demonstrates the validity of the proposed static analysis method.

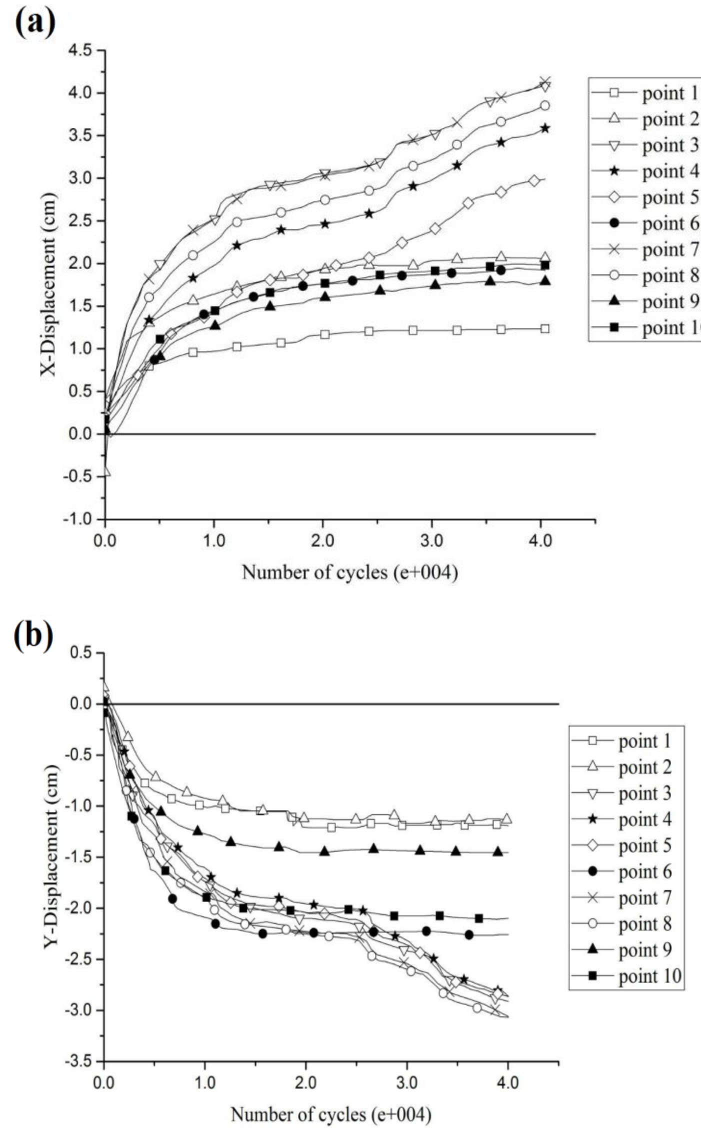


Fig. 9. Defining the control points on the rock slope for DFN1 (a) X-displacement; (b) Y-displacement.

3.3. DFN-DEM analysis with ground water condition

Due to shear strength reduction, water pressure threatens the slope stability. High water content also increases the weight of rock mass. As mentioned earlier, joints aperture is an important factor affecting the permeability of rock mass joints. Fluid flow in plate-shaped connection regions is considered as laminar viscous flow so that the flow rate has a direct relationship with the third order of aperture. In this method, the fluid flow is determined using the pressure difference between adjacent domains [31]. A sample of hydro-mechanical analysis by the aforementioned approach is performed by Bonzanigo [34] where the effect of draining on rock mass stability was studied. According to piezometric wells, the water level varied between 120 m to 100 m depth in right and left sides of abutment, respectively. Fig. 10 shows the surface of water table in the right abutment of Karun 4 dam.

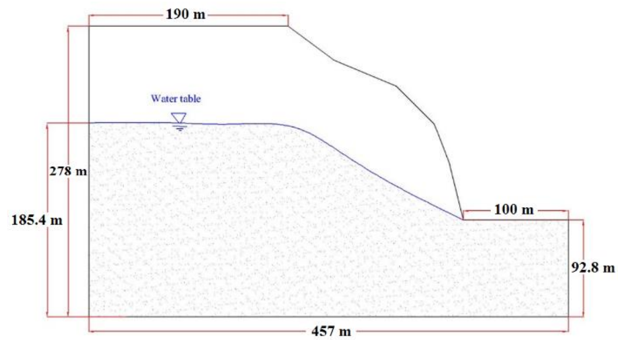
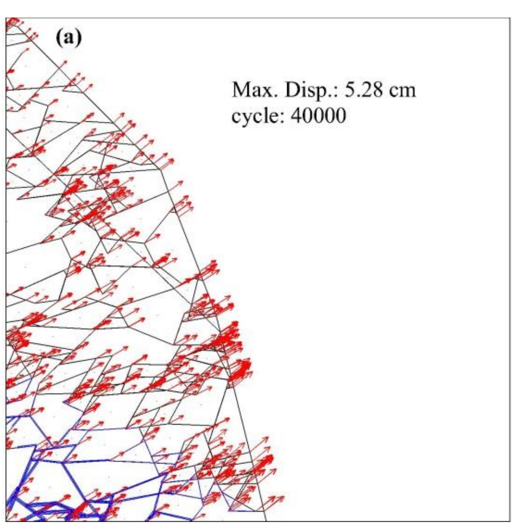


Fig. 10. Ground water table in the right abutment of Karun 4 dam.

Two methods are often employed to specify pore pressure distributions within slopes. The most rigorous method is to perform a complete flow analysis using resultant pore pressures in the stability analyses, while the most commonly used method is to specify a water table, and the resulting pore pressures are given by the vertical depth below water table. In this study, the second method was applied owing to the fact that the water table approach was identical to specifying a piezometric surface according to reference [35]. The results of analysis on slope with ground water conditions show that the maximum displacement are approximately 0.5 cm bigger than displacement of dry conditions, but the blocks around the toe of slope suffer larger displacements compared to other blocks due to the pore pressure inside discontinuities in the mentioned area. These vectors are inclined to upward directions (Fig. 11), whereas in dry condition they are conversely inclined to downward directions. The maximum measured displacement for blocks around the toe of slope is equal to 5.28 cm and 5.24 cm for DFN1 (Fig. 11(a)) and DFN2 (Fig. 11(b)), respectively. Fig. 12(a) demonstrates the slightly downward inclination of blocks in dry condition, however the overall direction of displacement vectors in saturated condition were upward as shown in Fig. 12(b). Reduction of effective stress and upward movement of blocks cause easier sliding and higher probability of local instability. This is mainly due to the ground water pressure acting on the blocks in upward direction. For instance, a comparison was made for dry and saturated cases in Fig. 11. As can be seen in Fig. 11(a), displacements have a somewhat downward inclination in dry condition, while for the saturated case, Fig. 11(b) indicates that the water pressure in discontinuities under blocks is higher and leads to an increase in upward displacements. It can be inferred that if the desired structure is below the water table or it is just prone to saturation, the risks associated with potential instability is high, as well.



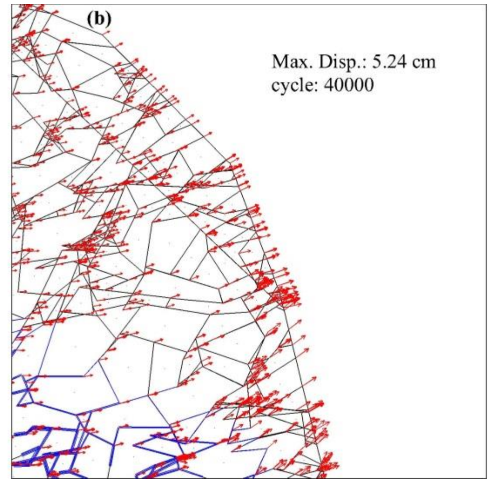


Fig. 11. Pattern of displacement vectors with ground water condition in DFN1 (a) and DFN2 (b) models.

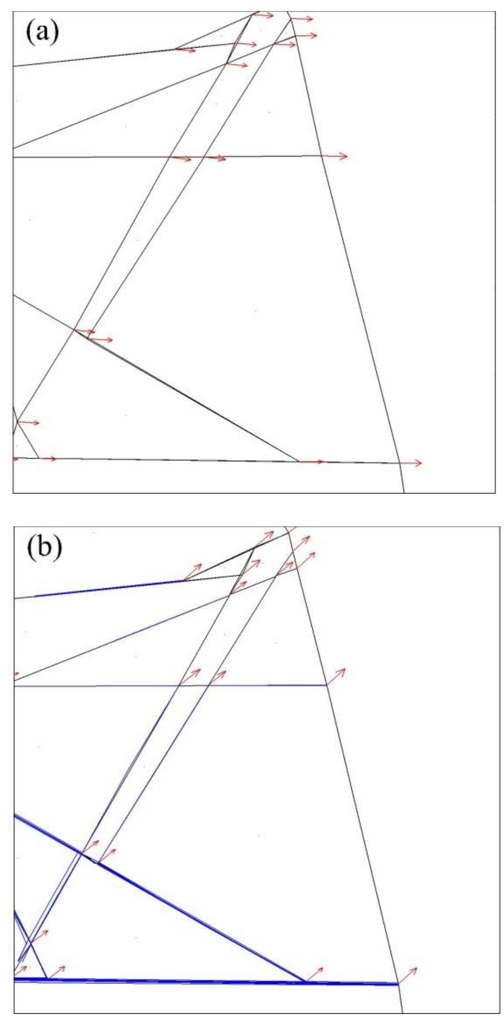


Fig. 12. The direction of displacement vectors in dry (a) and with water table (b) conditions.

3.4. Investigation of slope stability using probabilistic approach

Shotcrete and bolts are utilized as the support system for right abutment of Karun 4 Dam. In this procedure, different bolt patterns were determined in dry and saturated conditions. The applied bolts and shotcrete properties are inserted in table 4, and bolts are of class A-III steel. Fig. 13 shows a proposed bolt pattern around the slope face. The sequence of support design procedure from unstable blocks to insufficient and minimum required bolt pattern for stability of blocks are illustrated in Fig. 14. The mechanical properties of bolts and shotcrete, density as well as length of installed bolts around the slope face are key parameters for designing minimum required support pattern for rock slope stability. The significant effect of two latter parameters (bolt density and lengths) on slope stability are considered in this research. In this regard, the total required bolt lengths were estimated for different DFN models. Fig. 14(a) shows the potential instability hazards, and therefore, suggests that some reinforcements are required for the slope. Fig. 14(b) shows insufficient properties of 3 m spacing and 9 m bolt length. However, the slope was stabilized by a 12 m length with 1.5 m spacing pattern of bolts in Fig. 14(c).

Table 4. Properties of bolts and shotcrete.

Bolt		Shotcrete	
Diameter (mm)	40	Density (kg/m^3)	2500
Axial stiffness (GN/m)	3.82	Elastic Modulus (Gpa)	20
Shear stiffness (GN/m)	0.431	Poisson's Ratio	0.15
Ultimate shear capacity (MN)	0.14	Tensile yield strength (Mpa)	450
Ultimate axial capacity (Mpa)	Varies with changes in bolt length	Residual tensile yield strength (Mpa)	450
Active length (m)	0.5	Compressive Yield Strength (Mpa)	20

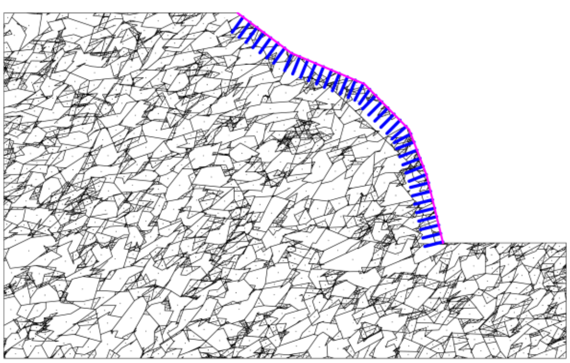
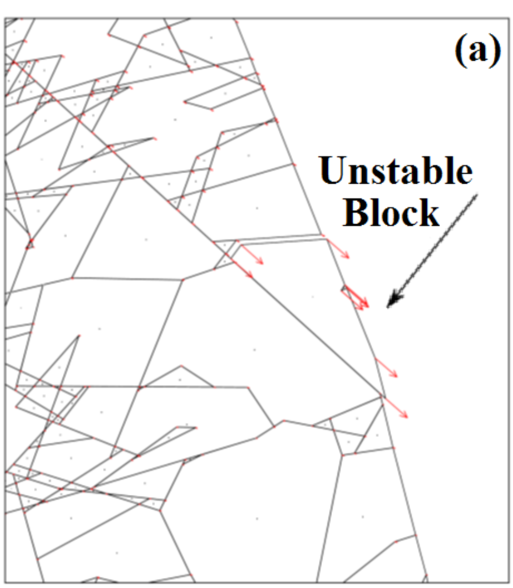
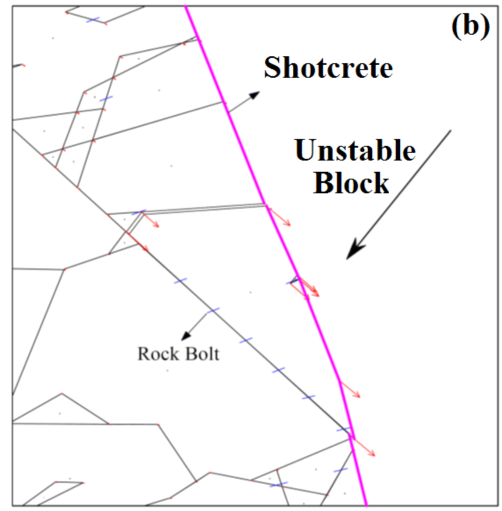


Fig. 13. Position of bolts in slope face.





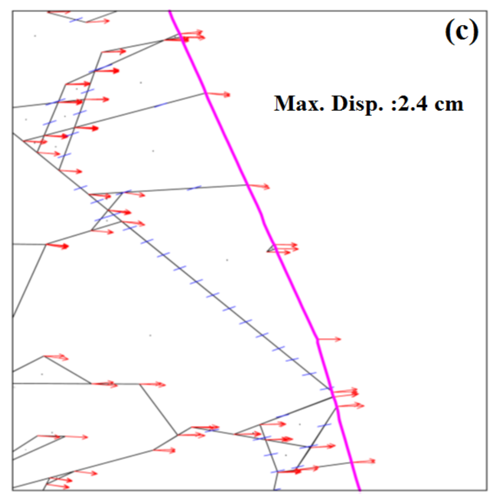
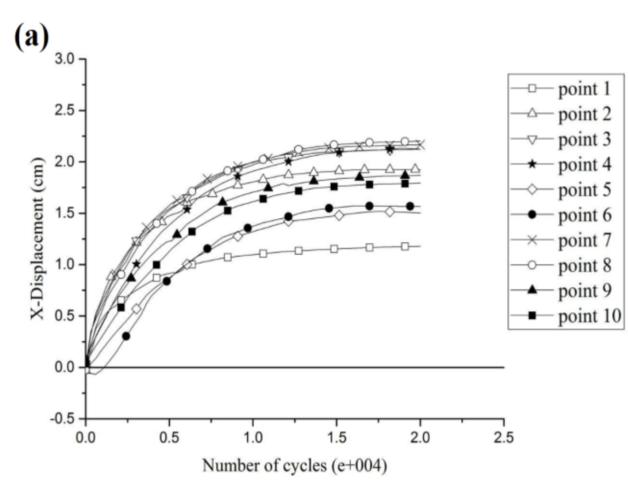


Fig. 14. (a) Unstable block without support, (b) Insufficient blot pattern with bolt length of 9 m and spacing of 3 m and (c) Stable model with bolt pattern of 12 m length and 1.5 m spacing.

Fig. 15 shows X and Y displacements plots versus numerical cycles of the monitored history points indicating that the slope is in equilibrium state after applying the reinforcements. The aforementioned monitoring history points are indicated by numbers in Fig. 6.



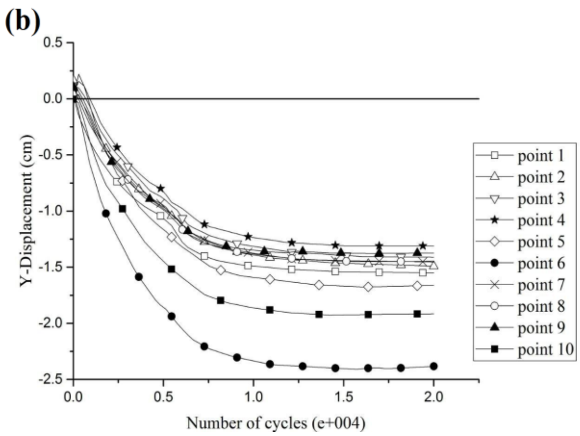
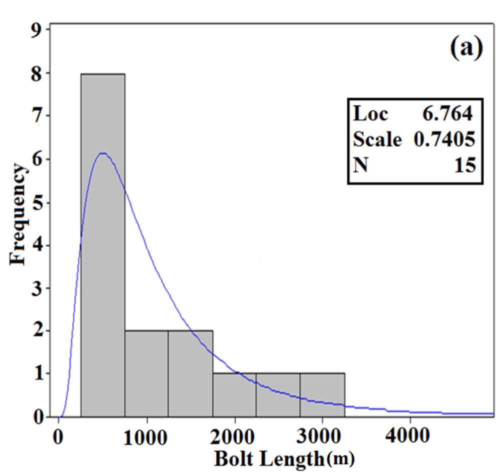


Fig. 15. Displacements along the monitored history points indicating equilibrium conditions. (a) X-displacement; (b) Y-displacement.

Table 5 shows minimum required bolt patterns for 15 DEM models, and Fig. 16 illustrates the frequency distribution (Fig. 16(a)) in addition to empirical cumulative distribution of required bolts lengths (Fig. 16(b)) for all fifteen models. The Mean value of required bolt length is 1125.3 m. As can be seen, the variations of required bolt length follow the lognormal distribution function (Fig. 16(a)). More than 80 percent of DEM models need only 1614.99 m bolt length which means that the bolt pattern with 2m spacing and 12m length is generally the most sufficient bolt pattern for slope stability. However only 5 percent of DEM models require relatively heavy support pattern with 2927.36m which is equivalent to a pattern with 1 m spacing and 12 m length of bolts (Fig. 16(b)).

Table 5. Results of DFN-DEM analysis in dry state.

DFN numbers	Bolt length (m)	Bolt spacing (m)	Number of bolts	Total bolt length (m)
1	9	1.5	177	1593
2	12	3	89	1068
3	12	5	54	648
4	9	1	265	2385
5	12	7	38	456
6	9	5	54	486
7	9	7	38	342
8	12	1	265	3180
9	15	7	38	570
10	12	1.5	177	2124
11	15	7	38	570
12	9	7	38	342
13	12	7	38	456
14	12	3	89	1068
15	9	1.5	177	1593



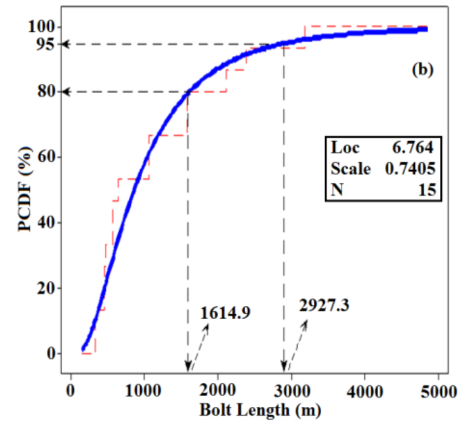
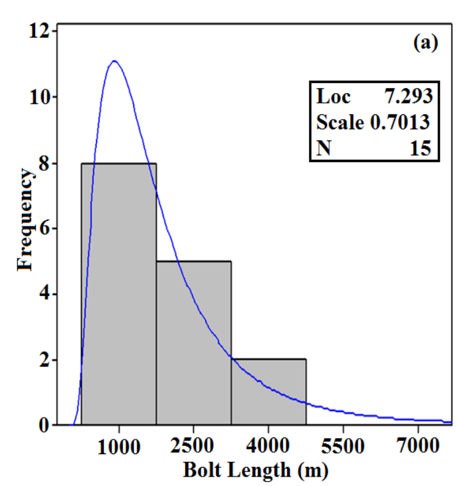


Fig. 16. (a) Distribution of required bolt patterns in DFN-DEM analysis and (b) probabilistic cumulative distribution function of bolt length.

In order to minimize the sliding of blocks on slope face, the applied reinforcement in saturated condition should be slightly stronger than that of the dry condition. Table 6 shows the minimum required bolt patterns of 15 DEM models in saturated condition, and Fig. 17 shows the frequency distribution (Fig. 17(a)) and cumulative distribution of required bolt lengths (Fig. 17(b)) for all of the fifteen models. Mean value of required bolt length is 1821.8 m. The lognormal distribution function is well fitted to the required bolt length for both dry and saturated conditions. Besides, more than 80 percent of DEM models need 2653.49 m bolt which means that the bolt pattern with 1.5 m spacing and 15 m length is the sufficient bolt pattern for the slope stability, whereas only 7 percent of DEM models demand relatively heavy support system which is a pattern with 1m spacing and 15m length of bolts (Fig. 17(b)).

Table 6. Results of DFN-DEM analysis in saturated state.

DFN numbers	Bolt length (m)	Bolt spacing (m)	Number of bolt	Total bolt length (m)
1	15	1.5	177	2655
2	15	7	38	570
3	15	7	38	570
4	15	3	89	1335
5	12	3	89	1068
6	12	1.5	177	2124
7	12	1.5	177	2124
8	15	1	265	3975
9	15	1	265	3975
10	15	7	38	570
11	15	1.5	177	2655
12	12	5	54	648
13	15	1.5	177	2655
14	12	3	89	1068
15	15	3	89	1335



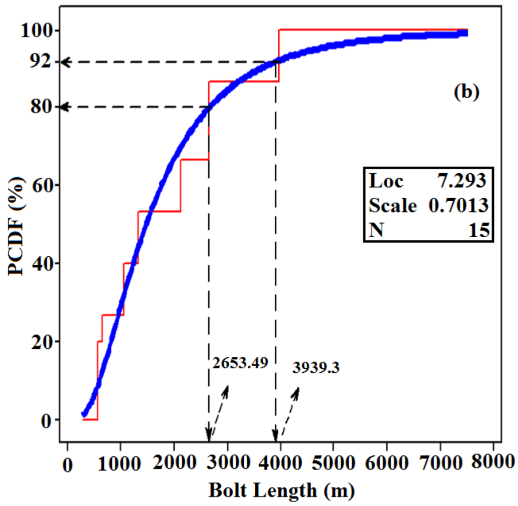


Fig. 17. (a) Distribution of required bolt patterns in DFN-DEM analysis and (b) probabilistic cumulative distribution function of bolt length in saturated state.

4. Discussion

The rock slope stability analysis can be deterministically carried out if the nature and magnitude of the variables included in the analysis are definite. Unfortunately, many of these variables are probabilistic rather than single-valued quantities.

Rock slope stability is highly dependent on discontinuity characteristics, and the random geometrical properties of discontinuities play a critical role in the probabilistic analysis. In comparison to deterministic method, modelling with probabilistic approach is more realistic, since it offers a strong decision making tool in rock slopes reinforcement. The rock slope with widely scattered and variable discontinuity characteristics cannot be properly represented by single value of input parameters. Therefore, it is recommended to apply probabilistic analysis especially in cases where significant variety exists in the rock slope parameters.

In order to validate the numerical probabilistic analysis results, an empirical slope stability analysis is conducted on the right abutment of Karun 4 dam as follows in the next subsection.

4.1. Slope stability analysis using an empirical method

Various classification systems have been developed and reported in the literature to study the rock mass quality. The major application of these systems is developed for tunnels and underground excavations. However, recently, modifications on the common classification systems have provided the possibility of empirically designing the rock slopes. In this regard, Slope Mass Rating (SMR) and Modified Rock Mass Rating (MRMR) systems have been adopted for rock slopes by modifying the RMR classification system. In addition to existing parameters for RMR, SMR consists of some extra coefficients including the spatial position of discontinuities with respect to dip as well as the excavation method. SMR can be evaluated by the following equation (Eq. (1)) [36],

$$SMR = RMR - (F1.F2.F3) + F4 \qquad (1)$$

Where F1 is the indicator of between slope surface strike and the dominant joint set strike. F2 is related to the dominant joint set dip in plane failure mode or the angle of the intersection of two dominant joint sets in wedge failure mode. F3 is the joint dip condition. F4 is associated with the bench surface dip and the bench excavation method.

All of the coefficients can be obtained according to different joint set orientations by Table 7.

Table 7. Quantification of Slope Mass Rating system [36].

ADJUSTING FACTORS FOR JOINTS (F_1, F_2, F_3)	αj = DIP DIRECTION OF JOINT; αs = DIP DIRECTION OF SLOPE; βj = DIP OF JOINT; βs = DIP OF SLOPE				
	VERY FAVOURABLE	FAVOURABLE	FAIR	UNFAVOURABLE	VERY UNFAVOURABLE
PLANE FAILURE $\lvert\alpha j-\alpha s\rvert$ = / TOPPLING $\lvert\alpha j-\alpha s-180°\rvert$ =	> 30°	30° - 20°	20° - 10°	10° - 5°	< 5°
F_1 VALUE	0.15	0.40	0.70	0.85	1.00
RELATIONSHIP	$F_1 = (1-\sin\lvert\alpha_j-\alpha_s\rvert)^2$				
$\lvert\beta j\rvert$ =	< 20°	20°-30°	30°-35°	35°-45°	>45°
F_2 VALUE PLANE FAILURE	0.15	0.40	0.70	0.85	1.00
F_2 VALUE TOPPLING	1.00				
RELATIONSHIP	$F_2 = tg^2\beta j$				
PLANE FAILURE $\beta j-\beta s$ =	>10°	10°-0°	0°	0°-(-10°)	<(-10°)
TOPPLING $\beta j+\beta s$ =	< 110°	110°-120°	>120°	-	-
F_3 VALUE	0	-6	-25	-50	-60
RELATIONSHIP	F_3 (BIENIAWSKI ADJUSTMENT RATINGS FOR JOINTS ORIENTATION, 1976)				
F_4 ADJUSTING FACTOR FOR EXCAVATION METHOD	F_4 = EMPIRICAL VALUES FOR METHOD OF EXCAVATION				
	NATURAL SLOPE	PRESPLITTING	SMOOTH BLASTING	BLASTING or MECHANICAL	DEFICIENT BLASTING
F_4 VALUE	+15	+10	+8	0	-8

In this analysis, a set of data containing GSI values of the right abutment of Karun 4 dam were used for estimating the RMR by Eq. (2) [37] and then SMR was calculated accordingly. Fig. 17 shows GSI values measured from the rock slope media. The average value of GSI is 52 and RMR_{89} is correspondingly estimated by the following Eq. (2),

$$RMR_{89} = GSI - 5 = 47 \qquad (2)$$

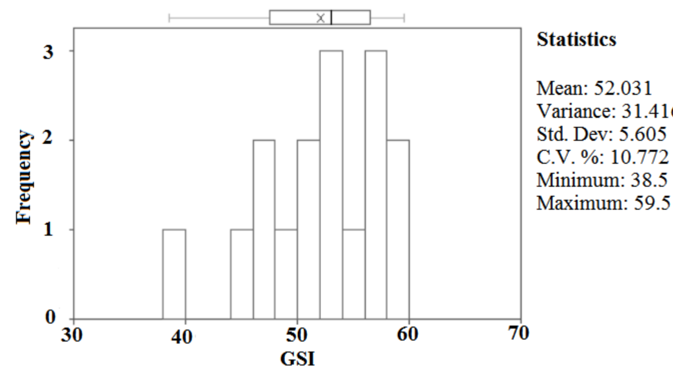


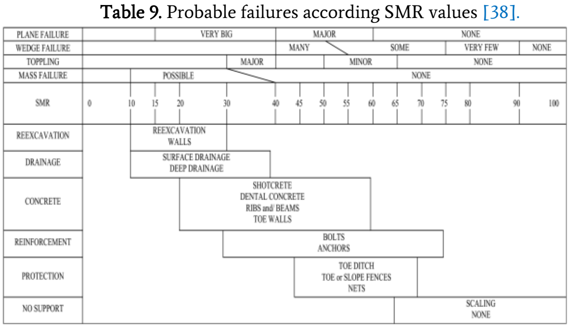
Fig. 17. Distribution of GSI values in the right abutment of Karun 4 dam [28].

There is a 10° difference between the slope surface strike and joint set strike. Thus, the equivalent F_1 is 0.7. By considering the main joint set with 44° dip angle, F_2 is determined as -25 and F_4 is 15 because of the natural and intact surface of the slope. According to the values of these coefficients and the obtained RMR, the SMR was 47. Considering different categories of stability for SMR system in Table 8, the overall condition of rock slope was evaluated as unstable [38]. This instability was the combination of wedge and plane failure in the direction of some joint sets. As indicated in Table 9, the systematic bolting together with the shotcrete layers were suggested as the support system [38].

Table 8. Tentative description of SMR classification [38].

Class no.	V	IV	III	II	I
SMR	0-20	21-40	41-60	61-80	81-100
Description	Very bad	Bad	Normal	Good	Very good
Stability	Completely unstable	Unstable	Partially stable	Stable	Completely stable
Failures	Large planner or soil-like	Planner or big wedges	Some joints or many wedges	Some blocks	None
Support	Re-excavation	Important/corrective	Systematic	Occasional	None

Table 9. Probable failures according SMR values [38].



Regarding the obtained results from numerical and empirical methods in slope stability analysis, slope failure is possible in accordance with both methods. In the empirical method, wedge and blocky failure in the direction of dominant joint sets is predicted. The same blocky failures in the direction of the main joint set are anticipated in the numerical models with different DFNs as well. The suggested support pattern for stabilizing the slope in the empirical method is a systematic rock bolting or shotcrete system, while according to the results obtained from the numerical approach, either of these support systems do not suffice. By employing the probabilistic method a combination of shotcrete and bolt pattern is considered as the best support system in both dry and saturated conditions.

5. Conclusions

In this study a probabilistic analysis procedure together with related algorithms using Discrete Fracture Network-Distinct Element Method (DFN-DEM) approach and Monte Carlo simulation technique was utilized.

This probabilistic analysis was applied to right abutment of Karun 4 hydro power plant. Different combinations of geometrical parameters for fracture sets were selected. For generated and validated DFN models, a series of numerical DEM modelling were conducted to measure the minimum required support pattern in dry and saturated slope conditions.

Besides, as an empirical approach, the numerical probabilistic results were compared with that of empirical SMR classification system. Although both methods verify the necessity of stabilizing the rock slope with wedge and blocky failure, the numerical probabilistic method was capable to model different types of fracture networks. In other words, investigation of a slope stability with different DFN realizations which conducted in different block patterns is more efficient than the empirical SMR method.

The following results are deduced from the numerical models:

- Realistic modelling of structural features properties that control the stability of the rock mass revealed that the DFN-DEM analysis has the potential to produce safer rock slope designs and optimized support requirements for rock slope stability.
- In all fifteen models for dry conditions, blocks on the major joint have the maximum displacements. As for the saturated conditions, displacements for blocks on the major joint and under regions of slope face are remarkably larger than displacements in other regions and clearly larger than that of dry condition. On the other hand, their displacement direction is usually inclined upward. There is also some local sliding in the mentioned regions.
- In dry conditions, more than 80 percent of DEM models need only 1614.99m bolts which means that the bolt pattern with 2m spacing and 12m length is the most sufficient bolt pattern for rock slope stability at this confidence level.
- In saturated conditions, more than 80 percent of DEM models need only 2653.49m bolts which means that the bolt pattern with 1.5m spacing and 15m length is considered as the most sufficient bolt pattern for rock slope stability at this confidence level which is heavier than the support pattern for dry slope condition.
- Probabilistic approach provides a flexible tool for engineers to design different support patterns based on the significance of a project and the required confidence level. The results acquired from the case study have demonstrated the applicability of the presented approach.

Further research on conducting DFN-DEM analyses for reducing the uncertainties of mechanical properties (in case of existing mechanical properties distribution functions), employing other discontinuity failure criteria, dynamic analysis in dry and saturated conditions and 3D static and dynamic analysis would be necessary for this site.

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