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Rock Slope Stability Analysis in the Left Abutment of Bakhtiary Dam, Iran

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ABSTRACT

In this research, directions of in-situ stresses in the rock slope in the left abutment of Bakhtiary dam (Center of Iran) are defined taking advantage of geological history, tectonic evolution of the area, and in-situ tests. To that end, the study draws on the kinematic analysis, limit equilibrium and numerical methods. It is of note that there is no possibility for toppling failure if kinematic analysis is used to study the stability in left abutment of Bakhtiary dam. The plane failure analysis indicated that there is a possibility of failure in the middle and upper walls based on joint set J1. Also, from geological perspective, wedge failure in the middle and upper walls is possible due to the intersection of bedding planes and Joint set J1. In the analysis of the slope stability using limit equilibrium, the least value of the safety factor obtained for plane failure belongs to joint set J1 in the upper wall, indicating that the left abutment is stable. Numerical analysis indicated that this slope needs support requirements.

1. Introduction

There are different methods, with their own advantages and disadvantages, to assess slope stability. The selection of analytical methods depends on the local conditions and the type of slide. Generally, the initial issues in the slope stability analysis are summarized as follows [1]:

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● Specifying the conditions of slope stability
● Assessing the type of slide potential
● Determining the reinforcement requirements
● Deciding on the optimum design for excavation of the slope

The ground slide can be characterized as falling, toppling, flowing behavior, or a combination of them [2]. Gurocok et al investigated the slope stability of left and right abutments of Kapicaya dam in eastern Turkey using kinematic and numerical methods [3]. Romana introduced rock mass classification to assess rock slope stability [4]. Wei compared LEM with SRM in two and three dimensional methods [5]. Alexanwach et al compared different conventional methods for slope stability analysis and concluded that the numerical methods combined with Monte Carlo are very effective [6]. Bhasin and Keynia demonstrated the application of DEM analysis to estimate the rock volume in a large failure with 700 m height of rock slope in Norway [7]. Steed and Abrhart investigated the effect of topography, lithology, water conditions, block shapes and modelling of rock slopes in the continuous and non-continuous mediums [8].

2. Bakhtiary dam location

The Bakhtiary dam is located in a strait through which Bakhtiary river passes nearly vertical to Siahkok syncline [9]. The strait has 150 m length and 25 to 35 m width. Bakhtiary dam will be made over Bakhtiary river with the height of 315 m, which will be the highest concrete dam in world. The upstream of dam axis consists of Siahkoh syncline which has low dip at lower levels, and the dip increases at higher levels of slope; sometimes the dip is vertical or even negative in some parts. Fig 1. Shows the valley in the dam location [10].

Fig. 1. Bakhtiary dam location [10].

The bed rock of Bakhtiary dam consists of limestone and marly limestone with silica nodules. The limestone may contain some percentage of dolomite. These sediments are have been introduced as Bangestan group in geological maps provided by National Iranian Oil Company. The limestones in the strait of dam are divided into seven groups from SV_1 to SV_7. Two asymmetrical synclines specified in the geological report are as follows:

1) Siahkoh syncline which is smaller, and located in the dam site, 2) Grayweh syncline which is bigger. Both of the synclines are bounded by faults. Fig 2. Shows the structure arising from Grayweh syncline over Siahkoh syncline, synclines axis, and F_1-F_3 fault system.

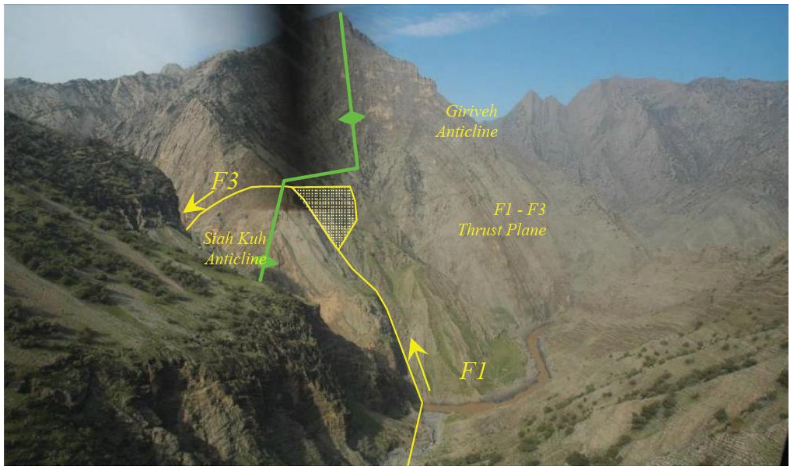


Fig. 2. The location of Grayweh, Siahkoh synclines, and F_1-F_3 fault systems.

3. The laboratory test results

A large number of core samples from Bakhtiary dam (SV_1 to SV_7 units) have been used to test intact rocks and rock discontinuities according to ISRM. The statistical analyses demonstrated that the values related to SV_2, SV_3, SV_4 units are close and sometimes identical. As these units are located in the dam site, they have been introduced as Group A and thus analyzed together. Group B and SV_7 unit have been assessed separately and presented in Table 1 [11].

Table 1
The laboratory results of samples in A, B groups and SV_7 unit [11].

Type of group and rock units			Group A (Sv2,Sv3,Sv4)		Group B (Sv5,Sv6)		Sv7 Unit	
Type of Laboratory test	Description	Unit	Dry	Saturated	Dry	Saturated	Dry	Saturated
Index Test	Weight unit, γ	kg/m^3	2640	2650	2650	2660	2640	2650
	Porosity, n	%	1		1		1	
	Water content, W	%	0.30		0.30		0.40	
	ν	-	0.30	0.30	0.30	0.30	0.32	0.33
Unconfined compressive strength test	UCS	MPa	120	105	125	110	77	75
	E	GPa	65	60	65	60	63	58
Triaxial compressive test	σ_{ci}	MPa	125		120		93	
	m_i	-	14		9		11	
	C	MPa	19.6		32.5		30	
	Ø	°	31		36		41	

Several tests were done to define geomechanical parameters of rock mass in dam site, which is presented in Table 2 [9]. Fig 3. shows the location of galleries and in-situ tests [12].

Table 2
Geomechanical parameters of rock mass [9].

Type of tests	Name of the tests	Number of the tests
Laboratory Tests	Uniaxial test	162
	Triaxial test	125
	Direct shear test	106
	Indirect tensile test (Brazilian)	47
In-situ tests	Plate bearing test	47
	Dilometer test	86
	Flat Jack test	9
	In-situ shear test	3

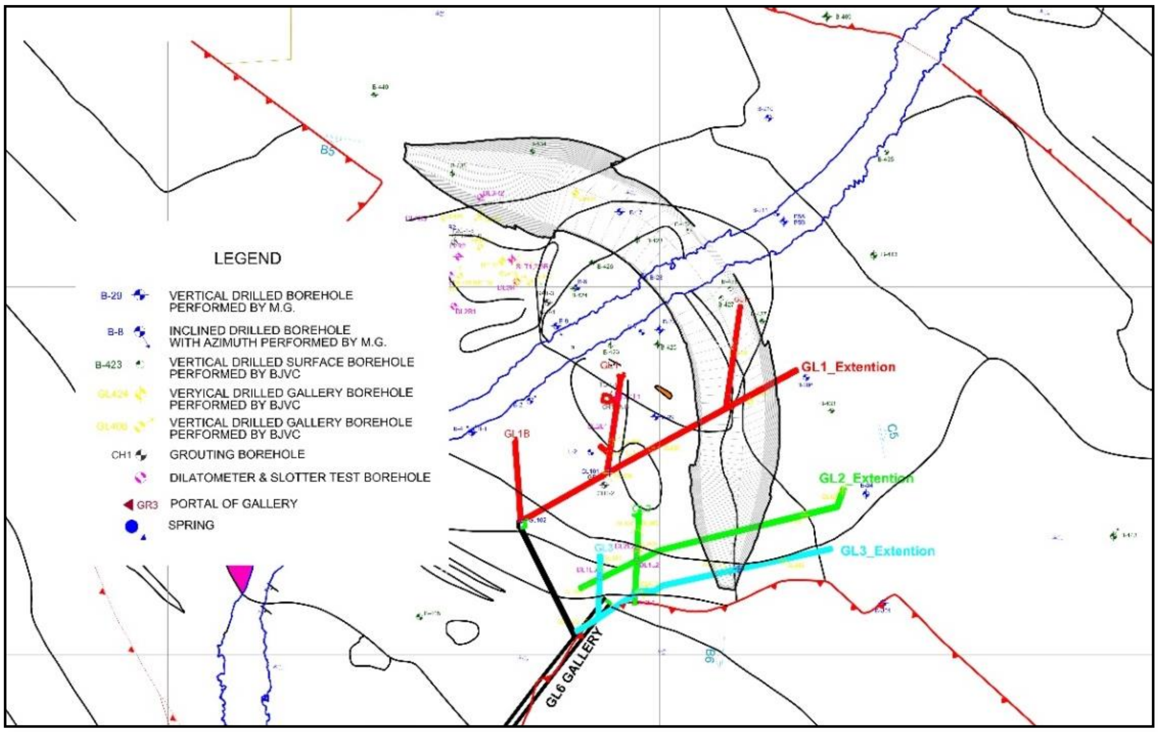


Fig. 3. The locations of galleries and in-situ tests [12].

4. System of discontinuities

Generally, the rock mass of Bakhtiary dam consists of four discontinuity systems which affect stability and bearing capacity. Due to thin to thick bedding, this discontinuity is the most abundant in Bakhtiary dam. Joint set J_1, is the major factor to create valley. This joint extends from several meters to 10 meters. Joint set J_2 is less abundant compared to joint set J_1. Joint set J_3 is the least abundant in the dam site. Using the data obtained from discontinuities and the stereographical map, there can be observed 4 discontinuity sets in the left abutment of Bakhtiary dam, consisting of two major and one minor joint set with bedding plane in the upstream and downstream of Siahkoh syncline. The results of joint system analysis using DIPS software are presented in Table 3 and Fig 4.

Table 3
Dip and strike of available discontinuities in left abutment of Bakhtiary dam.

Discontinuity system	Dip (◦)		Dip direction (◦)	
	Average	Standard deviation	Average	Standard deviation
Bedding planes above the anticline axis of Siahkoh	51	10.2	030	8.0
Bedding planes below the anticline axis of Siahkoh	69	8.6	216	6.1
J_1	66	13.5	315	10.9
J_2	59	15.0	123	11.4
J_3	15	4.64	031	6.6

5. Left abutment of Bakhtiary dam

In Bakhtiary dam, 106 direct shear tests and 3 in situ tests were done on discontinuities. The parameters were defined on blocks of 10 × 10 cm, and 15 × 15 cm in the laboratory and blocks of 70 × 70 cm in situ. By integrating the laboratory and in-situ results, the geomechanical parameters of discontinuities at dam site are presented in Fig 5 andnd Table 4 [13].

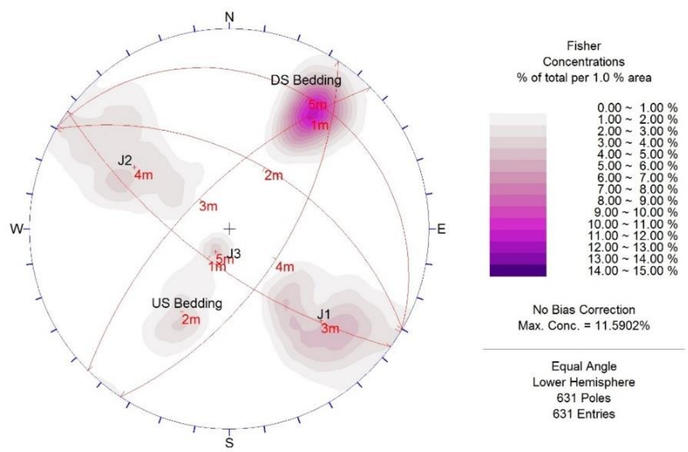
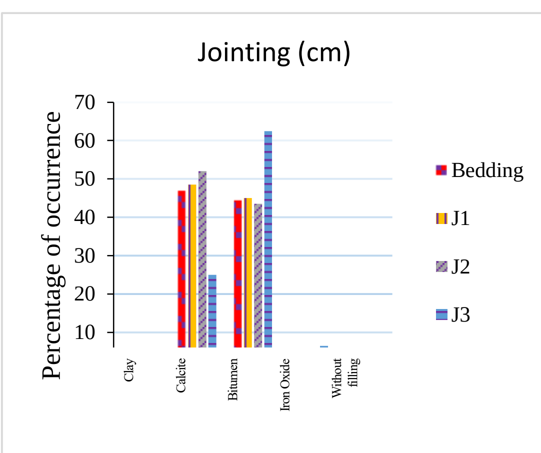
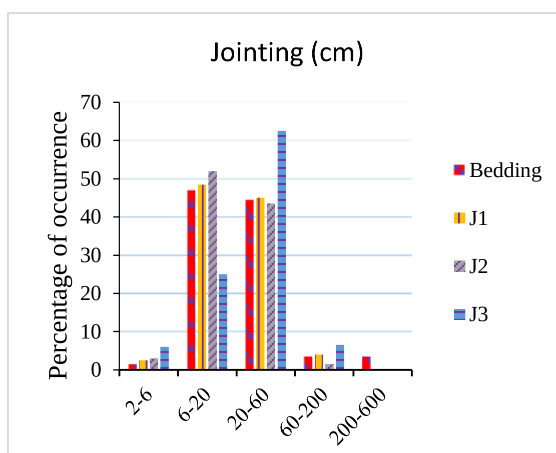


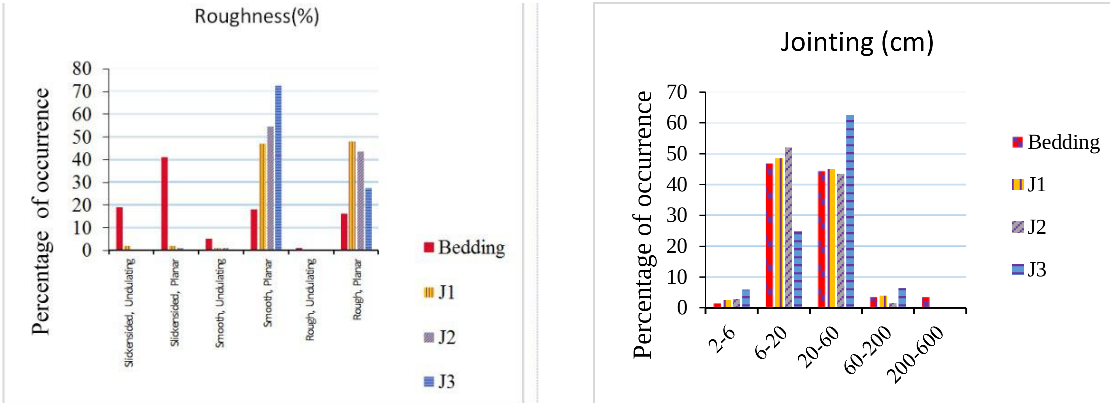
Fig. 4. Available discontinuities of rock mass in Bakhtiary dam.



A)



B)



C) D)

Fig. 5. Characteristics: 1) filling 2) aperture 3) surface roughness 4) jointing of available discontinuities surfaces in Bakhtiary dam [13].

According to Fig 5, it is clear that most discontinuities display and aperture of 0.1 to 1 mm, planar and smooth, planar and rough, and jointing of 6 to 60 cm. Also, infilling materials of the bedding planes are often clay, and calcite, and as for the joints the materials are calcite.

Table 4
Geomechanical parameters of discontinuities in Bakhtiary dam [13].

Type of Discontinuity	C (kPa)	Ø (∘)
Bedding planes	280	30
Joints	350	29

To measure shear strength of bedding planes, the in situ shear test was performed in stable normal loading conditions at GR2 gallery. To that end, the blocks with dimensions of 70 × 70 × 35 cm were selected according to ISRM. After preparation, the in situ direct shear tests were done on the three blocks (ST1R2, ST2R2, ST3R2). In each stage, the samples were subjected to shear, and the shear load, and natural load values together with shear, and normal displacements were measured [14]. The shear and normal stiffness values for the in situ shear test are presented in Table 5.

Table 5
Shear and normal stiffness for the in situ tests [14].

Block	Normal stress (MPa)	Normal stiffness (MPa/mm)	Shear stiffness (MPa/mm)
ST1R2	8	7.669	1.759
ST2R2	8	7.732	1.903
ST3R2	8	7.254	1.698

It is necessary to indicate that the results of joint set J_3 have been left out because of their low abundance.

To define in situ stresses in Bakhtiary dam, 17 hydraulic fracture tests were done in the dam axis and underground powerhouse. The location of boreholes for hydraulic fracture tests and the direction of in situ stress at HF1L1 borehole in Bakhtiary dam are shown in Fig 6. The in situ stress conditions around Bakhtiary dam were analysed with hydraulic fracture tests at HF1L1 borehole, the results of which are displayed in Table 6:

Table 6
The measured in situ stresses by hydraulic fracture at HF1L1 borehole.

Location	Vertical stress (MPa)	Major Horizontal Stress (MPa)	Minor Horizontal Stress (MPa)	Azimuth of the Major Horizontal Stress
Borhole HF1L1	5.35	2.46 ± 1.3	1.32 ± 1.25	058

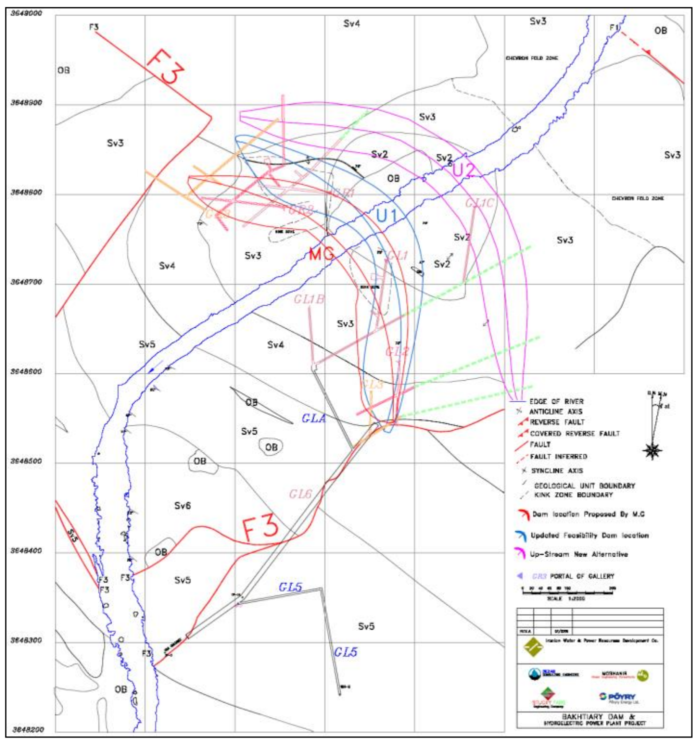


Fig. 6. Borehole location for hydraulic fracture tests.

6. Rock slope stability analysis using analytical method

Bakhtiary dam is located in a strait nearly narrow with high and vertical walls composed of limestone layers. Generally, the right abutment is formed with high walls and is nearly uniform. The dip of right wall is 70 º degrees and its height is 490 m around the dam axis, which increases to 620m in downstream of Siahkoh syncline axis. The left abutment is formed with separate walls. The first wall (lower wall) has a height of 110-120 m with 40 º to 45 º degree dip. The second wall (middle wall) has the height of 180-200 m with 65 º-76 º degree dip. It is necessary to indicate that middle wall is the major wall in the left abutment, where the dam crown is located. Upper wall is placed in the upstream of middle wall. The upper wall has the height of

100 m with 80 º degree dip. According to the results of discontinuity system analysis at the left abutment, (Figs 4, 5), four major discontinuity systems were specified. The dominant joints were defined as instability factor following the analysis of the results using DIPS software. Figs 7 to 9 show the results obtained from stereographical analysis in lower, middle and upper walls. According to these results, there is no possibility of failure in plane, wedge and toppling conditions in the lower wall. As Figs 8, 9 display, the joint set J_1 is placed inside the plane failure area in the middle and upper walls, but because of high cohesion and friction angle of this joint set, there is no plane failure. According to the same Figs, there is a potential for wedges to create with intersection of joint set J_1 and bedding plane in middle and upper walls. Considering Hoeking method, if there is a possibility of slide, wedge failure happens in the line intersection between joint set J_1 and bedding plane in the upstream. In the downstream, the slide is created on joint set J_1 . Also, according to the results obtained from stereographical analysis, there is no possible toppling failure in the middle and upper walls. Using stereographical graphs, it can be concluded that in addition to weak planes conditions and internal friction angle, the dip slope value is effective in rock slope stability.

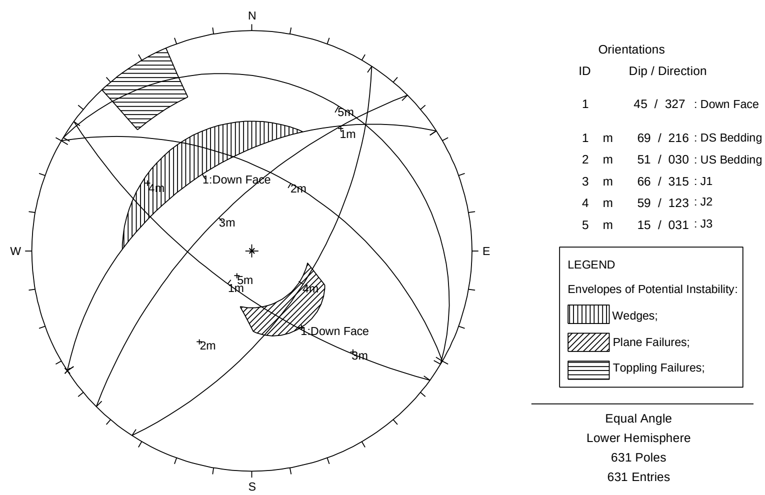


Fig 7. Kinematic stability analysis in lower wall.

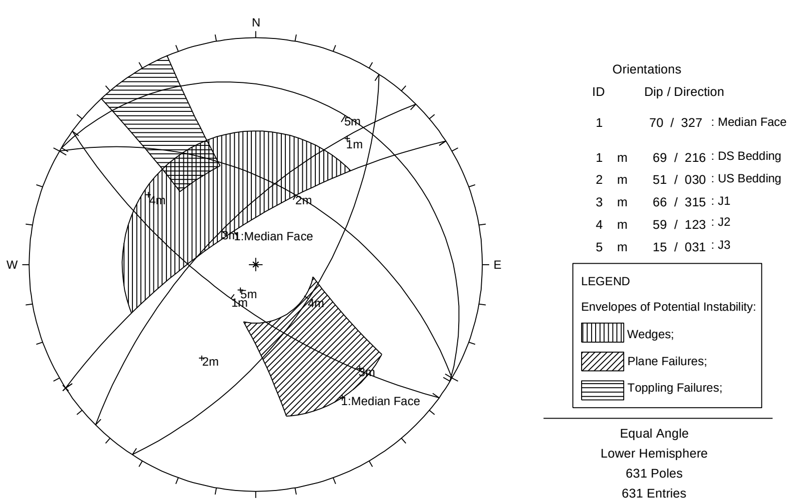


Fig. 8. Kinematic stability analysis in middle wall.

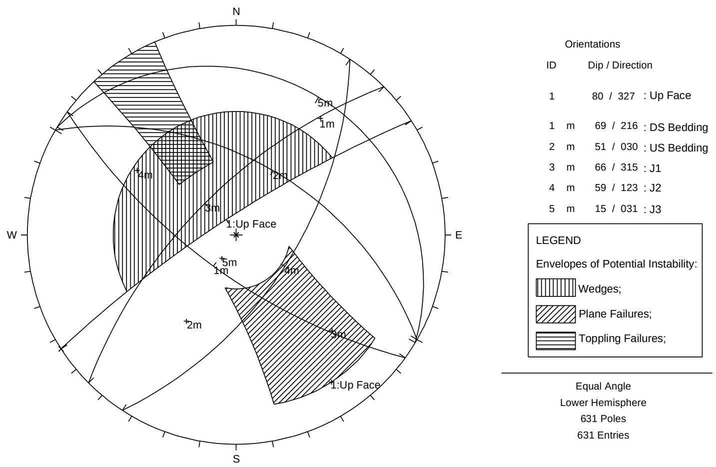
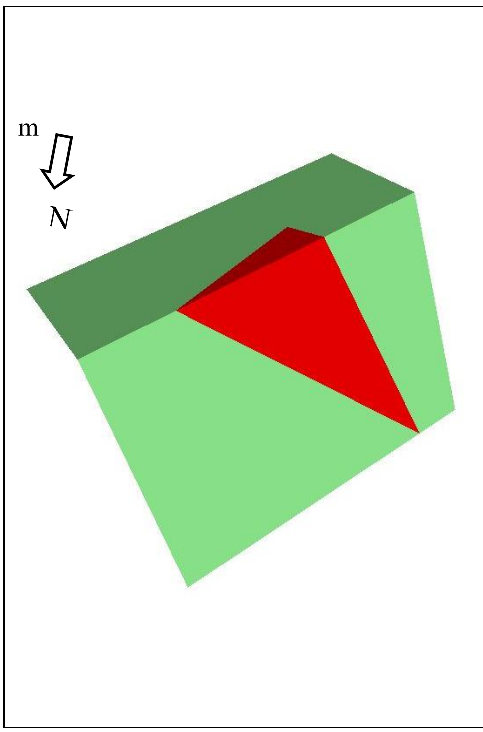


Fig. 9. Kinematic stability analysis in upper wall.

Stability analysis using Limit Equilibrium method

Swedge software was used to assess wedge failure potential in the left abutment. This software is based on Limit Equilibrium method which can assess the wedge failure potential and calculate the safety factor according to joints' strength parameters and geometry of wedge. The Rocplane software was used for the Limit equilibrium analysis of plane failure. Figs 9, 10 show the geometry of wedge and plane failure, respectively.



Wedge Information

Wedge Data

Volume: 19434.802 m3

Weight: 51502.23 tonnes

Line of Intersection

Plunge: 57.40 deg

Trend: 269.12 deg

Length: 119.04 m

Persistence

Joint1(Bedding): 119.04 m

Joint2(J1): 141.46 m

Fig. 10. Wedge failure in upper wall at left abutment using Swedge software.

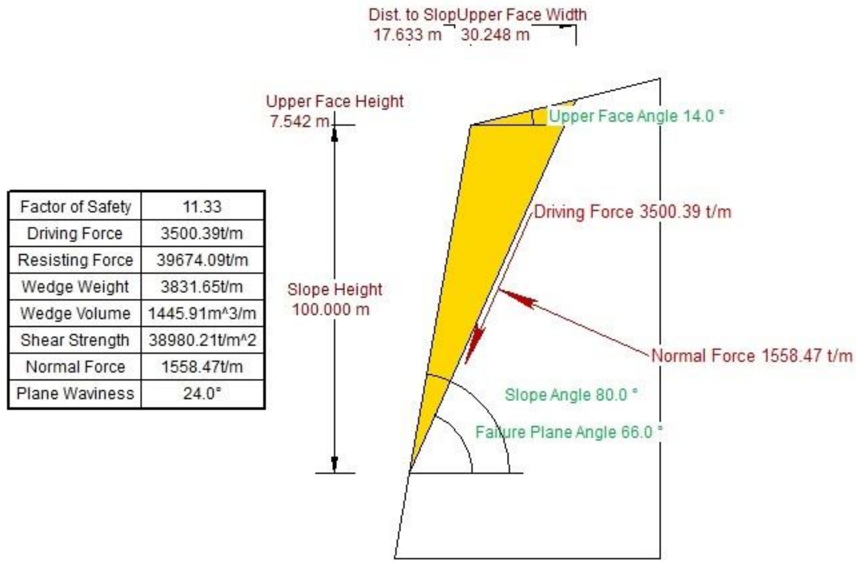


Fig. 11. Plane failure in intersection between joint set J_1 and upper wall at left abutment (RocPlane software analysis).

Stability analysis of left abutment at Bakhtiary dam was done using Limit Equilibrium software. The results of the analysis are presented in Table 7. After installation of support system, the safety factors for plane and wedge failure were assumed 1.5 for static conditions and 1.2 for semi-static ones [15]. The shear discontinuities parameters are shown in Table 4. Based on the earthquake risk of Bakhtiary dam, the MCE value (Maximum Credible Value) is considered as equal to DBE=0.28 (Design Basis Earthquake). Therefore, earthquake factor was chosen 0.14 g for semi-static rock slope stability analysis [16]. As can be seen in Table 6, the safety factor was highest for all the static conditions. In all failures, the earthquake conditions become stable after implementation of support system. Also, it is shown that the least safety factor for plane failure is related to joint set J_1 in the upper wall. On the whole, it is recommended that 5 cm shotcrete be performed for reinforcement of slope in order to prevent erosion, weathering, and water washing in the long run.

Table 7
Stability analysis of left abutment at Bakhtiary dam.

Location of the slope	Type of instability	Possible failure	Type of discontinuities	Safety factor without support		Wedge weight
				Static loading	Semi-seismic loading	
Upper wall	Planar	Stable	-	-	-	-
	Wedge	Stable	-	-	-	-
	Toppling	Stable	-	-	-	-
Middle wall	Planar	Stable	J_1	61.32	57.60	4405 t/m
	Wedge	Stable	Bedding & J_1	100	39.34	9401247 t
	Toppling	Stable	-	-	-	-
Lower wall	Planar	Stable	J_1	11.33	10.54	15326 t
	Wedge	Stable	Bedding & J_1	46.72	40.88	51502 t/m
	Toppling	Stable	-	-	-	-

7. The slope geometry of left abutment

A comprehensive model of Bakhtiary dam abutments were developed using 3DEC software (3 Dimensional Distinct Element Code) [17]. An attempt was made to develop the models that are close to the reality considering the variations of topography. The boundaries of the modeling were chosen as displayed in Fig 12. The model geometry and the dimensions for static analyses are shown in Fig 13. Also, Fig 14 depicts 3 dimensional model and geological units at Bakhtiary dam.

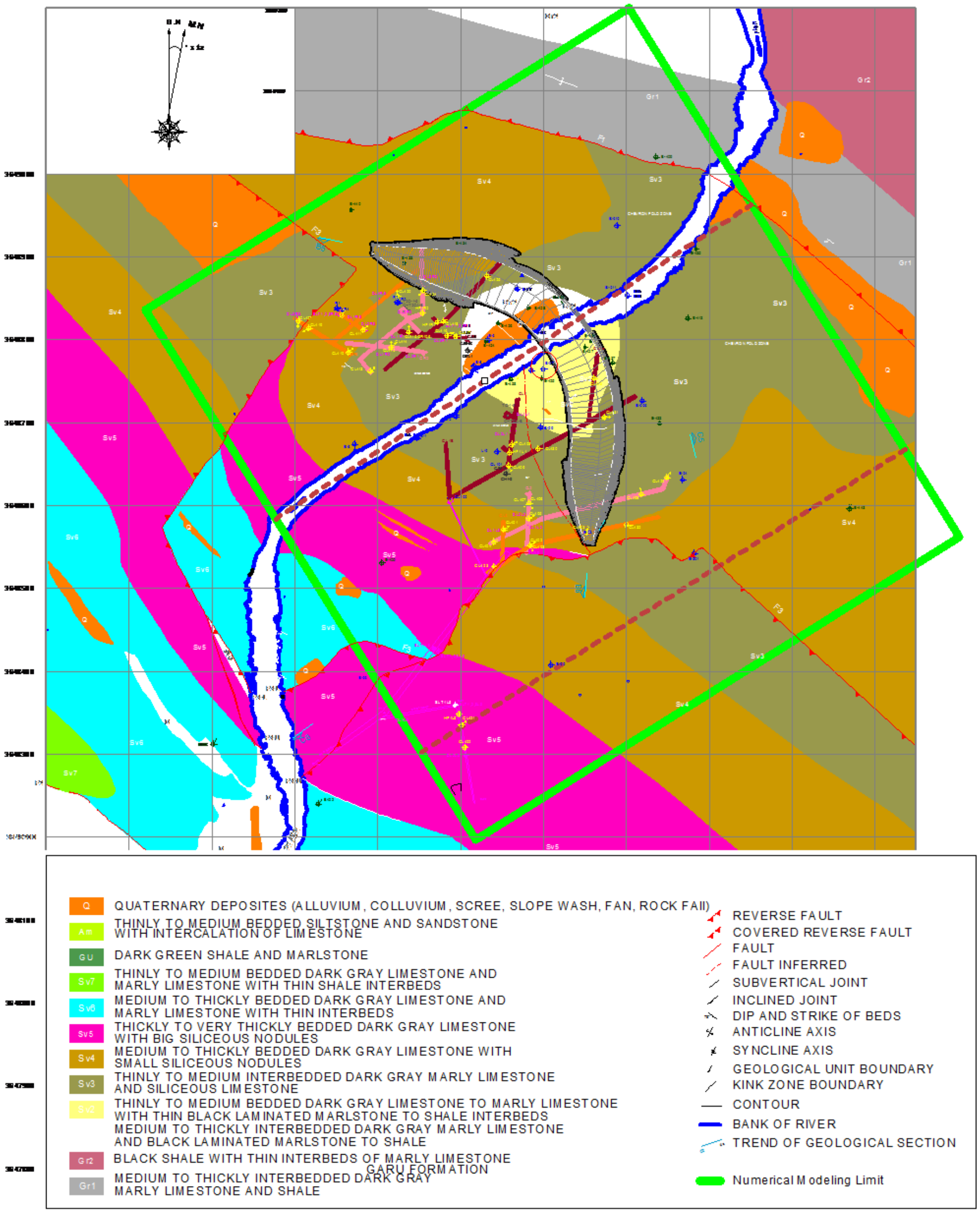


Fig. 12. The boundaries of the studied area.

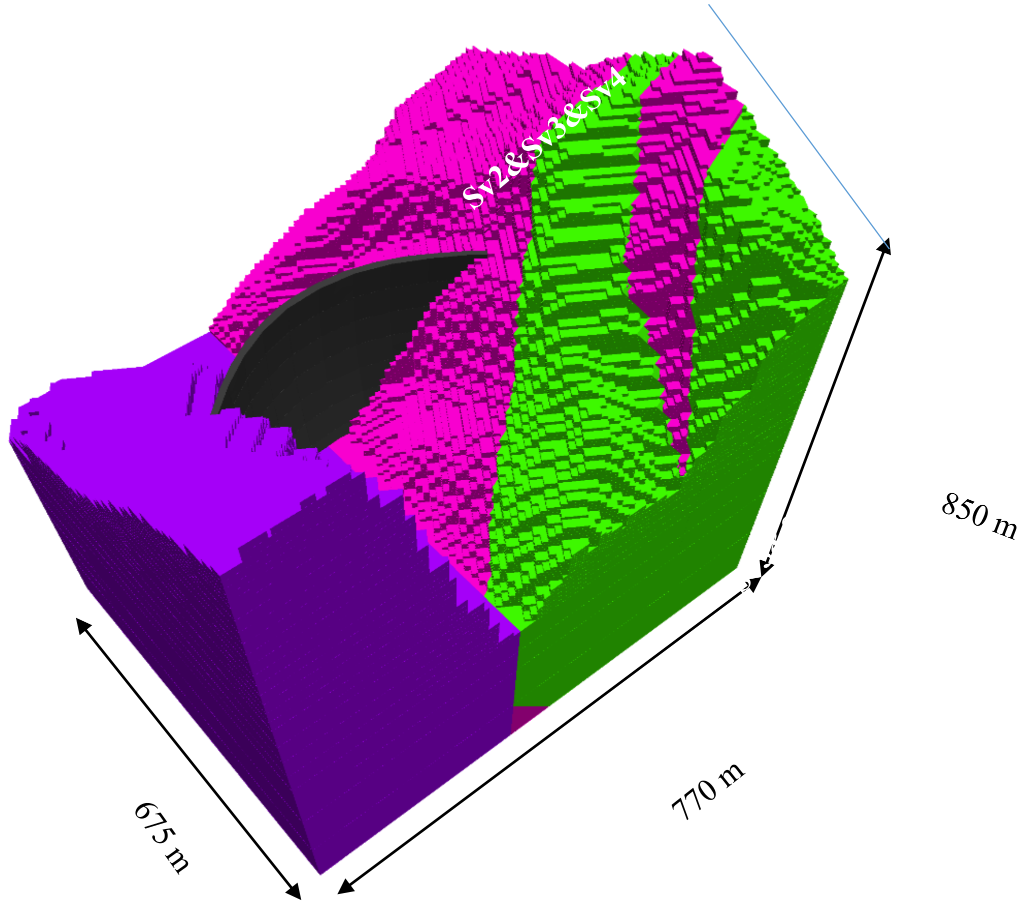


Fig. 13. The model geometry with geological units.

7.1. Geometry of joints

In this research, considering the large volume of the model, and high density of the discontinuities, the smaller area was chosen for the slope analysis of left abutment. The boundaries in discontinuous medium was selected based on the slope area with the most displacement variations.

7.2. The dam structure

To develop the dam structure model, at first the digital map was created, and the model was divided into different levels of height. In fact, these levels need to be appropriate for the selected levels for the abutments. The meshed model of the dam structure is shown in Fig 14.

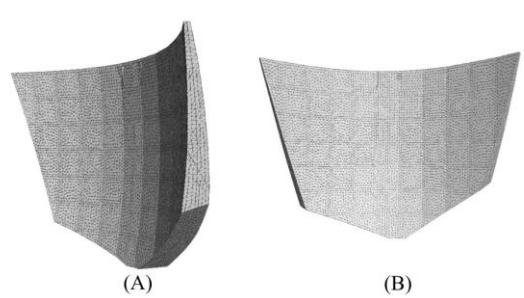


Fig. 14. The dam structure model in 3DEC software; a) view from left abutment, b) view from downstream.

7.3. Behavioral model selection

In selection of behavioral model, rock type and stress conditions are very important and play vital roles. In this research, for static analysis, the rock blocks were considered as deformable materials, and coulomb model was chosen for the analysis of the blocks. The geomechanical parameters of conventional concrete applied for the construction of the dam are shown in Table 8.

Table 8
Geomechanical parameters of the concrete for construction of the dam.

Modulus of deformation (GPa)	Poisson' ratio	Unit weight (kg/m^3)
23.6	0.2	2700

8. The boundary conditions and initial stresses

In the studied model, the stresses were based on overburden materials and the overall stresses of the area were calculated and implemented according to Table 6. Fig 15 shows the direction of initial stresses considering the topographical variations. The total displacements on 5 vertical cross-sections and perpendicular to river axis were recorded according to Fig. 16, using the analyses on model of Fig. 13. The difference between the values in each section was considered 50 meter. Fig 17. shows the location of different points in the slope for recording the displacement history in the middle vertical section (section C).

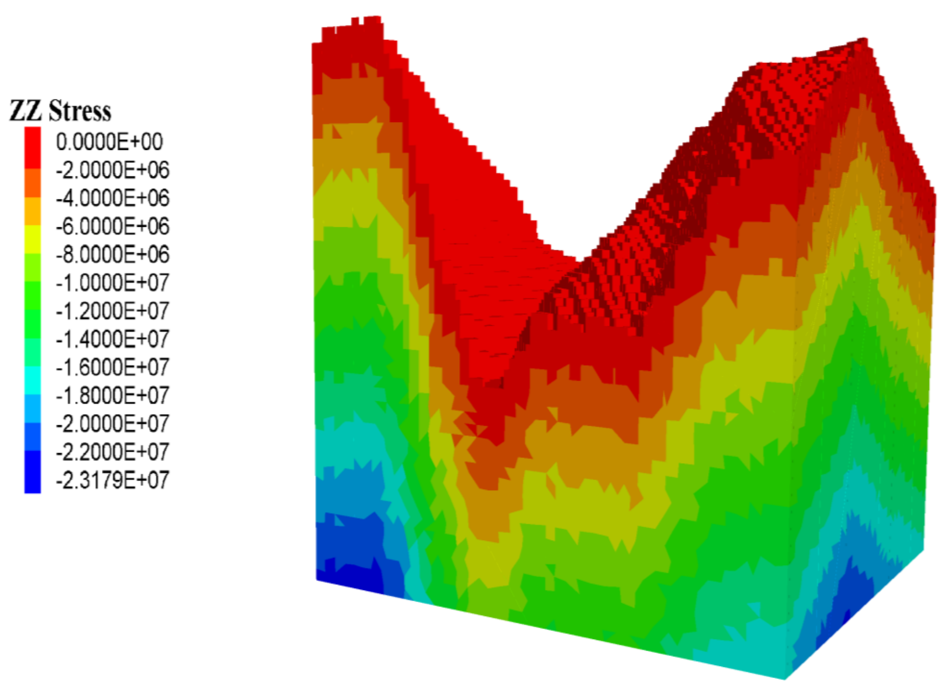


Fig. 15. Vertical stresses.

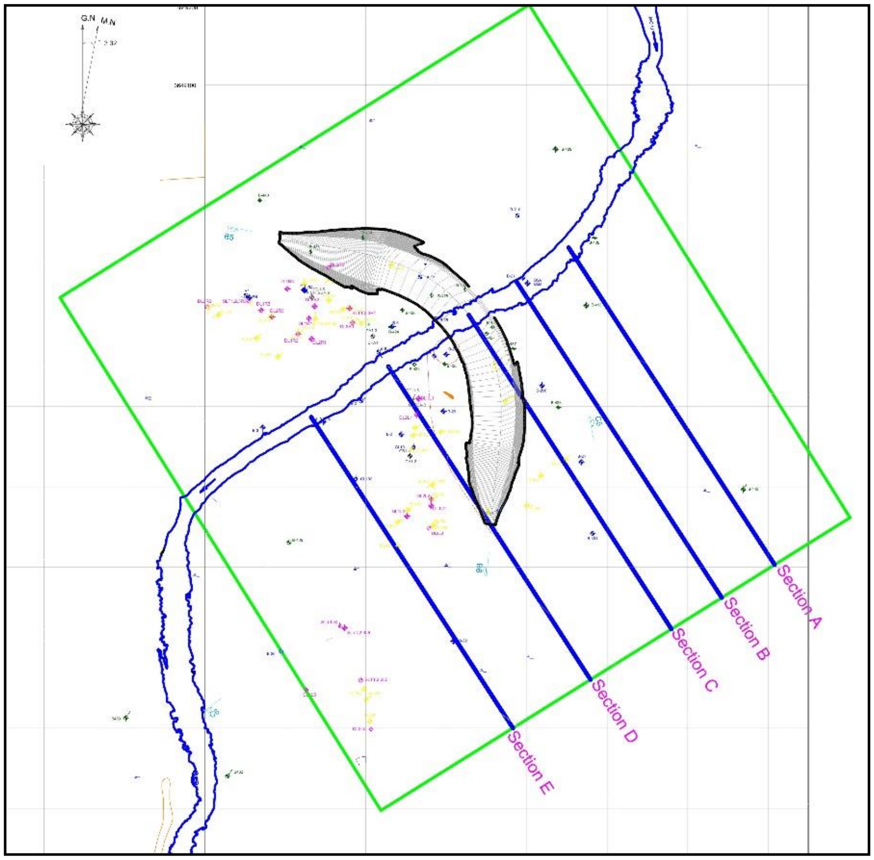


Fig. 16. The location of sections on slope for recording displacement history.

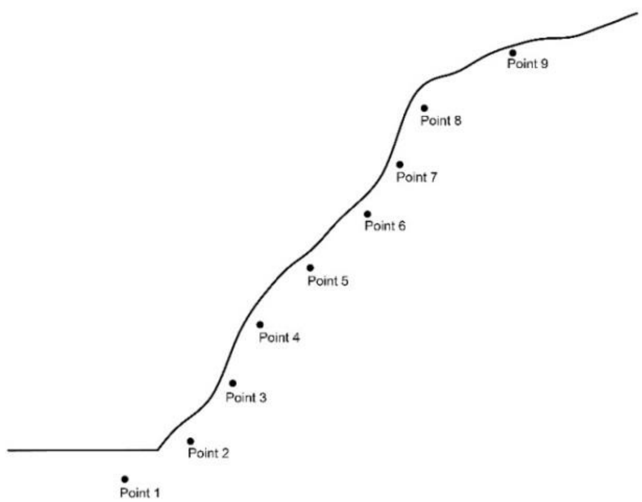


Fig. 17. The location of different points in slope in the middle section (section C).

Three models were developed to assess the behavior of discontinuities and their effects on stability. The first two models considered strength characteristics of rock mass as equivalent to continuum medium, without applying the discontinuities to the model, according to range of GSI values in each geological structures. In the third model, the rock slope stability was assessed taking into account all the discontinuities consisting of joints sets J_1, J_2 and the upper and lower surfaces of bedding plane in Siahkoh anticline. Table 9 shows GSI values for each geological structure.

Table 9
GSI values for each geological structure in the studied area.

Type of group and rock units	Group A (Sv2,Sv3,Sv4)	Group B (Sv5,Sv6)	Sv7 units
GSI	45-65	55-65	45-55

The highest values of displacements in equivalent continuum medium considering the range of each sections of Fig 16 are presented in Figs 18, 19. According to Figs 18, 19, most displacements of rock slope in equi-continuum medium are related to the lower part of dam axis and lower wall. After construction of the dam structure, as shown in Figs 18, 19, most displacements are related to the middle section at river bed and abutments in the levels lower than 650 m. Figs 18, 19 show most displacements in river bed and left wall after impoundment of the dam reservoir. As displayed in the mentioned Figures, most displacements occur in the middle section and far from dam axis. The highest displacement value is 3.2 cm at section A in the middle wall and at 600 m level. The comparison between the graphs of this stage and previous stages shows that the construction of the dam has supporting role in the model. Fig 20 shows a sample of displacement graphs for rock mass characteristics of section B in equivalent continuum model considering lower values of GSI.

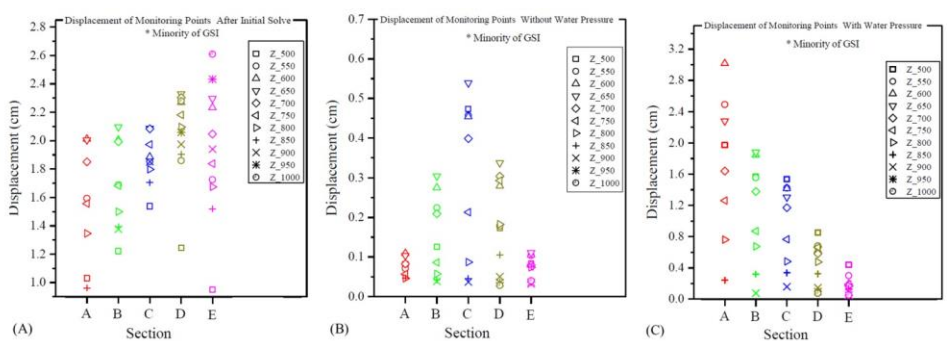


Fig. 18. The displacement variations of observation points a) initial equilibrium b) dam structure construction c) impoundment of reservoir dam on each section in equivalent continuum for lower values of GSI.

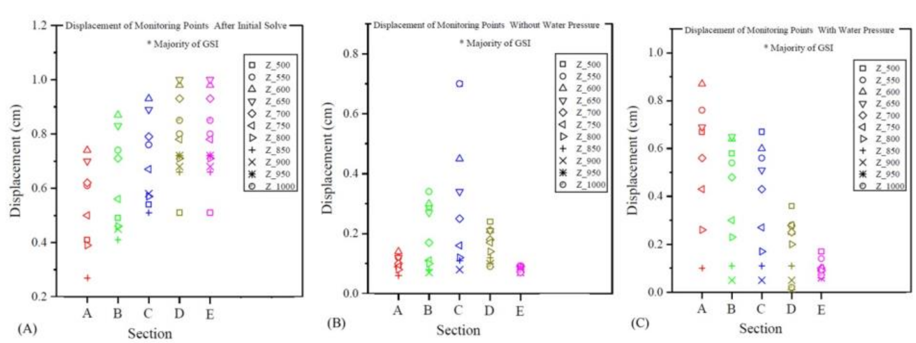


Fig. 19. The displacement variations of observation points a) initial equilibrium b) dam structure construction c) impoundment of reservoir dam on each section in equivalent continuum for upper values of GSI.

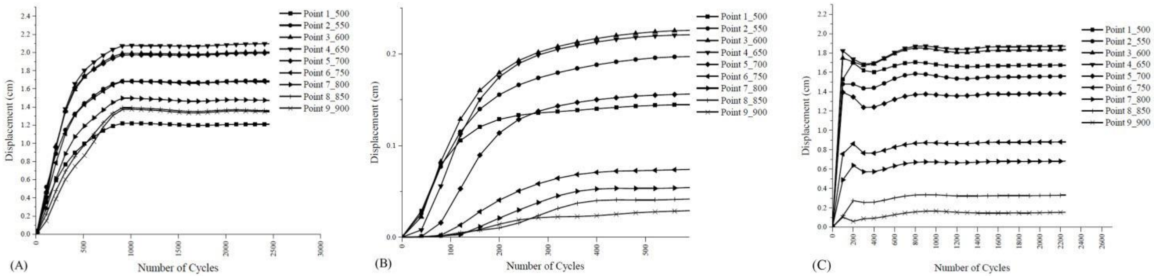


Fig. 20. The displacement history of the points on section B, a) after initial equilibrium b) after dam construction c) after impoundment of dam reservoir in equivalent continuum.

Fig 21 shows the displacement of dam body in equi-continous conditions for lower values of GSI. Also, the locations of points on dam axis are shown in Fig 22, and the displacement of dam body is displayed in Fig 23. According to the displacement results of dam body, there is highest value after impoundment of dam reservoir which is related to weaker strength characteristics.

The results obtained from kinematic analysis and limit equilibrium indicate the plane and wedge slide occurs when there is a high cohesion of discontinuities ending up in the possibility of slide and reduction of stability. As mentioned before, disregarding the important parameters of joints, unlimited length of joints, strain and intact rock failure, and moving slide can explain the above phenomenon. Thus, these methods are suitable only for the design of uncritical slopes. In the assessment of rock slope stability using equi-continuous method, it can be said that the displacements are limited and after the calculation, the model reaches a balance and the displacements are invariable. Whereas, in the discontinuous model, the failure is in a moving state, and during the calculations, the blocks continuously affect each other (even small blocks) and the model never reaches a balance. Also, in the discontinuous modeling, due to the defects (simplifying geometry of joints), it is necessary to introduce realistic methods for distribution of failure in rock mass. For this reason, to reduce uncertainties, the geometry of discontinuities is modeled according to the statistical methods.

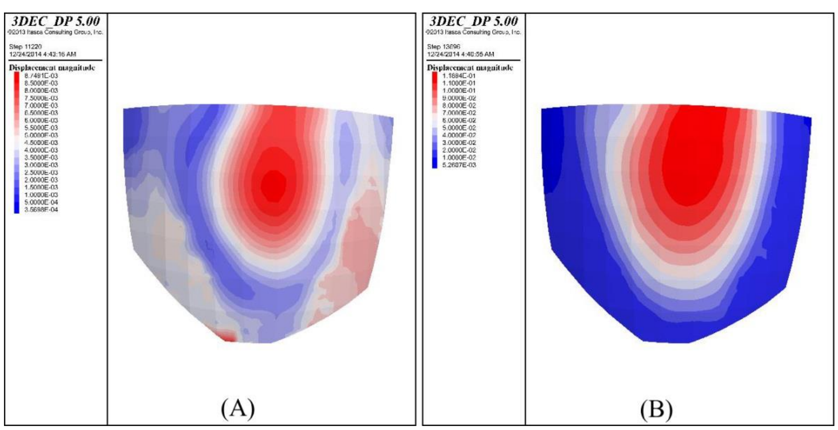


Fig. 21. The displacement of dam body in equi-continuous condition
a) before impoundment of dam reservoir b) after impoundment of dam reservoir.

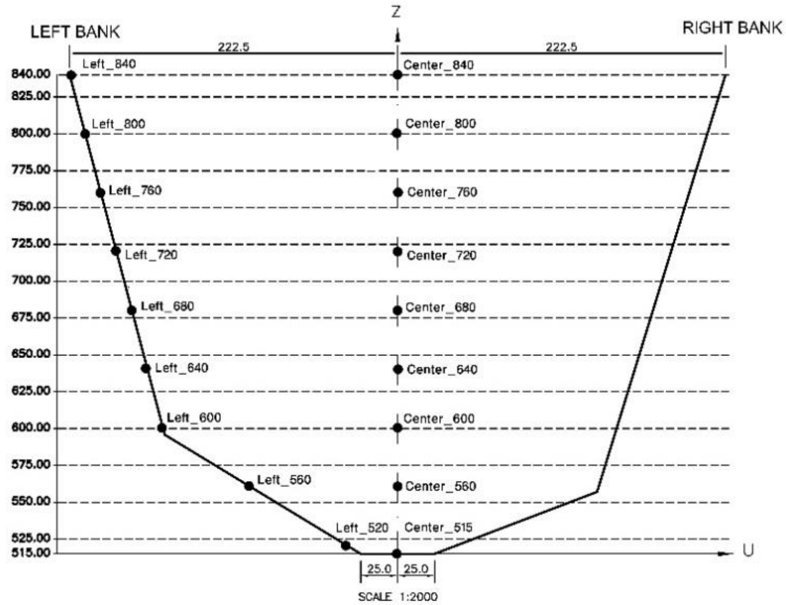


Fig. 22. The locations of points on dam axis for recording displacement history.

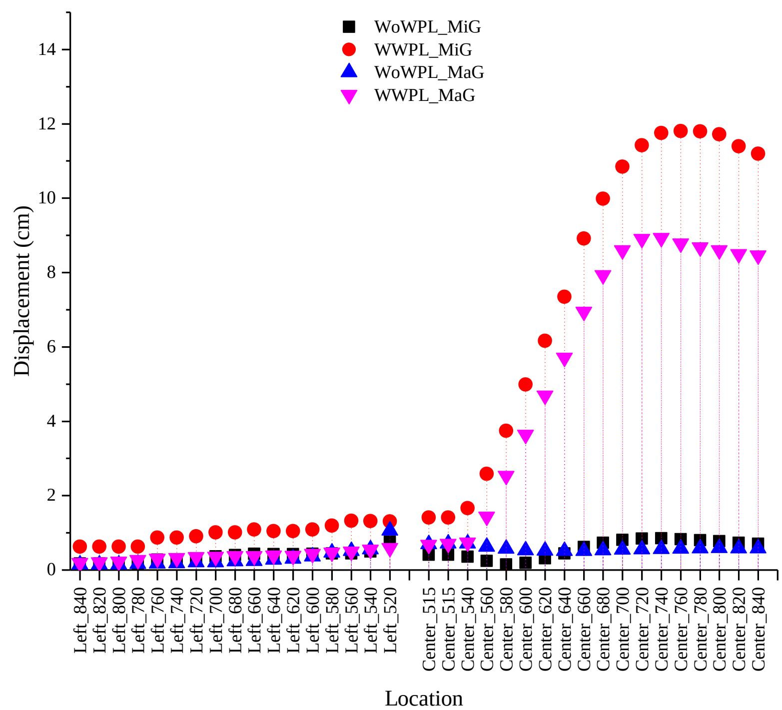


Fig. 23. The displacement of dam body.

9. Conclusions

● Considering the principal structures of geological site, the folding of Siahkoh is recognized as the geological structure which affects the rock mass of dam structure. This folding creates joints J_1 and J_2 throughout the dam structure

● There is no toppling failure at the left abutment of rock slope as shown by the kinematic method. The plane failure in the middle and upper walls is possible, due to joint set J_1 . According to the intersection of joint se J_1 and bedding plane, it is possible that the wedge failure occurs at the upper wall.

● According to the limit equilibrium method, the least safety factor values for plane failure of joint set J_1 at upper wall in static and dynamic conditions are 11.33 and 10.54, respectively, which emphasize the stability of left abutment.

● In the numerical analysis of slope stability using equi-continuous medium method, when the low values of rock mass strength characteristics are considered, the highest values of displacements in initial balance of slope, dam body, and the impoundment of dam reservoir are 2.6 cm for section E, 0.55 cm for section c, and 3 cm for section A. Where the high values of rock mass strength characteristics are considered, these values are 1 cm for sections D and E, 0.7 cm for section C, and 0.87 cm for section A.

● According to the numerical analyses of slope, the left abutment of rock slope is recognized as instable. Therefore, it needs a support system to get stabilized.

● Using distinct element method, the instable block volumes based on the permitted displacements of conventional rock bolts were specified, and it was found that the rock block volumes show good fit with Weibull distribution function.

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