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Evaluation of Stability of Spillway Gable Roof of Nazlu Dam via Slide Software

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ABSTRACT: The most prominent issue in the civil projects such as construction of dams is providing stable steeps of rocks and stones. Such studies aim at providing a gable roof's stability coefficient in order to provide stability as well as an economical solution. This survey wills analysis the gable roof's stability overlooking to the overflow body of Nazlu dam. According to results of exploratory boreholes related to the bed and results of investigation of particles existing on the gable roof, it can be deduced that crunch levels of gable roof's stones, their oxidation, continuation rate (the extent or duration of discontinuities), level spacing (the amount of vertical spacing between adjacent discontinuities of a joint sets), and patency rate (amount of vertical spacing between adjacent walls of a discontinuity) is variable between minor to moderate cases, minor to moderate cases,1 to 10 mm , 0.2 to 1, 2 to 10 mm, respectively. Filling-in rate of the stones is made of $CaCo_3$ and sometimes calcic or clay. The shale part of the gable roof is severely crunched which indicates its instability. The result of instability analysis linked to the gable roof via the slide software (Version 6.0) on the three cutting section AA' (initial section of overflow part), BB'(median section of overflow part) and CC' (terminal section of overflow part) showed that most suitable stable steep will be AA', BB' cutting section between 45 to 60 degree and CC' between 40 to 45 degree. To create such steeps the need of extreme stone lifting is incontrovertible. But instead the desired steep will be provided and those lifted material will be used in the body construction of the dam.

Keywords: Limit Equilibrium Methods, Nazlu Dam, RDQ Index, Stability of Slope

ORIGINAL ARTICLE

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INTRODUCTION

The slope stability and excavatability of rocks is an important problem in geotechnical engineering. This holds for both the design and construction stages. Currently, a number of methods are being used for the assessment of slope stability and excavatability (Goodman, 1989).

One of the practical issues in dam engineering related to the slope stability of embankment dams; however the importance of such surfaces should be designed more compliance with the principles of an economic point, have enough safety and make sure that slope must be stabled (Rahimi, 2012).

Stability of dam means maintaining a balance and prevent to movement of components of a dam against the static forces. It is observed that the stability of the dam is a relative problem and by changing the ratio of the destructive forces to the resistance forces, different degrees of stability can be existed. And in dam designing, the stability of slope is measured by safety factor criterion (Ghaffari, 2010). One of the important problems in slope stability analysis is determining the factor of safety. For example, neither factor of safety exceeding one of guarantees stability for such type of mass movement, nor the value less than one always implies failure (Petley et al., 2008).

Kinematical, limit equilibrium and numerical analyses are generally preferred for the evaluation of rock. Kinematical analysis refers to the motion of bodies without reference to the forces that cause them to move (Kirsten, 1982).

Numerical analyses such as finite element and distinct element methods are performed to confirm results occurred from kinematical and equilibrium analysis. A number of methods have been suggested by researchers to examine the excavatability of rocks (Pettifer and Fookes, 1994; Mintyand Kearns, 1983).

Aryal (2009) evaluated slope stability by Limit Equilibrium Method (LEM) and Finite Element Methods (FEM). The principal difference between these two approaches showed that LEM is based on the static of equilibrium whereas FEM utilizes the stress-strain relationship or constitutive law. Naeibzadeh et al. (2011) investigated the static analysis of the geometry of clay dams. They proposed that dams with inclined clay core show appropriate behavior against stress and forces due to leakage for different condition of loadings. Shamsaei et al. (2010) presented the effect of drainage systems for assessment of safety factor in stability of Brenjestank dam.

These upstream drains are capable of draining the upstream slope and making the equipotential lines tend to become horizontal. They have a very significant effect on the stability of the upstream slope during drawdown (Tran, 2008).

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Traditional limit equilibrium methods will be utilized in the slope stability analysis and the accommodation of saturated and unsaturated pore-water pressures will be considered (Fredlund and and Tiequn, 2011).

The scientific purposes of design include following items: Determining Geographical and geological characteristics of Nazlu dam region, determining engineering, geological characteristics and Geotechnic treats of location of dam, determining the features of joints existing in the stones of gable roof, detection of those factors that lead to instability of gable roof, specifying the stability rate of various section of gable roofs, identifying the best method for stabilization of gable roof. The practical purposes of this design include stabilization of that part of gable roof which faces toward the body of the overflow section of dam and reducing risk rate stemming from the unsuitability of a gable roof.

MATERIAL AND METHODS

Nazlu dam is located on the Nazlu River in the northwest region of Iran named West Azarbaijan and its distance to Urmia city is about 28 km. This dam is near mountains between the desert of the west Urmia Lake and hillside of the eastern region of the border between Iran and Turkey, called Targavar. The location of the dam has following geographical coordinates: latitude coordinates are 37 degrees and minutes to 38 degrees and east length coordinates are 44 degrees and 30 minutes to 45 degrees and 15 minutes. The main materials used in construction of Nazlu dam or gravelly soil with vertical clay core, and its height from the bed of the river is about 98.5 m, the length and width of dam crest is respectively about 420 m and 12 m. The width of the base is 480 m and the total capacity of the reservoir is 170 million m^3, which is able to hold about 230 million m^3 within itself. Figure 1 represents a view of overflow section.

Beside that we investigated the structure of stones, we also put effort in surveying the kinds and treats of discontinuities existed in the stones of a gable roof and the results implied that these discontinuities along with the various orientations and directions have affected the right abutment. These discontinuities are mostly free from any displacements and crunches and among these discontinuities only fault f possesses displacement. This fault has placed calcareous masses on the sale section, so we observed crunches and displacements most in the shoot part of the spillway. Consider Figure 2 as a view of fault from the outlet of the tunnel.

According to boreholes beneath the stillway which have 30,40 and 51 m depth, physical characteristics of discontinuities are studies and we observed that Crunch levels gable roof's stones, their oxidation, continuation rate (The extent or duration of discontinuities), Level spacing (the amount of vertical spacing between adjacent discontinuities of a joint sets), And patency rate (amount of vertical spacing between adjacent walls of a discontinuity) is variable between minor to moderate cases, minor to moderate cases,1 to 10 mm , 0.2 to 1, 2 to 10 mm, respectively. Filling-in rate of the stones is made of $CaCo_3$ and sometimes Sylyt or Clay. Figure 3 illustrates a view of barehole discovery box named $BHNT_1$ which was placed in21 to 27 depths.

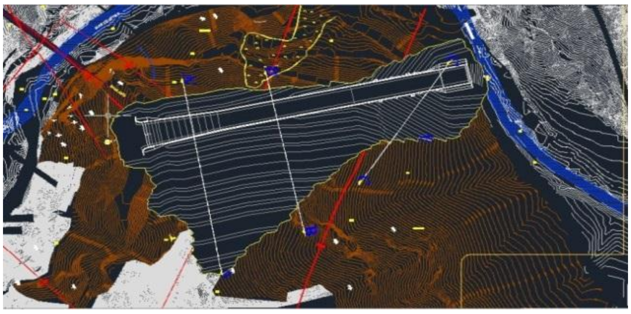

Figure 1. A View of the spillway of Nazlou dam

Figure 2. A view of fault f from outlet of tunnel

Figure 3. A view of barehole discovery box named $BHNT_1$ which was placed in21 to 27 depths.

In order to survey the structural patterns of the rock mass, we started with removing discontinuities which can be referred as linear, removing of discontinuities, within this approach a series of well-ordered measurements along a line of removing along the walls of the staircase or outcrop is being done and all discontinuities which disconnects the removing line, their characteristics like orientations, length, coarseness, kind and the thick of filler is being recorded.

The method that is used in classification of geomechanical rock mass RMR, Geological Strength Index GSI and Rock Mass quality dial RQD is among those stable analysis methods which are based on experiences that is grounded within different ilk of projects under different conditions. Engineering classification of rock constitutes the base of the experimental methods and procedures and can be considered as a powerful tool in designation. These kinds of classes are mostly based on observations, experience and engineering judgments which are considerably useful in quality assessment of the rock mass. Rock Mass quality dial RQD: This index was presented by Deere in 1963 and is one of most routine indexes which is used during the classification of rock mass in the desert when discovery

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bare holes in created. The value of RQD is provided in present form. RQD is defined as received core that is more than 10 cm divided by the total length of the dig. The high value of RQD is an indication of high quality rock mass.

$$\mathrm{RDQ} = \frac{\text{Sum of core patrs lenght of more 10 cm}}{\text{The lenght of all parts}}$$

According to RQD index, the stones in terms of quality are divided into 5 categories that are presented in table 1.

Table 1. Classification of rock mass based on RQD

Rock Mass Classification	RQD Percent
So bad	0-25
Bad	25-50
Partly good	50-75
Good	75-90
Excellent	90-100

The RQD values of the databases were determined using borehole cores. According to RQD divisions proposed by Deere (1964), due to the joint spacing values increase at deeper levels, the RQD values of database's increase at the deeper levels (Deere, 1964).

Engineering geological properties of the rocks exposed in the study area were determined on the basis of field observations/measurements and laboratory tests. The description of rock material and mass characteristics were based on the ISRM methods (ISRM, 1985).

Assessment of slope stability in rocks is usually done through kinematical analyses, limit equilibrium analyses and numerical methods such as finite element method. If the kinematical analysis indicates that the failure controlled by discontinuities is likely, the stability must be evaluated by a limit equilibrium analysis, which considers the shear strength along the failure surface, the effects of pore water pressure and the influence of external forces such as reinforcing elements or seismic accelerations (Turner and Schuster, 1996). Additionally, the results appeared form kinematical and limit equilibrium analyses are performed using numerical methods to confirm if slope is stable. In this study, the kinematical analysis and finite element method are done for the right and left slopes which will cut at the dam site.

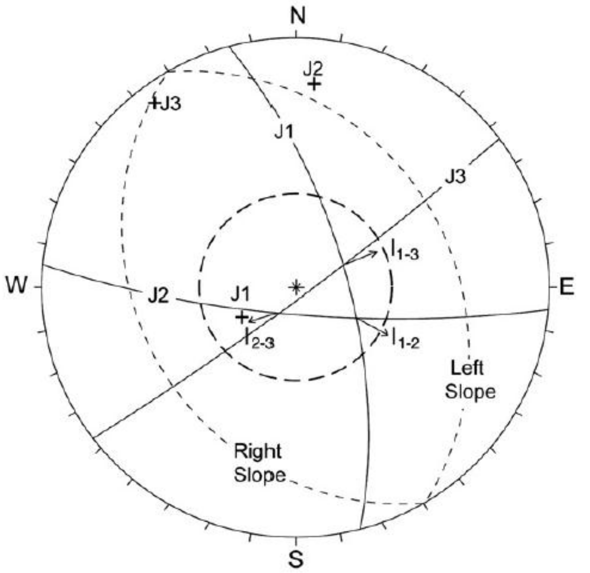

Figure 4. Kinematical analysis of the right and left slopes

Geological Strength Index GSI: Joints and discontinuities are not the only reasons of reduction in controlling the strength parameter of rock mass and creation of unstable conditions that lead to non-structural instabilities, but also along the formation of blocks and rock sections these joints and discontinuities can cause structural instability potential (slippy blocks, upset blocks and etc.). So, this index was introduced by Hook Kizerbawn in 1995 in order to propose a kind of system for evaluation of strength reduction in rock mass during diffirebet geological conditions.

Method of rock mass classification: In 1973, Benyaviski who is the member of industrial and scientific center of Africa (CSIR), offered a new classification method which is based on geomechanic treats of stones, this method is well-known as RMA or CSIR. RMR is among the most successful methods in cetograzation of the rock mass. In this method, the classification is done in terms of following parameters that is:

1. Uniaxial compressive
2. strength of the rock mass (σ_c)
2. Quality index of stones (RQD)
3. Discontinuities distance
4. Discontinuities status
5. The state of subterranean waters
6. The direction of discontinuities in stones

In this classification, the rock mass in terms of structure is divided into several regions and each of these sections is classified individually. The border between these sections is characterized with a patent alteration or fault within the stone (rock). In order to determine the numeral value for each stone in RMR classification method, a numeral point is appointed to each of these factors with regard to their influence on the behavior of the stone. By summing up these six points, we can reach to the value of RMR corresponds to the stone. The parameter that is linked to the orientation of joints is considered as negative factor which reduce the value of RMR. The value of RMR is variable between 0 to 100, and the rock mass in terms of RMR is grouped into 5 categories that are represented in table 2 to describe stone according to RMR index.

Table 2. Description of stone according to RMR index

Description of Stone	Group	RMR Value
So good	1	81-100
good	2	61-80
Partly good	3	41-60
Bad	4	21-40
So bad	5	20<

RESULTS AND DISCUSSIONS

According to the bareholes whose depth are 30,40 and 51, meters and with regard to investigations that are done in this area, we can conclude that the existing discontinuities are divided into three categories named Joints, layering and accidental joints. Because of the formation of such discontinuance and high, steep of the gable roof, we observed the instability potential in weaker sections of shale. By studying the features of these discontinuities within a shale section and calcic parts of the gable roof, it becomes obvious that in the shale stone gable roof maximum and the minimum angle of steep, respectively, is 80 and 18 degrees. The maximum value for direction of steep is equal to 200 and minimum value for this variable is 105. Oxidation is varying between minor to moderate. The Maximum and minimum value for patency is equal to 2 to 10, 1to 2 mm, respectively. The maximum and minimum value for continuity is 2 to 10, 1 to 3 mm, respectively. Also, spacing extreme values is 0.5to 1 and less than 0.5 m. The coefficients of adhesion's maximum and minimum values are 0.02 and 0.01. The maximum and minimum value for coefficient of friction is 32 degree and 28 degree, but in the calcic-dolomite section these values according to table4 are as followings:

Maximum and minimum values of steep degree are 80 and 30 degree, respectively. Maximum and minimum values of steep direction are 255 and 30, respectively. Oxidation is between minor to high. Maximum and minimum values of patency are 5-10 mm and 5 mm, respectively. Maximum and minimum values of continuity are 50-200 and 5-10, respectively. Maximum and minimum values of spacing are 1.5 m and 0.5 m, respectively. Maximum and minimum values of coefficient of adhesion are 0.02 MegaPascal and 0.01MegaPascal, respectively. Maximum and minimum values angle of friction are 43 and 38 degree, respectively, and filling-in rate in both shale and calcic-Dolomite sections are mostly made of $CaCo_3$ and occasionally calcite or clay.

Equal to 55.69 (moderate class) is related to the NT_3 bore from 0 to 51 depth and the minimum correspondence whose value is 16.9 (extreme minor class) is related to the NS_2 bore from 0 to 40 depths.

Table 3. The results of measurements in shale gable roof stone

Row	Discontinuities type	Angle of Steep	Direction of Steep	Oxidation	Patency (Mm)	Continuity (Mm)	Spacing (m)	Adhesion Coefficient (Mpa)	Angle of friction (φ)
1	Layering	18-20	205	Low	5 <	2-5	0.5 <	0.02	28
2	Joint	74	105	Moderate to low	4 <	2-4	0.5-1	0	31
3	Joint	80	187	Moderate	1-2	2-10	0.2-1	0	32
4	Joint	79	288	Low	2-10	1-3	0.5 <	0	32

Table 4. The results of measurements in calcite-Dolomite section of gable roof

Row	Discontinuities type	Angle of steep	Direction of Steep	Oxidation	Patency (Mm)	Continuity (mm)	Spacing (m)	Adhesion Coefficient (Mpa)	Angle of friction (φ)
1	Joint	80	70	High	<5	20-150	0.5-1	0.01	43
2	Joint	70	30	Low	2-5	50-200	<1.5	0.01	41
3	Joint	56	255	Low	2-5	5-10	0.5-1	0.02	40
4	Layering	30	210	Moderate	5-10	5-10	<0.5	0.02	38
5	Plenty Accidental Joints				2-5		<0.5	-	-

Table 5. Rock quality index (Dila) values linked to the different exploratory drilled boreholes

Bore Name	Depth	Rock Mass quality Dial (RQD Index)			Quality
		Minimum	Maximum	Total Average	
NT_1	0-30	17.67	61.83	33.91	Weak
NT_4	0-40	0	64	33.72	Weak
NT_3	0-51	22.5	75	55.69	Moderate
NS_2	0-40	0	48	16.9	Very weak
NS_3	0-40	15.43	57.57	35.07	Weak

Investigation of geological engineering and Geotechnical characteristics of the stone section of gable roof mostly is done by the use of results from experiments that are done on calcic- Dolomite stone, sandstone and shale samples of roof linked to the core of drilling boreholes, Moreover, we have used Hook approach, GSI parameter estimation with regard to the structure of rock mass and discontinuities conditions to determine the strength and deformation parameters of the rock mass. With regard to table 6 engineering categorization of rock mass of gable roof) and Table 7, value of resistance and deformation parameters linked to calcic-stone and shale masses involving gable roof in terms of its height, it has been demonstrated that the maximum value of RMR and GSI, respectively, is 65 and 58 linked to the right abutment of dolomitic limestone and minimum value of RMR and GSI is 26 which is linked to the all sites whose structure is made of shale sets.

In Table 7, the maximum and minimum value of high of steep, respectively, is 140 m and 20 m. The value of experimental parameters, such as the value of uniaxial compressive strength (resistance) σ_C within the calcic - Dolomite section and shale section is 100 and 30 MegaPascal, respectively. The value of mi within calcic-Dolomite and shale sections is 9 and 6, respectively. The value of the modulus of elasticity (E) within calcic and shale sections is 50 and 7.5 MegaPascal, respectively. Geological strength index's value for calcic-Dolomite in the range of 55 to 65 and in the shale section in the range of 30 to 35. In terms of strength parameters, the maximum and minimum value of cohesion (c) is equal to 1.25 MegaPascal and 0.15 MegaPasacal, respectively. The maximum and minimum value of internal coefficient of friction is equal to 54 and 10.8 degree and finally the same values for Deformation modulus of plasticity are 9.2 and 0.2.

Table 6. Engineering classification of rock mass related to stony gable roof.

Stone type	Situation	RMR	GSI
Dolomite calcic	Oxided stones within the stony bed of river with depth of 25 m	35	40
Dolomite calcic	Left Abutment	53	50
Dolomite calcic	right Abutment	65	58
The Shale Set	At all Sites	26	26

Table 7. The values of strength parameters and deformability of calcic-stone and shale masses involving gable roof in terms of its height

Row	Zone title	Situation	Steep height (m)	Experimental Parameters			GSI	Strength parameters		Module Plasticity (Mpa)
				$_c\sigma$ (Mpa)	m_i	E (Gpa)		C (Mpa)	Φ (°)	
1	Lim-AA	Calcareous rock AA' Cross	140	100	9	50	55-65	1.25	43	9.2
2	Lim-BB	Calcareous rock BB' Cross	100	100	9	50	55-65	1.1	45	9.2
3	Lim-CC	Calcareous rock CC' Cross	20	100	9	50	55-65	0.75	54	9.2
4	Sh-CC	Shale Rock Mass CC' Cross	50	30	6	7.5	30-35	0.15	25.4	0.3
5	L-Sh-Fault	Shale stones - limestone fault zone	General	30	6	7.5	20	0.30	10.8	0.2

After putting effort in investigation of total obtained data from exploratory bore holes, collection of joint-print the mechanical condition of the stone became evident and the dimensions of spillway in terms of height, direction of trench parapet and the lithologic state of the spillway's path that includes three cross sections named AA', BB' and CC' according to maps 5, 6 and 7 was determined. Within these cross sections the structural state, various rock mass classification and the function of fault is displayed.

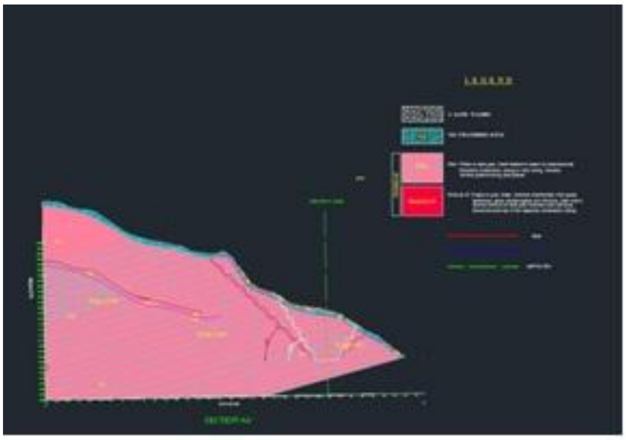

Figure 5- The cross-sectional plane AA'

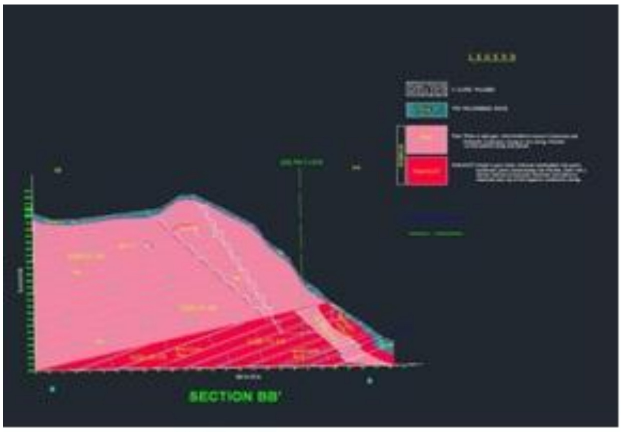

Figure 6. The cross-sectional plane BB'

According to prepared maps, the initial lengths to the median section of the spillway and the initial section of spillway shooting part (cross sectional AA' and BB') are made of calcic-stone mass and calcic-dolomite. As we approach toward the end of the spillway section (mostly situated on CC') the shale stones increases, when it is compared with calcic. As the cross sections 'BB and CC' display, because of Fault F, the calcic stones are located on shale layers. With regard to the structural conditions of rock mass involving parapets, instability potential of cross sections like AA', BB' and CC' according to investigation shapes and possible instabilities are identified. So, by considering measurements and geometry treats of steep and strength parameters, the discontinuities levels and rock mass, the analysis of stability was done by Slide software. It should be pointed out that, the stability analysis is done in static and quasi-static states. According to seismicity report, the numeral value of horizontal acceleration was equal to 0.3 g (equal to level of seismic design). It is worth stating that security factor in static analysis and quasi-static analysis was based on Din 1054, 1.5 and Din 4084, 1.1 Standard, respectively.

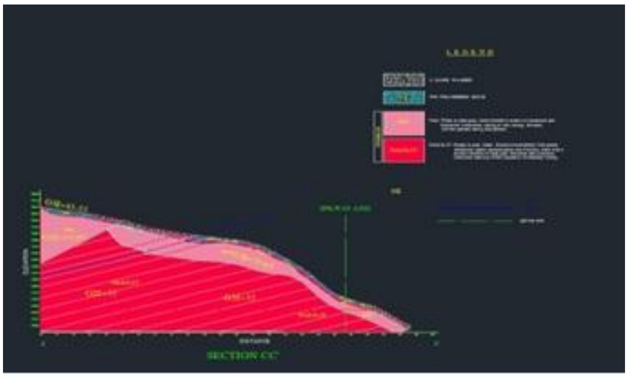

Figure 7. The cross-sectional plane CC'

The result of Stability analysis of the equilibrium point via Slide Software on AA', BB' and CC' cross sections

In this case the value of AA' and BB' for parapet steeps were 45 and 60 degree and security factors in static and quasi-static states for the AA' and BB' cross sections are presented in table 8 which it can be deduced that the sum of the exerted forces was constant. But, in quasi-static case, the force that is due to Horizontal acceleration of earthquake cannot be ignored. The average of confidence in a static state in AA' is equal to 4.39 and in quasi-static state is equal to 2.77. This parameter on BB' crosses section in static and quasi-static states are equal to 2.96 and 2.82, respectively.

Table 8. The results of non-structural stability analysis on AA' and BB' cross section via Slide software

Row	Treats of Parapet		Treats of Stairs			Security factor of AA' cross sectional		Security factor of BB' cross sectional	
	Steep	Height	Twirl Width	Height	Steep	Static case	quasi-static case	Static case	quasi-static case
1	45	140	5	10	63	4.4	2.7	3.9	2.0
2	60	65	5	10	84	4.4	2.7	3.9	2.1

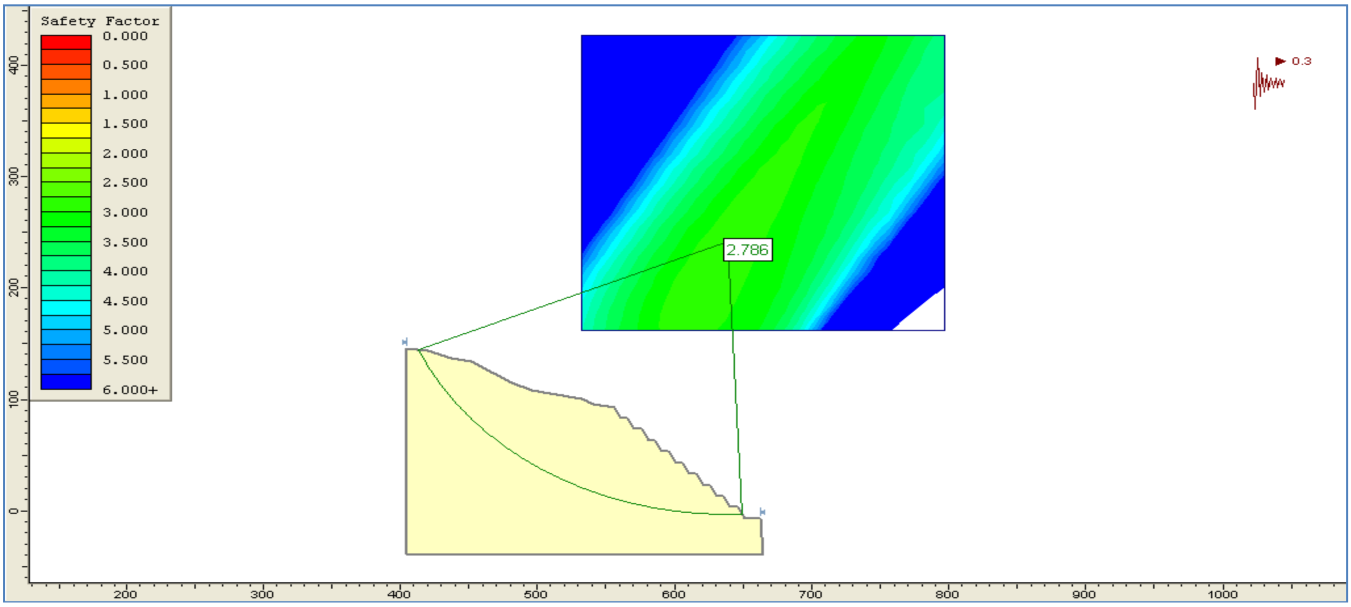

Figure 8. The results of quasi-static analysis on AA' cross section with parapet steep of 45 degrees

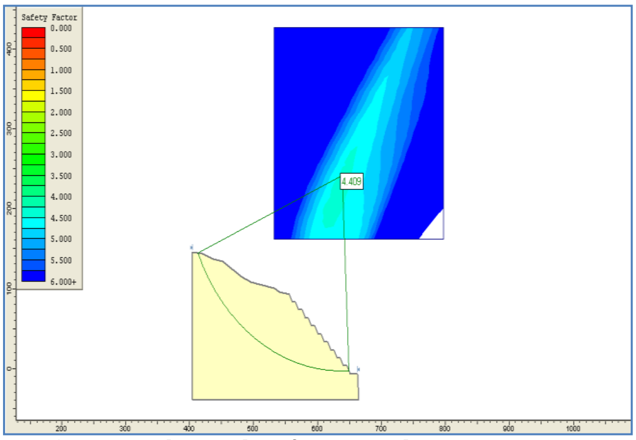
Figure 9- The results of static analysis on AA' cross section with parapet steep of 45 degrees

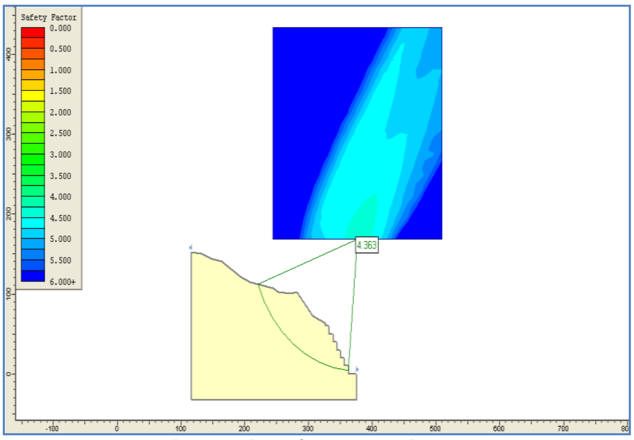
Figure 11- The results of static analysis on AA' cross section with parapet steep of 60 degrees

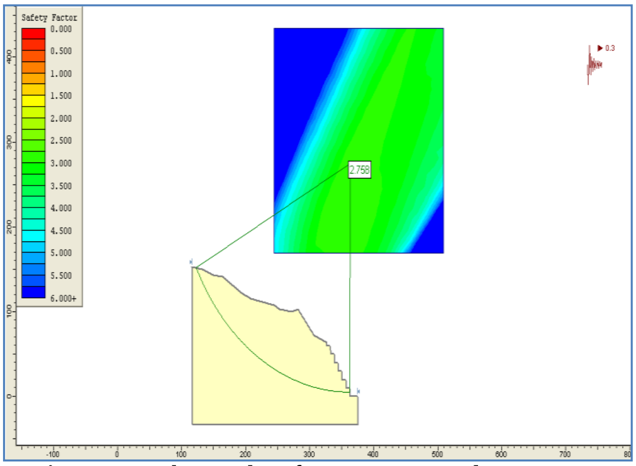
Figure 10- The results of quasi-static analysis on AA' cross section with parapet steep of 60 degrees

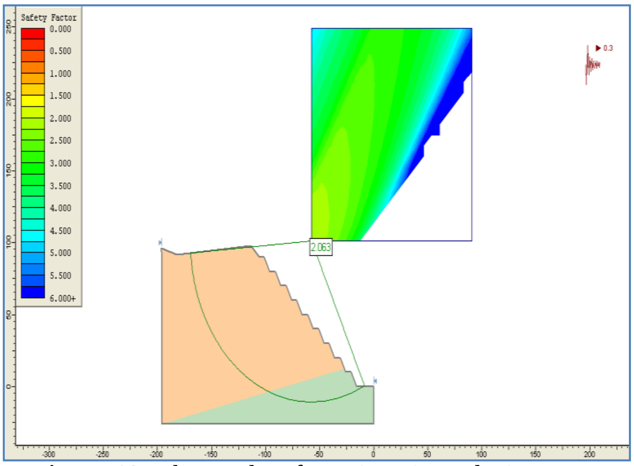
Figure 12- The results of quasi-static analysis on BB' cross section with parapet steep of 45 degrees

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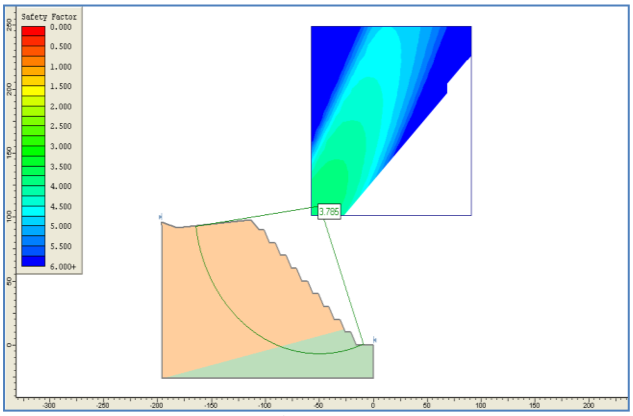

Figure 13- The results of static analysis on BB' cross section with parapet steep of 45 degrees

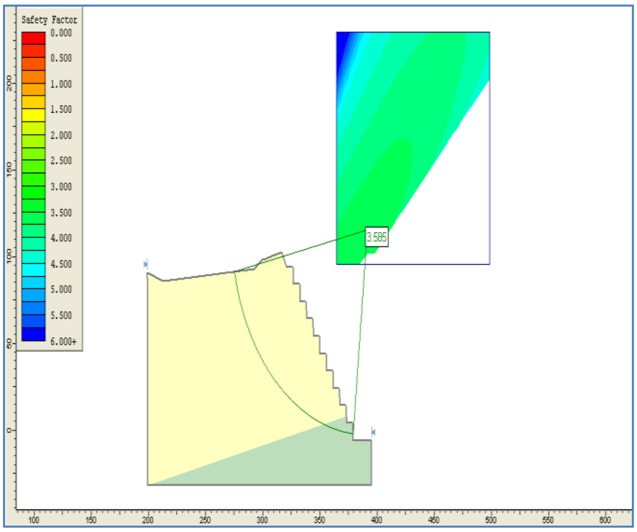

Figure 14- The results of static analysis on BB' cross section with parapet steep of 60 degrees

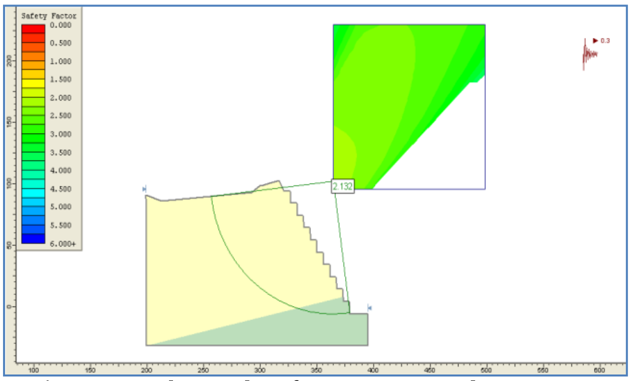

Figure 15- The results of quasi-static analysis on BB' cross section with parapet steep of 60 degrees

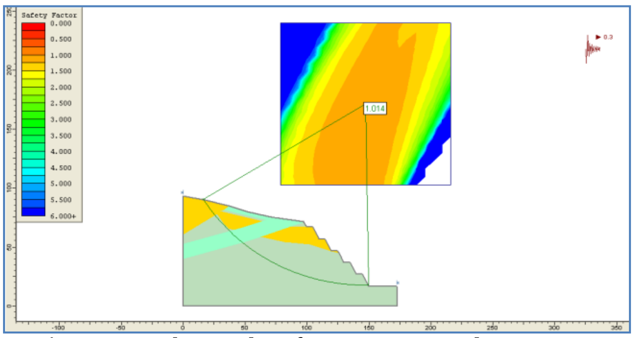

Figure 16- The results of quasi-static analysis on CC' cross section with parapet steep of 45 degrees

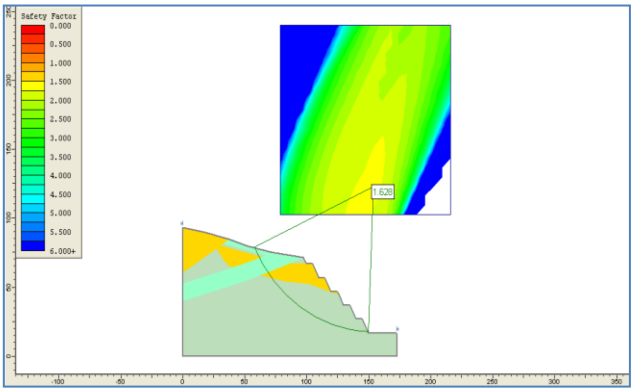

Figure 17- The results of static analysis on CC' cross section with parapet steep of 45 degrees

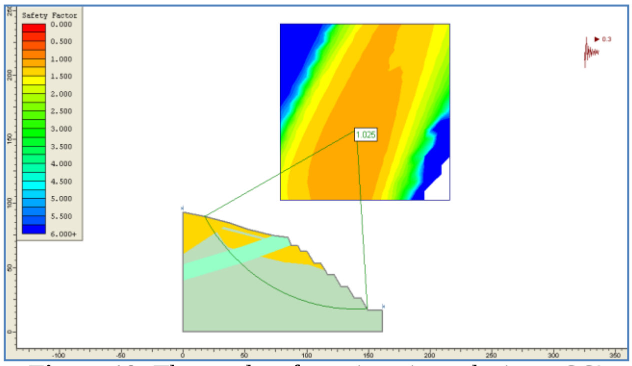

Figure 18- The results of quasi-static analysis on CC' cross section with parapet steep of 40 degrees

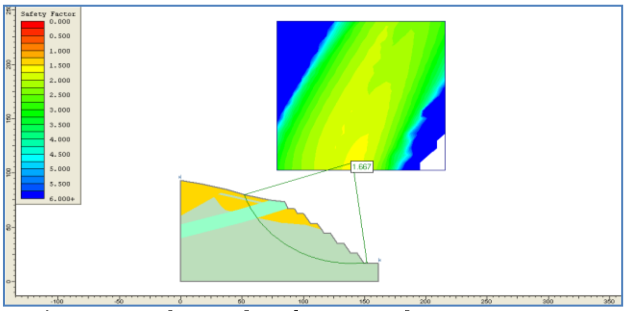

Figure 19- The results of static analysis on CC' cross section with parapet steep of 40 degrees

On CC' cross section the steep of parapet is equal to 45 and 40 degrees, the security factor in static and quasi-static cases for CC' cross section is presented in Table 9. The average of the security coefficient in static and quasi-static cases at CC' approximately is equal to 1.65 and 1.02, respectively.

Table 9. The results of non-structural stability analysis on CC' cross section via Slide software

Row	Treats of Parapet		Treats of Stairs			Security Factor	
	Steep	Height	Twirl Width	Height	Steep	Static	quasi-static
1	45	54	5	10	63	1.6	1.01
2	40	58	5	10	83	1.65	1.02

By considering the status of joints existing within the stones of site of Nazlu dam, it can be concluded that the structure of joints is the most rampant structures in the formation and construction of Nazlu dam, and these joints are mostly noticeable in shale and calcic stones. So, by the formation of such structures and with regard to high steep of a gable roof, the instability potential is formed within weak shale sections. In order to study the various aspects of these phenomena's and the coordinates of these discontinuities existing in the shale and a calcic section of the gable roof, we have used stereo graphic depiction diagrams and statistical method. The amount of crunch of these discontinuities and oxidation is variable between minor to moderate, the rate of continuity spacing, Disruption is variable between these values, 1 to 10 mm, 0.2 to 1 m and 2 to 10 mm, respectively. Filling in the rate of discontinuities is mostly made of calcic or occasionally silt and clay. By paying heed to engineering quality of Nazlu dam's site it became evident that since the spillway of the dam is located in the right abutment within the stony bed which is made of calcic –dolomite, So, RQD and RMR indexes is equal to 35 and 65, respectively. Moreover, the average value of GSI in calcic masses is variable between 55 and 65. This parameter value for shale and calcic stones is equal to 30 to 35. As a result, the stones kind of this area is variable between weak to partly good. Proposals For completion of information linked to the site of Nazlu dam consider following items as suggestions:

1) As it was mentioned the range of total steep should be between 45 to 60 degrees. Since materials such as calcic-Dolomite stones can be used in construction of body of the dam, it is advisable to utilize the low range of steep degree for the papapet.

2) In practical cases, the parapet should be drilled with steep of 45 degrees, also during the execution of the project all executives should consider real conditions of rock mass throughout the revise design. Generally speaking, the best method to increase the stability is the removal of steep head and reduction of rock mass weight which in practice is feasible.

3) In order to thwart oxidation and reduce the parameters of rock mass it is useful to use Shot Crete with a diameter of 7.5 to 10 cm along with a bolt network (with spacing of 2.5*2.5 length of 4 to 6 m) which acts as needed involve between Shot Crete and ground.

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