Application of Screening Analyses for the Stability of Landslide in Seymareh Dam Project

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APPLICATION OF SCREENING ANALYSES FOR THE STABILITY OF LANDSLIDE IN SEYMAREH DAM PROJECT

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ABSTRACT

Seymareh dam and hydropower plant project is located in western province of Ilam, in Iran. This concrete arch dam with 178m height and 196 m crest length is to be placed on Seymareh River, the axis of which is located on the north wing of an anticline including Asmari limestone layers. The dramatic variation of dip angle of beddings in this part of anticline has created many fractures that result in slides. In October 1st 2003, when excavation in the left bank was being operated to construct an access road to the valley, a large landslide triggered on the top of power tunnel intake. The landslide completed in November 10th 2003 few hours after an intensive rainfall by leaving some 300000 m^3 mass of debris. The sliding was a planar failure on a thin marly limestone of 26° dip and 40 cm thick. Due to the future location of water intake structure at the entrance part of power tunnel, it was quite vital to save this structure from any further movements of the debris left by the landslide. Further investigations showed that is possible to keep debris in place by flattening the material in forms of stable slopes and berms. In order to decrease the remedial costs, the new recommended method of screening analyses for seismic stability proposed by SP117 Guideline in southern California Earthquake Center (2002) was adopted. Finally using this method, a slope design with almost minimum costs was achieved. This paper contains a description to geotechnical conditions, the history of landslide, and remedy works to protect water intake structure.

INTRODUCTION

During civil engineering activities on mountain areas, it is very important to recognize the most key factors affecting the slope stabilities. In the case of arch dams which usually placed on rocky foundations with considerable resistance and rigidity, the planar and wedge failure is a common problem. The factors which may govern the rock slope stabilities in such natural grounds are:

-Marginal stability in natural condition
-Previous movements during older landslides
-Possible low shear resistance of discontinuities such as those filled by marly and clayey materials
-Existing tension cracks
-Excavation in region of slope toe
-Ground water due to the precipitation
-Uncontrolled explosions in excavation activities

On the other hand, one of the major concerns of the engineers involved in design and evaluation of slope for the countries located in highly seismic areas is how to make a design with adequate safety against earthquake-induced instability. Due to the catastrophic consequences of this slope failure on dam performance, it must be able to resist their design base earthquake. The conventional evaluation method of slope for earthquake loading is based on equivalent static approach in which a horizontal seismic ratio is applied on rigid sliding masses in limit equilibrium method. In this approach seismic stability is assessed by the so-called safety factor which is the minimum ratio of resistance forces over driving forces on all kinematically possible sliding surfaces. The method assumes that the blocks are rigid and sliding failure surfaces behave in a perfect plastic manner. There are two elements associated with a pseudo-static stability analysis procedure. First, a horizontal destabilizing seismic coefficient must be specified and second a minimum acceptable factor of safety should be determined. The Seed (1979) procedure calls for k=0.15 (proposed for highly seismic areas) and FS=1.15. This seismic coefficient can lead to underestimate slope stability if liquefaction is not probable and result in higher cost for flatter slope. The SP 117 Guidelines (2002) modified Seed criterion to more properly account for the effect of seismicity on slope deformation hazard. This method is called Screening analysis of slope and applied in the landslide remediation problem of Seymareh project.

Seymareh dam and hydropower plant project is nearly located 25 km far from the northwest of Darreh Shahr city in Ilam province. The objectives of the project are to control the upstream water flow, to supply and to distribute water to the neighboring agricultural lands and to generate 835 GWH hydroelectric energy annually. This concrete arch dam with 178m height and 196 m crest length is to be placed on Seymareh River. The construction work started in 2003 and most of excavations have been finished by now.

The dam is to be founded on the north wing of Ravandi anticline, which geologically includes the Asmari formations and Gachsaran formations containing limestone layers and marl-gypsum masses, respectively.

In October 1st 2003, when the excavation in the left bank was being operated in order to construct an access road to the valley, a large landslide triggered on the top of power tunnel intake.Figure 1 shows the landslide zone in the project area.

Fig. 1. Situation of landslide zone adjacent to the power tunnel entrance

The landslide completed in November 10th 2003 few hours after an intensive rainfall. The event created a huge mass of debris by 300000 m^3, in which the size of some rock pieces could exceed 10 m (Fig. 2). The scope of area before and after final collapse is displayed in Fig. 3. The sliding mass can be defined by a tension crack of 20 m height and 200 m long in the above and a sliding planar surface of 26° dip and with about 100 meters horizontal lengths. In the west part of failure zone, where the power tunnel entrance is located in the bottom, many large blocks were stopped unstably. Because of the future location of water intake structure at the entrance part of power tunnel, it was quite vital to save this structure from any further movements of the debris left by the landslide.

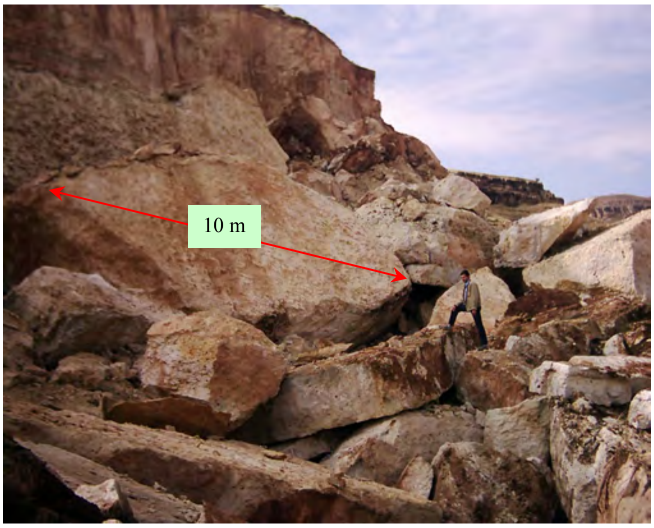

Fig. 2 The large blocks left by landslide

GEOLOGICAL CHARACTERISTICS

<u>Regional Geology</u>

Based on the regional geological divisions of Iran, the project area is located on the southwestern part of folded Zagros. The mountains have a NW-SE trend in this part of Zagros. Also, the morphology is markedly influenced by the structural-geological arrangement of the area. The trend of tectonic structures follows the trend of folded Zagros zone. A succession of rocky ridges (anticlines) and low-lying areas (synclines) is abundant in the region. The maximum and minimum elevation of the region belongs to the great Kabir Kuh anticline by 2700 m.a.s.l. and the plains among the anticlines paralleled to the Seymareh River with 600 m.a.s.l, respectively. The known faults in the area are generally parallel with the axis of the anticlines and occur in their limbs, but sometimes they are perpendicular to the axis of these anticlines.The stratigraphic sequence of the rock units in the area includes the formations belonging to cretaceous up to Polio-Pleistocene. This sequence consists of limestone, dolomitic limestone, marly limestone, reef limestone, calcareous marl, sandstone, siltstone and conglomerate.

Dam SiteG eology

The morphology of the site includes a U shaped nearly symmetrical valley with steep and someplace negative slopes at abutments, crossing the bedding strike perpendicularly. At the gorge of dam site the width of river varies from 40 to 50 m and its trend varies from NW-SE to NE-SW. To the downstream direction, beyond the anticline axis and its southern limb, the river again adopts a NW-SE trend and flows in to the Talkhab plain and finally joints Karkheh River (Fig. 4).

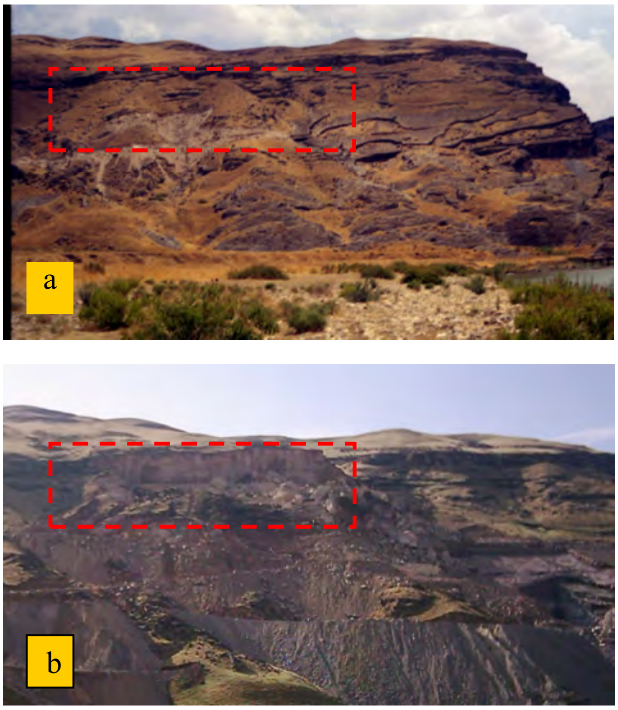

Fig. 3. The view of landslide, a) before and b) after landslide occurrence

The northern limb of Ravandi anticline, where the dam will be constructed, consists of slopes with varying dips, ranging from 45-50 degrees at the start of gorge, to 10-15 degrees at the downstream of dam (Fig. 5). At dam abutments there are small gullies with steep walls formed along the bedding planes and fractures. Lack of vegetation around the dam site has made a rocky nature with a surface showing resistance against erosive factors. The overburden materials in Seymareh dam site include alluvial deposits, Lacustrine deposits, Talus and Old alluvial deposits. The main part of the materials is Lacustrine deposits, which have a 35 m thickness. This deposit consists of greenish gray silty clay with high plasticity & sandy intercalations resulted from the formation of natural lake due to the huge Kabir Kuh landslide. The well-known Kabir Kuh landslide, sometimes so called as Great Seymareh landslide is the largest in the world located 50 km far from the dam site.

Fig. 4.The morphology of Seymareh site, a) U shaped valley, b) Ravandi anticline

Fig. 5. Dramatic variation of bedding dips at the northern limb of Ravandi anticline

The rock units in the area consist of Bakhtiari Fm. (Plio-Pleistocene age), Gachsaran Fm. (Miocene age) and Asmari Fm. (Oligo-Miocene age). The October 2003 landslide in Seymareh dam project has occurred in Asmari formation on the top of power tunnel intake. Figure. 6 illustrates the geological
section of this formation along the power tunnel as well as the location of landslide zone with respect to the power tunnel in

take. In the project area, Asmari formation can be divided into 3 units as follows:

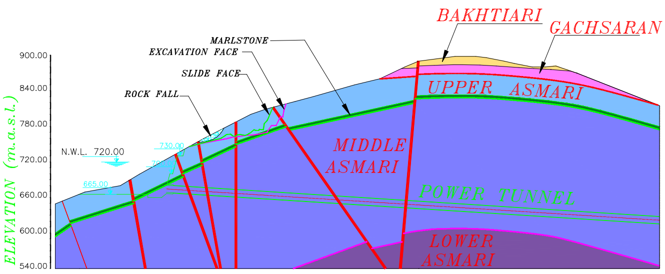

Fig. 6. Geological section of Seymareh project along the power tunnel

Upper Asmari Unit

This unit has outcrops in the high elevations of Ravandi Anticline and at the entrance of canyon. The predominant lithology of this unit consists of medium bedded bioclastic and crystalline limestone, which turns into marly (dolomitic) limestone & porous limestone at the upper parts of the unit. An intercalation of indurated marl marks the boundary between the Upper Asmari unit & the Middle Asmari unit. Moreover, there is an intercalation of sedimentary breccia with a maximum thickness of 1.0 m at the bottom of upper Unit. This unit has 50-55 m thickness in places without any erosion. Several old landslides as well as the new Seymareh dam landslide have taken place in this unit. Power tunnel inlet & outlet, diversion tunnels inlet and powerhouse are also situated in this unit.

Middle Asmari Unit

Most of the structures in Seymareh dam project are situated in this unit. The predominant lithology of this unit consists of crystalline limestone, bioclastic limestone and porous dolomitic limestone. Generally, the Middle Asmari unit is 220 m thick and clearly displays epigenetic karstification throughout itself.

Lower Asmari Unit

This unit has a small outcrop in the outlets of diversion tunnels and near the axis of Ravandi anticline. The unit is composed of medium-bedded marly limestone and crystalline limestone.

Structural Geology

Ravandi anticline is the main geological structure in this area. As mentioned before, the Seymareh dam site lies in the northern limb of Ravandi anticline. The axis of the anticline trends E-W and plunges E at the dam site. In fact, Ravandi anticline is a double plunge fold which it's another plunge in the reservoir area is NW. The axis of anticline in dam downstream has been turned due to tectonic forces. Because of this, the E-W trend of anticline has changed to NW-SE as illustrated in Fig. 4b. This situation has caused more abundant fracturing & crushing relative to that of the northern limb in which the dam body will be situated. The emergence of a great number of springs in the southern limb of the Ravandi anticline can be attributed to this.

The dip of bedding layers increase gradually from horizontal (near the axis of anticline) towards the north (northern limb in which the dam axis will be situated) and finally, reaches 40º at the entrance of the gorge where the northern limb is buried by alluvium. The Ravandi anticline is an asymmetrical anticline with an obtuse peak so that the beds are quite horizontal within a length of about 150 m. The southern limb of the Ravandi anticline is relatively more regular than the northern one. After anticline axis the dips change from horizontal to a maximum 25º at the intersection of the southern limb with the Seymareh riverbed.

In northern limb which the layers are dipper, many tension faults have been created due to the bending action of anticline. Figure. 7 shows the scope of such faults in the left bank which extended to the east. The failure zone arising from the landslide is seen in the Figure. The pre-existing fault F12 played the role of main tension crack for the landslide of Seymareh project. Several normal faults & main joints are recognized in Seymareh dam site, most of these fractures are parallel with strike of bedding & anticline axis. Two sets of major joints and one set of minor joints excluding bedding planes constitute the discontinuities of the rock masses. In the Upper Asmari unit at a distance 200 m east of the gorge and upstream of the dam axis there were dislocated rock masses covered an area of about 250*300 m at elevations 620 to 800 m.a.s.l. These have been brought by old slides occurred along the bedding planes. The event of landslide on October 1st 2003 has occurred in the top of this area.

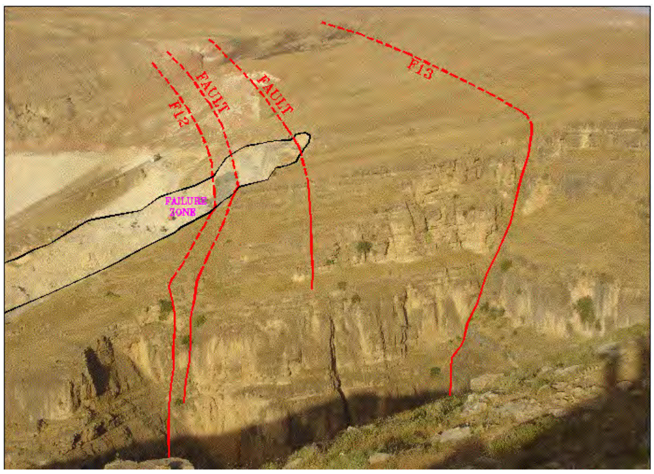

Fig. 7. The extension of tension faults and major joints in the left bank (view from the west)

LANDSLIDE EVENT

Sliding Process

The landslide commenced by appearing a 20m high vertical joint in the top of the region in October 1st 2003. The sliding happened when the excavating operation of an access road to the valley was underway in the left bank at level of 730 m.a.s.l. In this excavation the toe of sliding mass was partially cut. After creation of the main joint and movement of the total mass some other joints were also detected gradually on the lower elevations so that the total area were subdivided into smaller pillars. Figure. 8 shows how the main joint has opened on ground surface during the progressive sliding.

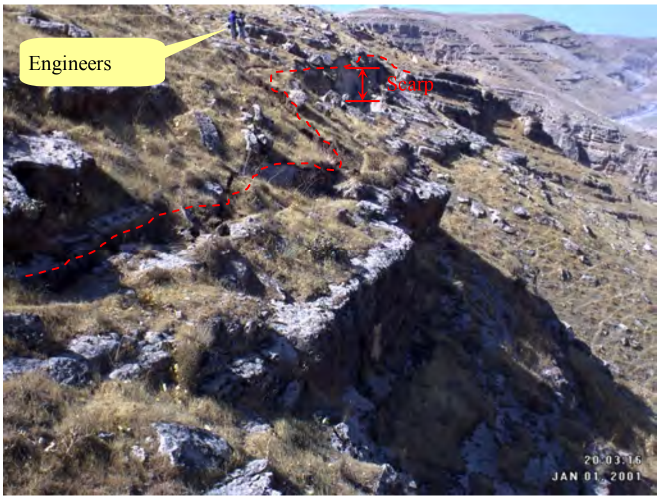

Fig. 8. Trace of main tension joint of the sliding mass on ground surface and resulting scarp

Fig. 9. The appearance of vertical joints in the southern wall of affected area and on the ground surface

The total displacement of sliding mass before the global collapse happened reached approximately to 4m on the main joint location. Figure. 9 illustrates the trace of main tension joint on the ground surface and the resulting large scarp. In the last hours of November 9th, 2003 an intensive rainfall caused the total mass to be saturated. Finally, at the midday of November 10th the whole mass collapsed totally and remained a huge mass of debris estimated by 300000 m3 as shown in Fig. 2 and Fig. 3b. After this final failure, the fault F12 wall was clearly seen in a length of some 200 m with variable elevation from 20 to 40 m (refer to Fig. 3b). It was also observed that a clay filling material by 50 cm thickness is pasted to the most parts of fault F12 face. This indicated that the fault F12 was an old tension crack filled during a long history. After the total collapse of sliding zone some of the big dislocated blocks moved towards the power tunnel intake and stopped unstably on the above of this area as illustrated in Fig. 10.

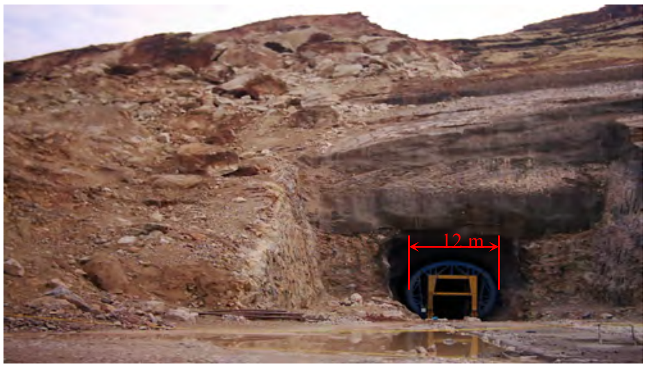

Fig. 10. The view of power tunnel intake and unstable rock blocks in the above after intensive rainfall leading to final collapse

Landslide Reasons

The field study indicated that the affected mass was in the upper Asmari, which slipped on a marly limestone layer with 40 cm thickness. As shown in Fig. 6, this thin layer separates the upper Asmari and lower Asmari. In addition, it was concluded that the main tension joint was a part of pre-existing fault F12 (see Fig. 7). The thick clay filling in this joint as seen in Fig. 8 confirmed that the main vertical joint was not a new crack caused by tension force. It should be noted that before the sliding was detected, there was an excavating operation in the toe of sliding mass. This was considered as the main factor for the initiation of sliding. However, the small dimension of excavated mass compared to the total mass indicated that the safety factor before excavation was close to unity. In other words, the stability of area was previously marginal and excavating toe only triggered the sliding. Further investigation indicated that the above-mentioned landslide is a part of a larger old landslide in the left bank area. The main factors, which contributed in October 2003 land slide of Seymareh project, can be summarized as:

-Pre-existing fault F12 as the main tension crack
-Thin marly limestone layer as the sliding surface with low shear resistance under wet condition
-Marginal stability of area (sliding safety factor close to unity) before sliding happened
-Toe excavation which decreased the shear resistance of sliding surface
-Probable intensive explosion for excavation of access road to the valley

STABILITY ANALYSES

In this section the result of stability analyses of Seymareh landslide are presented.

Geometry

The most critical section of slope which was selected for stability analyses is shown in Fig. 11. On the basis of exploratory boreholes and site visits before and after landslide occurs, three different layers were considered in two dimensional geometric model (Fig. 12).

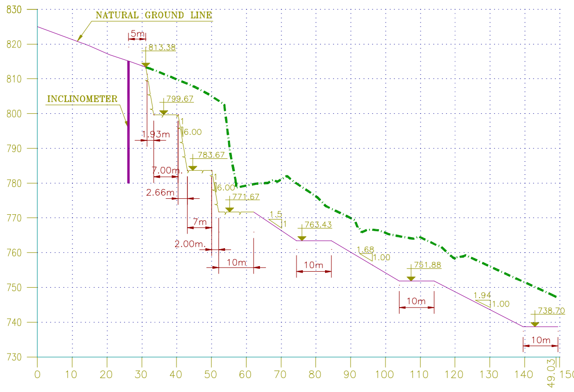

Fig. 11. The critical section of slope

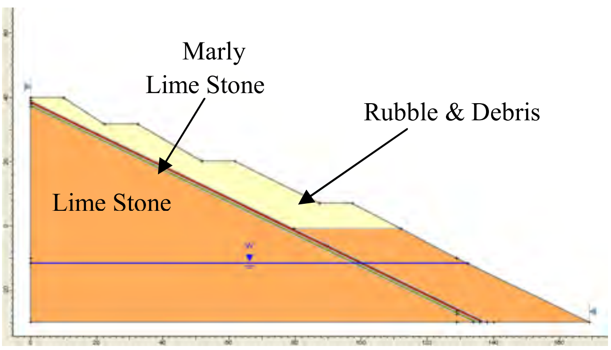

Fig. 12. Model geometry and dimensions

Material Properties

Physical and mechanical parameters of these materials are listed in table 1. Material properties of limestone layer were chosen from laboratory and insitu tests, recommended in rock mechanics report of the project (MGCE-a 2004). Slope stability analysis was carried out after landslide has already occurred. As discussed before, the Asmary limestone layer slipped on marly limestone layer. Therefore, residual cohesion sharply decreased after this landslide and as listed in the following table cohesion of marly limestone layer can be regarded as zero to be in safe side. The residual friction angle of marly limestone layer is selected as 20 degree which is conservatively less than 26 degree of its dipping angle of slope. Also, the average friction angle of Rubble and debris based on measured values in the field is adopted as 38 degrees.

Table 1. Material properties used in stability analyses

Material type	Dry density $\left(\frac{KN}{m^3}\right)$	Wet density $\left(\frac{KN}{m^3}\right)$	Cohesion $\left(\frac{KN}{m^2}\right)$	Friction angle (degree)
Limestone layer	26	27	1000	39
Marly limestone	22	23	0	20
Rubble and debris	23	24	0	38

STABILITY IN STATIC CONDITION

Before any seismic slope stability analysis, examining the static stability of current slope is required. The safety factor is determined using simplified Bishop Method. As the results are presented in Fig. 13, the safety factors obtained in static condition are greater than minimum recommended safety factor. (F.S> 1.5)

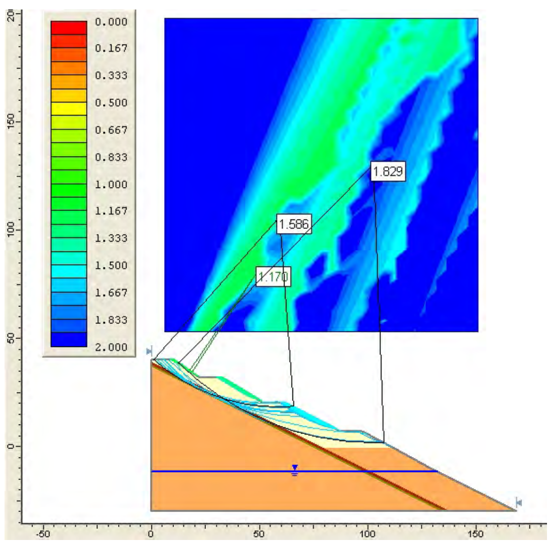

Fig. 13. Result of static stability analysis

STABILITY IN SEISMIC CONDITION

Screening Analysis

For seismic safety evaluation of slope, the simple proposed method of Seed (1979) was adopted in first step. The results shown in Fig. 14, illustrates a safety factor of 1.087 under seismic horizontal force with K_h=0.15. Since the minimum allowable safety factor of 1.15 in Seed method (and even recommended minimum F.S=1.10 of Los Angeles method) is not obtained, the slope design should be changed or another analysis approach must be used.

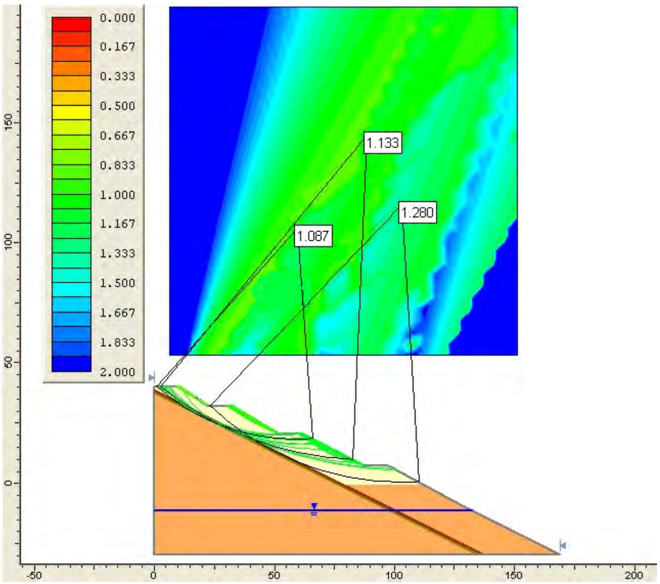

Fig. 14 Calculated factor of safety in Seed criteria by Keq=0.15

In this project, the new method of screening analysis in SP117 Guideline prepared by southern California Earthquake Center in 2002 was adopted in the next step. The screen analysis is a preliminary analysis for slopes within hazard zones. The purpose of this analysis is to determine if slope has a significant landslide potential. The procedure is implemented by applying a horizontal seismic load on centroid of the sliding mass that is equal to fraction of weight of the slope wedge. If the factor of safety is greater than one (FS > 1), the slope is stable and passes the screen, otherwise further quantitative evaluation of landslide hazard potential is requird.

The seismic coefficient (K_{eq}) is estimated by the following relation:

$$K_{eq} = f_{eq} \times (MHAr / g) \qquad (1)$$

In this relation, MHAr is the maximum horizontal acceleration at the site for a soft rock site condition; g = acceleration of gravity; and f_{eq} is a function of magnitude and site-source distance. By developing a range of f_{eq}, some unnecessary conservatism of Seed method omitted. Magnitude and distance dependent f_{eq} values were developed using a model for seismic slope displacement based on a Newmark type analysis. The values of threshold Newmark displacement were used 5 cm and 15 cm.

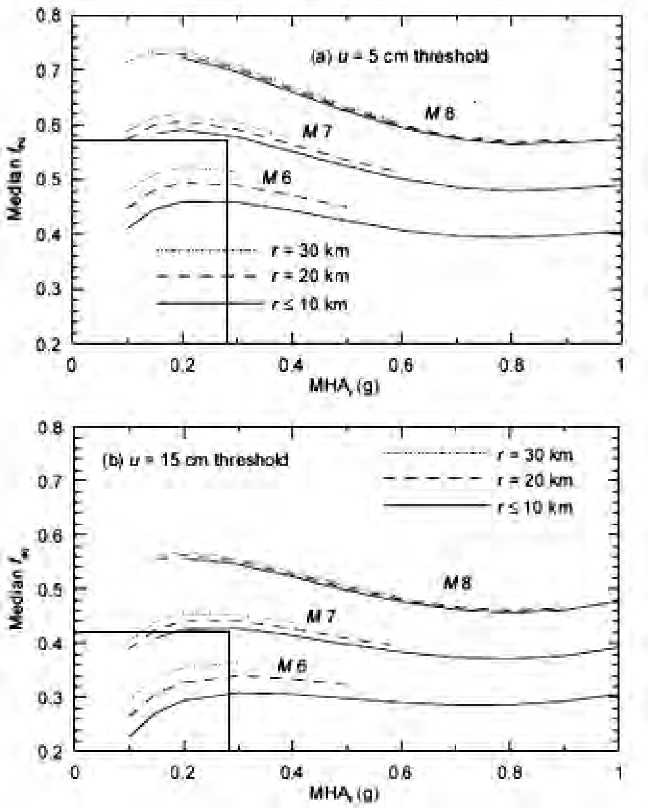

Fig. 15. Required values of feq as function of MHAr and distance from source for threshold displacement of a.5 cm b 15 cm (SP 117 2002)

MHAr and M (earthquake magnitude) are 0.27g and 6.8 respectively, using probabilistic seismic hazard analysis for a 475 year return period. (MGCE-b 2004).This earthquake can be caused by Gowar fault located at 18 km far from the site (r=18). Using Fig. 15 for threshold displacements of 5 and 15 median f_{eq} and K_{eq} are:

$$u = 5cm: \ feq = 0.57 \rightarrow \text{Keq} = 0.57 \times 0.27 = 0.154 \qquad (2)$$

$$u = 15cm: \ feq = 0.42 \rightarrow \text{Keq} = 0.42 \times 0.27 = 0.113 \qquad (3)$$

Pseudo stability analysis was then carried out with these two seismic coefficients showing minimum safety factor of 1.05 and 1.153 respectively for u=5 cm and u=15 cm (Figs. 16 and 17) Therefore, FS>1 and site passes the screen.

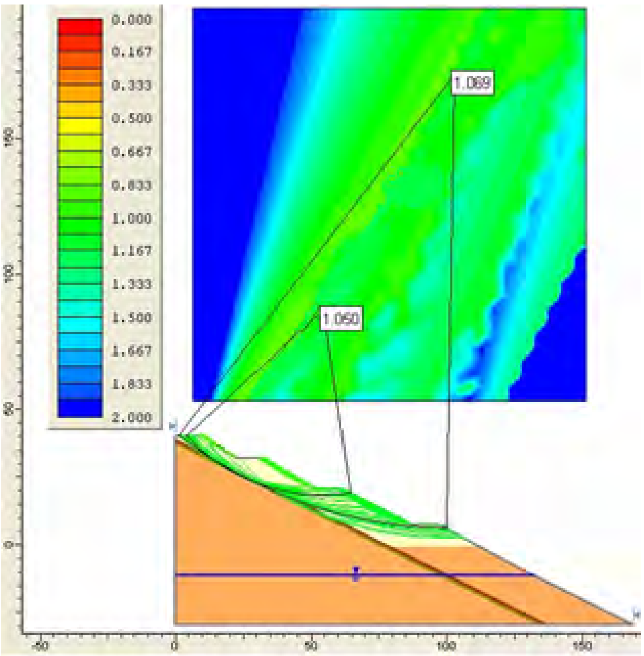

Fig. 16 Calculated factor of safety for threshold displacement of 5 cm and K_{eq}=0.154

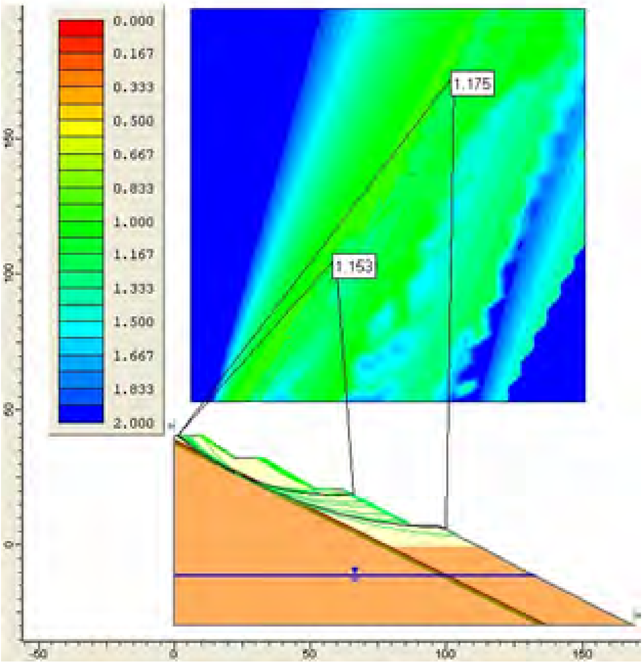

Fig. 17 Calculated factor of safety for threshold displacement of 15 cm and K_{eq}=0.113

Based on the above results, it was concluded that designed slopes and berms in Fig. 11 will be stable under static and seismic conditions.

CONCLUSION

The landslide in Seymareh dam and hydropower plant project which triggered in 1st October 2003 and completed in 10th November was a result of almost all possible factors which undermine a natural. During the landslide a large mass of debris by 300000 m^3 left unstably in the place. Finally, a cut slope and flattening method with reasonable costs was adopted to supply adequate safety for water intake structure below and the access road to valley in static and seismic conditions In this project recommendations by SP 117 Guideline were used to evaluate seismic stability of designed slope. SP 117 recommendations improve Seed criterion for seismic stability analyses.

Though, the minimum factor of safety in conservative method of Seed (1979) is less than F.S=1.15, the results of screening analyses proposed in SP117 indicate that the designed slope is stable. This approach led to have a slope design with reasonable remedial costs.

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