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Evaluating the Behavior of Geogrid-Reinforced Earth Dams under Static and Dynamic Loads

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Abstract

Since many earth dams are constructed already or are under construction in the earthquake prone areas, their safe design against earthquake is very important. The precise investigation of seismic stability of the earth dams is among the complex issues in the dam construction area. Besides, diversity of the dam body dynamic properties as well as diversity of the components and thickness of the foundation is among the factors that play a significant role in the dam dynamic response. Today, thanks to the science and technology progresses, it is possible to develop and construct the earth dam with considerable heights. The most common type of the dams is the earth dam with clay core. The function of the clay core is to seal the dam and to reserve the water. The more important issues in dam designing are analysis of their slope dynamic stability against the seismic loads. Therefore, in this study the seismic dynamic analysis of an earth dam is investigated by the finite element method and the GeoStudio software. The aim of these analyses is to investigate the dynamic response of the earth dams in different heights of the dam core and crest as well as to obtain the optimal position of the clay core slope in designing these dams. Studies in this field can be investigated under different conditions for unreinforced slope sand for those slopes reinforced with geogrid materials. This article is composed of two main parts. The first part, using the GeoOffice software and based on the limit equilibrium method is addressed to analysis the earth dam slope stabilities under the steady state and rapid draw down conditions, with and without the seismic loading. Then, based on the finite element method in the PLAXIS software, the analysis of the slops reinforced with geogrid is addressed. Besides, the effects of the static and dynamic loadings on analyses are considered and the safety factor values in each status are compared with the unreinforced slopes. The obtained results showed that by using the geogrid in the slopes under these conditions, the safety factor is enhanced. Likewise, by comparing these results, it can be concluded that the slope angles of the earth dam are increased and the dam embankment volume is decreased. Besides, through comparing the results, the optimal values to recognize the various geogrids, number of the geogrid layers, vertical distances and length of the geogrids are obtained.

Keywords: stability, safety factor, reinforcement, geogrid, dynamic loading

Introduction

Since many dams are constructed already or under construction in the earthquake prone areas, their safe design against earthquake is very important. Therefore, precise investigation of the earth dam seismic stabilities is among the complex issues in the area of the earth structures (Shelley and Santoyo, 2001). The following factors play an important role in the dam dynamic response: diversity of the dam body dynamic properties as well as diversity of the components and thickness of the foundation, which can play a significant role in transfer, weakening and strengthening of the earthquake waves (Hatanaka, 1995); the presence or absence of the active fault(s) within the dam axis area, the earthquake characteristics such as distance from the earthquake focus to the dam, intensity and duration of the earthquake; type and direction of the waves achieved to the dam; and the frequency content of the waves (Georgiannou, 2001; Mononobe and Matsuo, 1929; Hatanaka, 1995).

Considering the importance of the earth dam stability issue, it seems that the earth dam stability is very important. Since, one of the most important issues affecting the analysis and design of the earth dam is to investigate their stability; it is essential to perform an appropriate research on the factors affecting the earth dam stability in different static and dynamic scenarios. To achieve accurate analyses and designs as well as to apply an appropriate safety margin, the effect of the seismic destructive forces must be properly taken into account within analyses and designs. For this purpose, the seismic analyses must be done for investigating the earth dam stability (Shelley and Santoyo, 2001; Ambraseys, 1990; Gazetas, 1987).

In the dams constructed in narrow valleys, the behavior of earth dams is relatively more complicated compared to elevated dams. their behavior depends on the dimensions, slope and location of the clay core (Seed, 1973; Seed, 1979; Chhatre and Muralidhar, 2010; Feng et al., 2010). In such dams, since the dam initial stability didn’t supply, in the case of the earthquake occurrence, it is essential to reinforce the dam body in many ways such as using the geogrid (L´opez-Querol and Moreta; 2008; Koerner, 2000; Ozkan, 2008; Jewell, 2004). Considering above mentioned issues, in this study, it is attempted to evaluate the Alavian earth dam, as a case study, its dynamic stability and the effect of geogrids on the Alavian dam reinforcement.

The aim of the static and dynamic analyses of the Alavian dam, in various states in terms of applying the geogrid and without geogrid to reinforce the dam, is to investigate its dynamic behavior and response as well as to obtain optimal reinforcement position with geogrids in terms of length, number, distance and angle of the geogrid, based on the results of above analyses. On this basis, the water level in the reservoir is considered as stable. Likewise, below and above the phreatic water level (below the highest water level line) is assumed as saturated and dry. Besides, it should also be noted that to perform the analyses the finite element method (FEM) in the Plaxis and GeoOffice software are used. Accordingly, the objectives of this study can be outlined as follow:

- Evaluation of the Alavian dam static and dynamic stability
- Investigation of the earth dam behavior in response to the change of the location, size, length and number of the geogrids to reinforce the Alavian dam body and finally determining the optimal slope of the dam, based on the static and dynamic analyses

A case-history of analysis of the dynamic stability and reinforcement of the earth dams

The first step in dynamic modeling of the dams was taken in 1929 by Mononobe (Mononobe and Matsuo, 1929). Probably he was the first person who considered the earth dam body as a ductile substance. besides, he established a model for the earth dam, which is known as the share wedge or share beam model. But 20 years were passed before the time that Ambraseys and Hatanaka developed this model and introduced it as the analysis engineering method (Hatanaka, 1995; Ambraseys, 1990). By neglecting the rotational and bending deformations against the shear deformation in the shear beam model, they obtained a two-dimensional response from a dam existing in the rectangular valley. Therefore, the shear beam method is based on the note that an earth dam would be changed as a simple shear deformation, thereby would cause only horizontal displacement. In this study, it is also assumed that both of the shear stresses and shear strains are applied on the uniform horizontal plates. Accordingly, the stresses and strains are constant through the dam and they will reach zero only in small areas near the upstream and downstream surface (Hatanaka, 1995). Terzaghi (Terzaghi, 1990) stated that the concept of the earthquake effects on the Dam Slopes discussed in this method is very approximate. He also stated that the smallest characteristic of this method is that it is very imprecise for explaining the effects of the earthquake on the dam Slopes. Besides, despite the calculated static safety factor is greater than 1, dam Slope is unstable. For example, in the Sheffield dams, upper and lower Senfeznando dams, the quasi-static analyses show safety factors greater than 1, but in the earthquake events, they are disrupted (Ambraseys, 1990). These indicate the inability of the quasi-static method in reliable assessment of stability of the Dam Slopes, which are prone to loosening instabilities. Besides, another major shortcoming of this approach is that no idea is stated concerning the note that whether or not the plastic deformations under seismic loading are expected (Terzaghi, 1990).

Martin and Seed (Gazetas, 1987) criticized the concept of seismic coefficient due to its difficulty in selecting an appropriate value for the seismic coefficient. They stated that if the coefficient multiplying by potential mass weight is assumed as the indicative of maximum inertia force created on the mass, in most cases resulted in conservative calculation of the static force. If it is asked to choose the coefficient in such a way, that static force which in terms of effects is equivalent with real dynamic inertia forces, to be allocated, then it is not clear how the equivalent static force must be determined. As Martin and Seed stated, if, for example, the safety factor designer, equal to 1.1 is considered under the seismic conditions, then by applying the seismic coefficient of less than 1.0 there is no need to correct the embankment section to the section that meets the static stability requirements. This is not consistent with the actual conditions; thus the designer must consider a greater seismic coefficient, for instance0.15.

Sarma and Barbosa presented charts to estimate the critical acceleration of the rockfill dams with clay core using the principles of limit equilibrium and assumption of dichotomous wedge failure plate and undrained conditions for the clay core (Seed, 1979). Kikusava implemented a quasi-static analysis on a dam with impermeable central wall using the rigid-plastic finite element model. Kikusava presented a chart to show the critical slope and critical seismic coefficient for given stability number (Chhatre, 2010). Besides, in another study, the seismic response of the Liyutan earth dam in Taiwan is analyzed and evaluated by Feng under the effect of the ChiChi earthquake (with a magnitude of 7.3), by considering the upstream water level equal to 60 m. after it was found that the dam is unstable against earthquake, Feng studied the dam in order to reinforce it (Feng, 2010). In another study, Queroland and Moreta by dynamic analysis of the earth dam against earthquake, investigated the impact of the clay core position as well as the reinforcement on of such dams and its effect on seismic stability (Queroland and Moreta, 2008).

The site and technical geological characteristics of the study dam (Alavian dam)

The multi-purpose SoofiChay projects (which one of its projects is the Alavian dam) are implemented in northwestern Iran, south hillsides of the Sahand Montains and southeast of Ormieh Lake. The project regionin cludes the Bonab and Maragheh cities or the catchment area of the SoofiChai and Mardagh Chai rivers. It is located at 120 km southwest of the Tabriz city and in the coordinates of 46° - 46° 25´ E and 37° 11´ - 37° 28´ N. The Alavian dam, which is constructed on the Soofi hai River, is located at 3.5 km upstream of the Maragheh city and near the Alavian village. The objectives of construction of the Alavian dam are to reserve and control of the SoofiChai surface currents to supply the drinking water of Maragheh and military presidio and to compensate part of shortages of the irrigation and agricultural requirements in the Maragheh plains and its surrounding gardens as well as to produce the energy/power. The desired dam is an earth dam with the characteristics according to the following table. The right hillside slope is mild and the alluvial terraces represent the stair- like perspective. Likewise, in the left hillside, 15- 20 m of the river bed represents a sharp slope and above it represents a mild slope. In the Alavian dam the alluvial terraces are situated in three horizons including: the upper terrace (T1) with 20 m thickness and mainly composed of coarse grained materials; the middle terrace (T2) with 31.5 and 25.5 m thickness, respectively in the right and left hillsides and a clay layer in the upper part of the left hillside with a maximum thickness of 12- 13.5 m; and the lower terrace(T3) in the right hillside with 8-12 m thickness is composed of coarse- grained materials together with fine-grained materials.

Table 1: the technical characteristics of the Alavian dam

characteristics of the gravel and sandstone shell

Row	Description	Allowable values
1	The loss percent (Los Angeles)	34%
2	The water absorption percent	<2/4
3	The material loss percent	<4

characteristics of the dam body

Row	Description	Allowable values
1	The loss percent (Los Angeles)	24%
2	The actual specific gravity	<2/6
3	The water absorption percent	<2
4	The material weight loss percent	<2
5	The uniaxial saturated resistance (kg/cm^2)	1000
6	the uniaxial dry resistance (kg/cm^2)	1200

Technical characteristics of the drainage materials

Row	Description	Allowable values
1	The loss percent (Los Angeles)	30%
2	The material weight loss percent	2%
3	Minimum GS of the material	<2/6
4	D14	<13

characteristics of the dam body

Type of the dam	Earth, rockfill with clay core
The reservoir normal values	1568 m from sea level
The dam crest values	1572 m from sea level
The river bed values	1558 m from sea level
The dam foundation values	14 m^2 from sea level
The dam height from the river floor	70 m
The dam height from the foundation	80 m
The crest length	935 m
The crest width	10 m
The reservoir volume in the normal values	160*106 m^3
The volume of 40 years sediment	7/4 *106 m^3
The volume of adjusted water	73*106 m^3
The length of the gallery below the body	340 m

Technical characteristics of the fin- grained and coarse- grained filter

Row	Description	Allowable values
1	The loss percent (Los Angeles)	30%
2	The material weight loss percent	2%
3	Minimum GS of the material	>2/6
4	Percent of passed –material- through sieve 200	<4
5	Plasticity index (PI)	0

Besides, the river bed with a thickness of 5 to 10 m, which is extended in the right side, is composed of the coarse-grained materials including volcanic rubbles, gravel and sand. The Alavian dam foundation is composed of three types of tuff. They are: (1) a relatively dense white tuff with low porosity, with a thickness exceeding 50 meters in the left hillside and a thickness 15-30 m in the right hills ideas well as below the river bed; (2) a gray tuff that generally is very porous and weathered with a thickness of 10-15 min the right hill; and (3) a pink tuff with a maximum thickness of 15 m and heavily weathered, brittle and with low mechanical strength in the right hillside.

Besides, a sedimentary stratum composed of a variety of sandy clay, sandstone and brittle conglomerate is recognized as a relatively homogeneous and continuous horizon in the depths of 35 to 70 m in the river bed, the right coast of the dam abutment. Based on the implemented investigations in the Alavian dam location there is three main faults and a numbers of the minor faults with more or less the north-south direction, which formed three separate large blocks.

Based on the results of the permeability tests by the Lauferan method the permeability value in the fluvial alluvium of the study site, the dense alluvium of the T2 and T3 terrace sand in the silt clay sediments in the left abutment is estimated equal to 10-2 – 10-3, 10-3 – 10-4 and 10-6 – 10-8 m/s, respectively. In tables (1) and (2) the technical characteristics of different parts and the dam design parameters of the study dam are presented, respectively. Profile of the study dam is shown in the following figure.

Table 2: parameters of different layers used in the dam analysis

ID	Name	Type	γ_{unsat} $[kN/m^3]$	γ_{sat} $[kN/m^3]$	k_x $[m/day]$	k_y $[m/day]$	ν $[-]$	E_{ref} $[kN/m^3]$	c_{ref} $[kN/m^2]$	ϕ [deg]
1	Clay Core	Undrained	18	20.5	4.32E-5	4.32E-5	0.29	20000	60	35
2	Silty Sand	Dreained	18.5	20	0.0432	0.0432	0.25	90000	1	35
3	Inverse Filter	Dreained	18	20	1.728E-5	1.728E-5	0.27	40000	1	30
5	Transition Zone	Dreained	18.5	20.5	9.5040	9.5040	0.27	50000	1	35
7	Alluvial	Dreained	20	21.5	2.57E-5	2.57E-5	0.28	30000	1	35
8	Tuff- 1	Dreained	21	22	0.0864	0.0864	0.37	12000	600	22
9	Tuff- 2	Dreained	20	21	0.0864	0.0864	0.36	10000	500	22
10	Tuff- 3	Dreained	19	20.5	0.0864	0.0864	0.35	8000	300	22
ID	Name	Type	γ_{unsat}	γ_{sat}	k_x	k_y	ν	E_{ref}	y_{ref}	R_{inter}
4	Cut of Wall	Non-Porous	20	20	0	0	0.17	2E8	0	1
6	Riprap	Dreained	18	20	0.0346	0.0346	0.38	01.9262E7	0	1

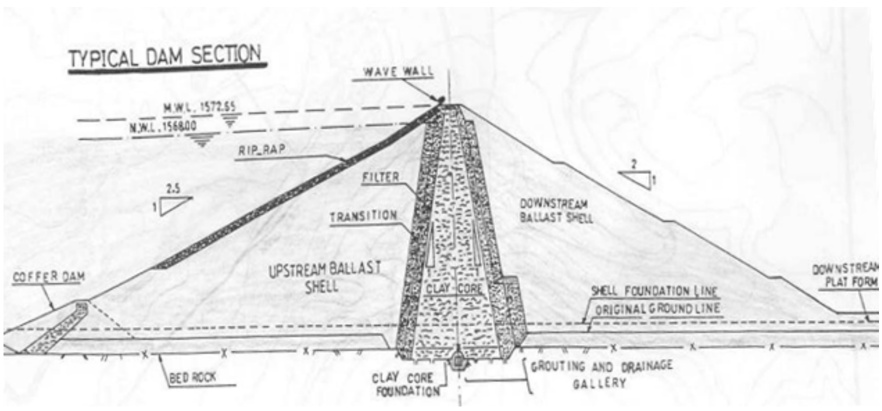


Figure 1: Cross section of the study earth dam

Modeling and result analysis

In order to analyze the study dam, at first using the finite element software of GeoOffice the dam was modeled. Besides, since the dynamic analysis of the earth dams is among the complex problems in geotechnical engineering, to obtain the precise and acceptable answers from the dam behavior it is necessary that the dam behavior during the earthquake to be understood properly, then based on the available software capabilities the modeling is conducted properly. The dynamic analysis of the study dam is performed according to the following steps.

1- Analysis of the leakage, pore-water pressure and the total head of the water behind the dam before applying the earthquake

2-Analysis of the stress existing in the dam before applying the earthquake

3- Analysis of the dam stability for upstream and downstream before applying the earthquake

4- Applying the earthquake and analysis of the changes occurred under the stresses and pore-water pressure

5- Analysis of the dam stability for upstream and downstream during and after applying the earthquake

6-Determiningthe displacements after applying the earthquake

7- Determining the critical slope behind the dam in both of the static and dynamic statuses

8-Investigationof the reinforcement effect of the critical slope behind the dam using the geogrid

Figures 2 and 3 show the profile of the Alavian dam, which is modeled in software of Plaxis and GeoOffice. It should be noted that one of the main problems in finite element analysis is the definition of the type of elements used in the modeling. In this study, the combination of triangular 6-node and quadrangle 8-node elements is used. To perform the analyses, at first the leakage and stability analysis as well as the stress, strain and displacement analysis were performed at different statuses including: at the end of construction (when the dam is constructed, but impoundment is not implemented yet);steady state (when the dam is constructed and impoundment is implemented to height of 80 m, so that it reached to the steady state);and the rapid draw down (the rapid draw down would be occurred during one year after watering) in two positions, namely in static conditions and in association with the earthquake by a quasi-static method. However, due to the limited volume of the paper, the different statuses of the dam in the static state have not been described and only the dynamic conditions of the dam is analyzed. The results of the dam stability behavior analysis under static and quasi-static conditions are presented in table 3.

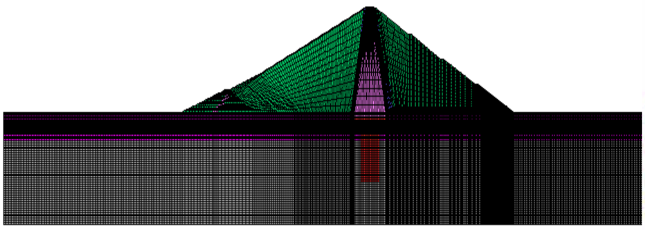


Figure 2: modeling of the Alavian cross section in the Geo-Studio software

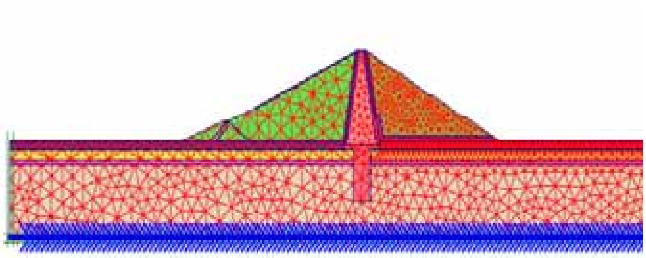


Figure 3: modeling of the Alavian cross section in the Plaxis software

Table 3: analysis of the dam stability behavior before applying the earthquake under the static conditions

Construction or utilization status	Safety factor			
	The downstream slope	The upstream slope	The downstream slope	The upstream slope
	In the quasi- static status		In the static status	
End of Construction	2.404	1.967	1.336	1.468
Steady State	2.208	1.805	1.266	1.449
Rapid draw down	2.170	1.852	-	1.365

Based on the results obtained from above table and their comparison with minimum stability safety factor of the dam in different statuses according to the American regulation, it can be seen that in all above statuses the dam stability is in a desirable condition.

The dynamic analysis of the dam behavior after the earthquake occurrence

In the earth dams, which the reciprocating motion of the earthquake results in the loss of the soil cohesion and consequently results in their more and more instability, the use of the nonlinear dynamic analyses is the best option for seismic analysis. In this section to perform the seismic analyses for the study earth dam the accelerograph of the SanFernando earthquake with a maximum duration of 14 seconds and a magnitude of 6. 6 Richter (on the Richter scale) and horizontal maximum acceleration (PGA = 0. 6g) at -the time of- 8. 22 seconds after the beginning of the earthquakes is used.

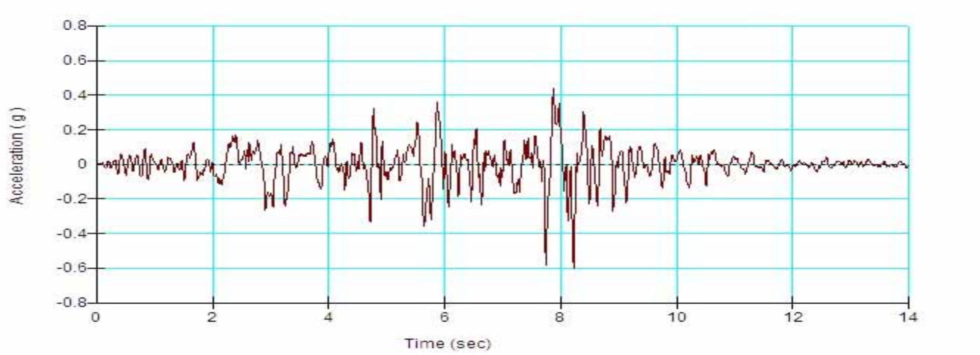
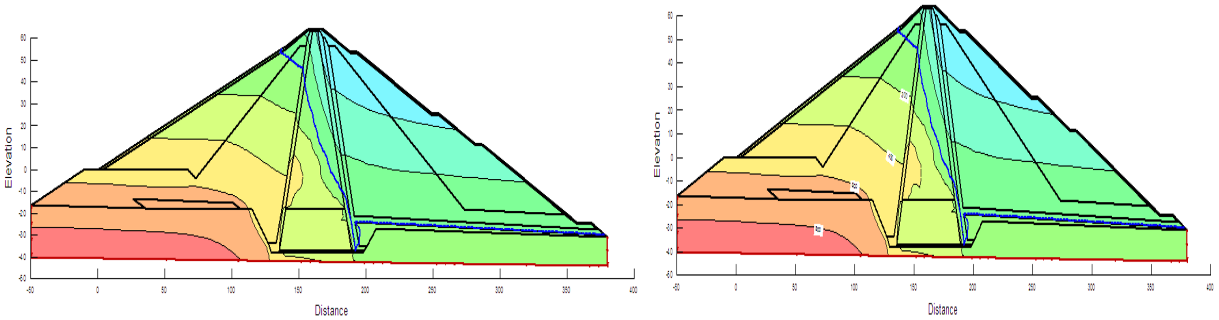


Figure 5: The accelerograph applied in the analyses

In this chapter the status of the pore-water pressure, total head, and existing stresses after applying the earthquake for second 8. 22 (peak of the earthquake) and second 14 (end of the earthquake) is investigated. Accordingly, the status of the study dam at the beginning, at the moment of maximum acceleration occurrence and at the end of the earthquake can be compared.

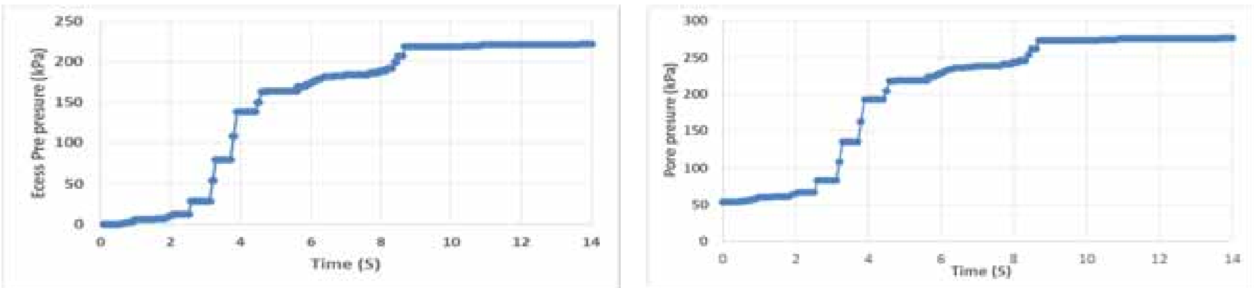
Analysis of the seepage in the dam due to the earthquake

Figure 6 shows the distribution of pore-water pressure in the moment of maximum acceleration occurrence and at the end of the earthquake. By comparing these figures with the distribution of the pore-water pressure before the earthquake occurrence the enhancement mode of the pore-water pressure within the dam body is easily observable. Since one of the most important issues in the analysis of earth dam with clay core is the moment and great increasing of the water pressure in the core during the earthquake, in the following charts the pore-water pressure changes during the earthquake has been drown based on the time. As shown in figure 7-a, at the end of the earthquake the amount of the pore-water pressure in center of the clay core is increased by 220 kPa. Accordingly, the general chart of the pore-water pressure distribution in the center of the clay core can be illustrated in the form of figure 7-a.



a b

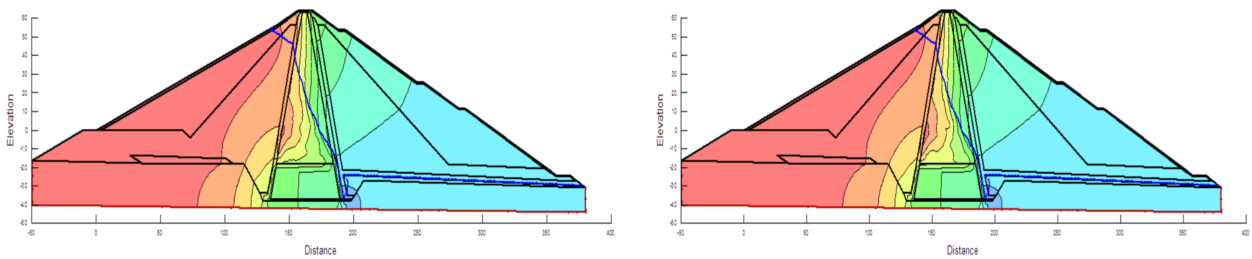
Figure 6: the pore-water pressure distribution in the dynamic analysis: (a) in the moment of the earthquake peak (occurrence of maximum acceleration); (b) at the final moment of the earthquake



a b

Figure 7: the pore-water pressure changes over the time: (a) during the earthquake; (b) in the center of the clay core

Besides, figure 8 represents the changes of the total head of water and its distribution in the dam body at the earthquake peak and final moments. By applying the earthquake the value of the pore-water pressure and total head in the fine grained parts of the dam body is significantly increased. This resulted firstly in decreasing the effective stress and consequently resulted in decreasing the soil share resistance and rapid increasing of the pore-water pressure is observable in these figures clearly.



a b

Figure 8: changes of the total head in the dynamic analysis: (a) at the earthquake peak moment; (b) at the final moment of the earthquake

Evaluation of the stress statuses caused by the earthquake

Figure 9 shows the total stress distribution at the peak moment and at the final moment of the earthquake. By occurring the earthquake, the value of the pore-water pressure is increased. It is resulted in significant increasing of the total stress which is clearly shown on the figure. It should be noted that this increase in the stress was higher than other parts in the clay core. Accordingly, the effective stress distribution in the study dam is shown in figure 10.

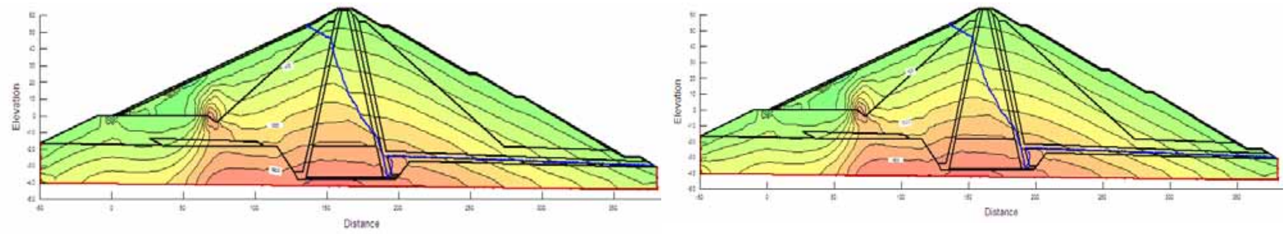


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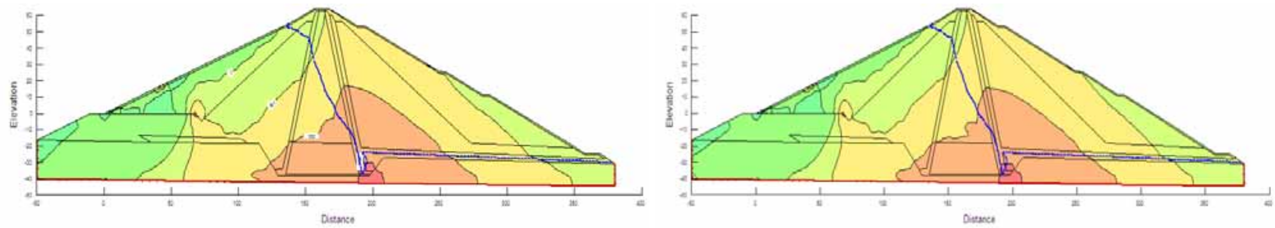
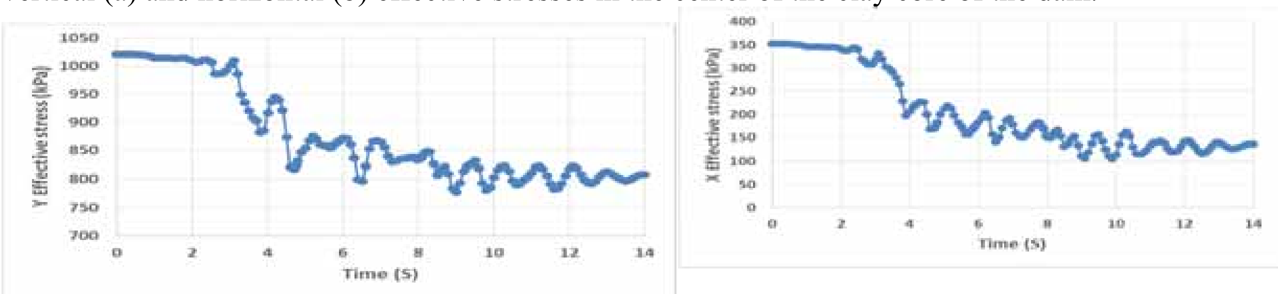
Figure 9: The total stress distribution in the dam body: (a) at the earthquake peak moment; (b) at the final moment of the earthquake



a b

Figure 10: The total stress distribution in the dam body: a) at the earthquake peak moment; b) at the final moment of the earthquake

One of the most important parameters that affect the design of the dams is the effective stress values existing in the dam different parts. The existing effective stresses depend highly on the pore-water pressure. Besides, since by occurrence of the earthquake the pore- pressure value is increased somewhat, thus the effective stress value will be reduced. Figure 11 shows the changes of the vertical (a) and horizontal (b) effective stresses in the center of the clay core of the dam.



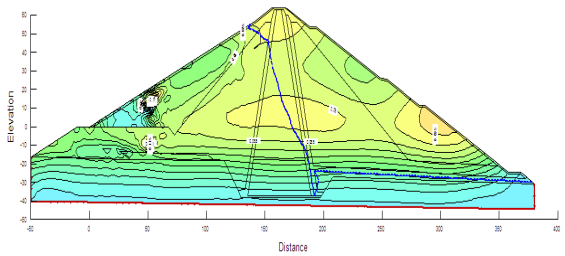
a b

Figure 11: The changes of the effective stress in the center of the dam clay core: (a) the vertical effective stress; (b) the horizontal effective stress

Analysis of the displacements caused by the earthquake

After the earthquake occurrence, one of the most important issues that is widely considered in the dam design is to investigate the value of the displacement occurred in the dam. Figures 12 and

13 show the horizontal and vertical displacements –occurred-in different parts of the dam at the peak and final moments of the earthquake.



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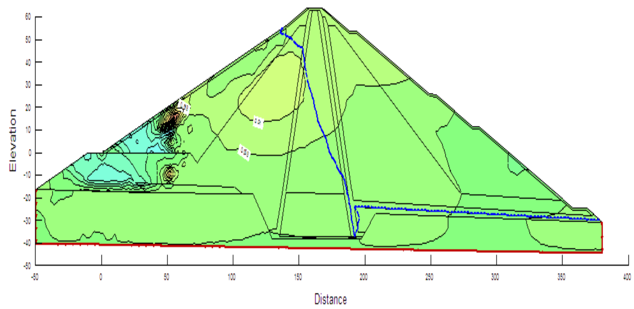
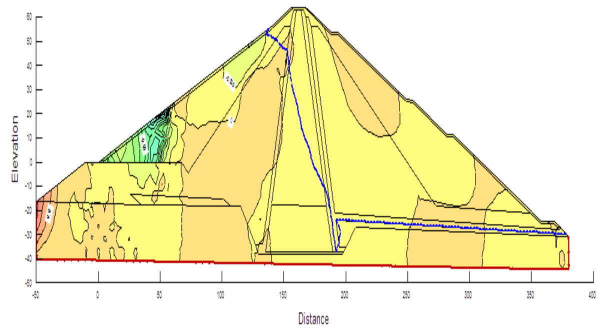


b

Figure 12: The dam horizontal displacements caused by earthquake: (a) at the earthquake peak moment; (b) at the final moment of the earthquake



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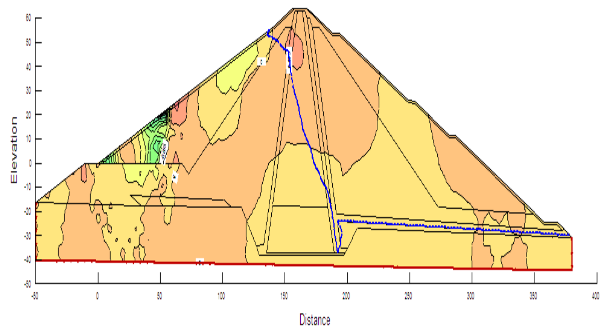

b

Figure 13: The dam vertical displacements caused by earthquake: (a) at the earthquake peak moment; (b) at the final moment of the earthquake

Based on the above figures it can be seen that maximum horizontal displacement is occurred at the peak moment of the earthquake in the dam crest and clay core. In parts of the dam the concentration of the occurred displacements is high. These areas are places prone to starting the failure. One of the most influential factors in designing and dynamic stability analysis of the earth dam is to investigate the vertical and horizontal displacements in the dam crest and mid part. For this purpose, to proper understand the seismic behavior of the study earth dam in the following figures the horizontal displacement values created in the middle of the clay core and the dam crest are shown.

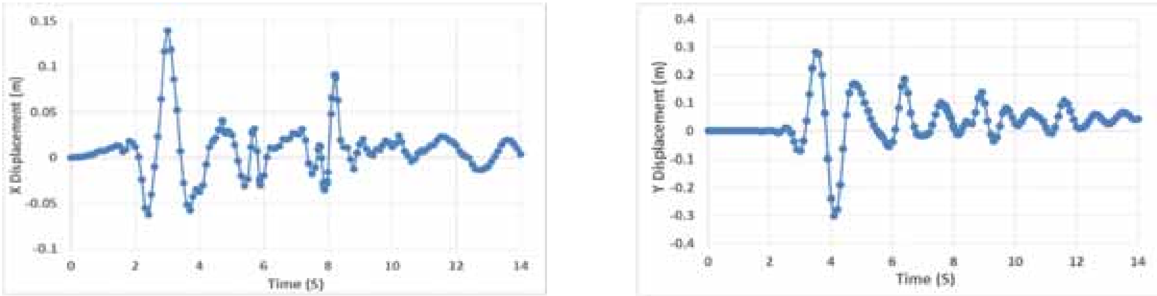

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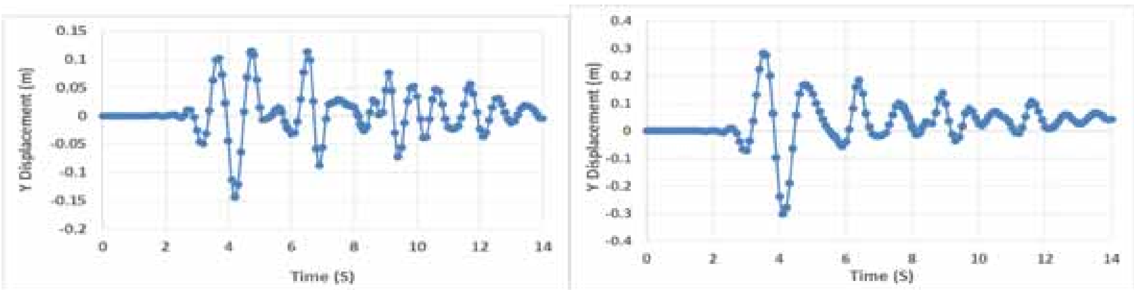
Figure 14: the horizontal displacements created by earthquake: (a) in the mid part of the clay core; (b) in the dam crest

The vertical displacements created in the dam after the earthquake are consistent with the after earthquake shear stress distribution.



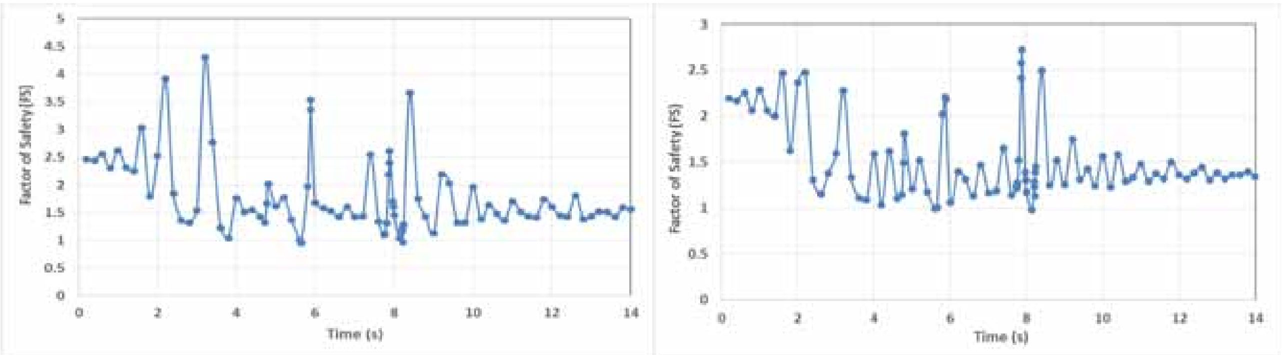
a b

Figure 15: The vertical displacements created by earthquake: (a) in the mid part of the clay core; (b) in the dam crest

This properly shows the effect of reducing the shear strength on increasing the displacements. In the following figures the vertical displacements created in the middle of the clay core and the dam crest are shown.

The dam dynamic stability against earthquake

In the dam dynamic analysis, the dam stability safety factor is changed according to the earthquake acceleration. It can cause to creation instability in the earth dams. For this purpose and to investigate the effect of the earthquake different accelerations the safety factor will be provided for different times. The safety factor proposed by valid authorities for dynamic analysis of the earth dam upstream is equal to 1.3; based on the performed dynamic analyses and the results presented in the figure below the safety factor of the dam upstream, except for very short times, is lower than this value. Likewise, the safety factor is mainly more than 1.5. In figure 16 the safety factor changes for the upstream and downstream stability of the study dam is presented in time. As can be seen in figure 16, the dynamic stability safety factor value of the study dam is decreased rapidly at first; after passing a short time, these changes are relatively reduced; by reaching to the maximum earthquake acceleration the lowest value of the safety factor is obtained. In above figure the dynamic stability safety factor value of the study dam is lower than the allowable limit that this can effective in the final designs; but the dam stability safety factor would be considered by applying all coefficients during the earthquake. Based on the valid references, the safety factor value of the dam downstream during the earthquake should at least equal to 1.3 -1.5.Based on above figure this condition can be considered as unsatisfied and it can be concluded that the stability of the study dam is not satisfied in the dynamic conditions.

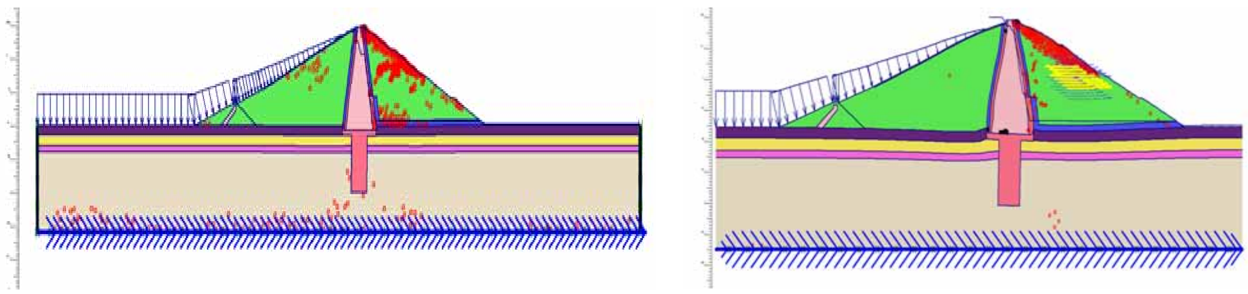


a b

Figure 16: the dynamic stability safety factor changes in the study dam based on the time: (a) in the dam upstream; (b) in the dam downstream

Reinforcement of the critical slope of the slope behind the dam

As it is concluded in previous chapter, the stability of the Alavian dam under seismic loading is not met. Therefore, in this chapter it is attempted that after determining the critical slope behind the dam, in both statuses of with and without geogrid, the use of the geogrid to increase the dam stability to be studied. Based on the performed analyses (Figure 17) it was found that in the study dam, in both statuses of with and without geogrid the dam downstream is affected by sliding. Besides, as critical slope behind the dam considered as the base of reinforcement using geogrids.



a b

Figure 17: recognizing the critical slope behind the dam: (a) without geogrid; (b) with geogrid

In order to reinforce the critical slope (downstream) of the study dam, several patterns of geogrids are used. Likewise, to increase the stability safety factor, given the changes of the length, number and distance between the geogrids as well as by assuming that the downstream slope is variable, 150 analyses are performed in both of the static and dynamic statuses. The study patterns are outlined in table 4. Besides, the stability safety factors resulted from the PLAXIS program are illustrated in tables 5 and 6.

Table 4: Different patterns to reinforce the downstream critical slope of the study dam using the geogrid

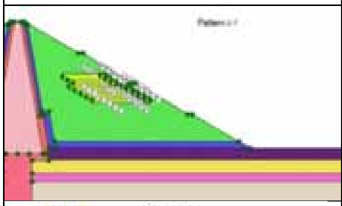
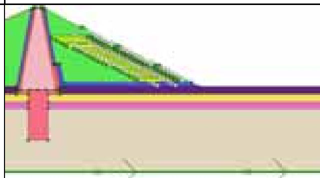
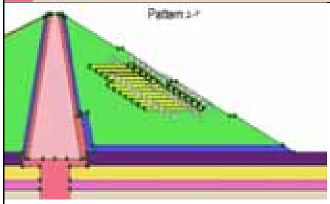
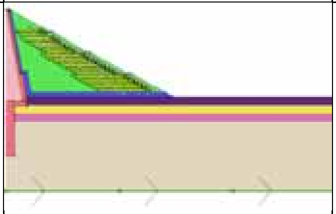
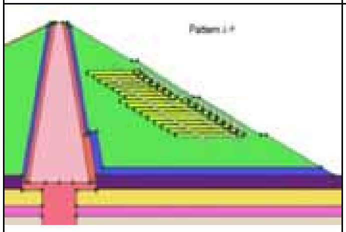
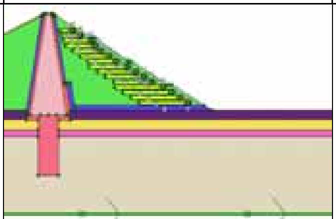
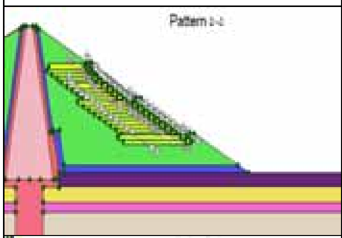
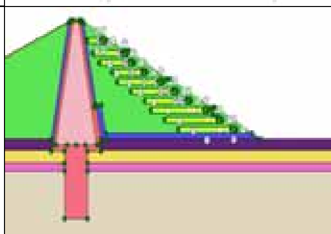
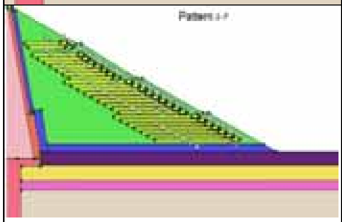
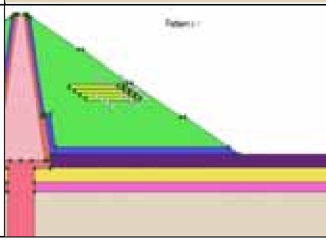
Figure	Modeling mode	Pattern number	Figure	Modeling mode	Pattern number
	Pattern no. 2 and with 4 geogrids in the middle	5-2		21 geogrids with the constant length of 40 m and constant distances of 2 m	1
	Pattern no. 2 and with 8 geogrids in the middle	5-3		25 geogrids with the variable length of 50, 40, 30 and 20 m and constant distances of 2 m	2
	Pattern no. 2 and with 16 geogrids in the middle	5-4		15 geogrids with the variable length of 50, 40, 30 and 20 m and constant distances of 4 m	3
	Pattern no. 2 and with 20 geogrids in the middle	5-5		10 geogrids with the variable length of 50, 40, 30 and 20 m and constant distances of 6 m	4
	Pattern no. 2 and with 24 geogrids in the middle	5-6		10 geogrids with the variable length of 50, 40, 30 and 20 m and constant distances of 6 m	55-1

Table 5: The stability safety factor of the Alavian dam with regards the changes of the length, number and distance between the geogrids in different statuses

Analysis and Geogrid Type	Pattern-1	Pattern-2	Pattern-3	Pattern-4
Statically without geogrids	1.7387	1.7387	1.7387	1.7387
Statically with strong geogrid (CE 151)	1.9968	2.2524	1.9092	1.8256
Statically with medium geogrid (CE 131)	1.9532	2.1315	1.8579	1.7788
Statically with weak geogrid (CE 121)	1.839	2.0889	1.794	1.7513
Dynamically without geogrid	1.0795	1.0795	1.0795	1.0795
Dynamically with strong geogrid (CE 151)	1.5867	1.6435	1.515	1.4655
Dynamically with medium geogrid (CE 151)	1.4645	1.6108	1.4103	1.3479
Dynamically with weak geogrid (CE 121)	1.2641	1.318	1.214	1.1986

Table 6: the stability safety factor of the Alavian dam with regards the changes of the geogrid number in pattern 5 of the different statuses

Strong Type of geogrid (CE 151)	Pattern-5	
number of geogrid	Statical	Dynamical
4 geogrid	1.7395	1.1722
8 geogrid	1.7404	1.2378
12 geogrid	1.7981	1.2595
16 geogrid	1.8114	1.3754
20 geogrid	1.9625	1.521
24 geogrid	1.9832	1.5712
25 geogrid	2.2524	1.6432

Investigation of the effect of the parameters affecting the stability safety factor of critical slope behind the dam

At the next part of this paper, different parameters have been analyzed by program–used-, the results are resulted in the charts and are shown in the next figures. Each chart is indicative of effects of one parameter in the analyses.

The effect of the distance between the geogrids in the static and dynamic statuses

In figure 18 can be seen that by increasing the distance between the geogrids in the critical slope, the safety factor is reduced. It is worth mentioning that considering 3 types of the geogrids, as it is expected they are arranged from the strong geogrid to the weak ones.

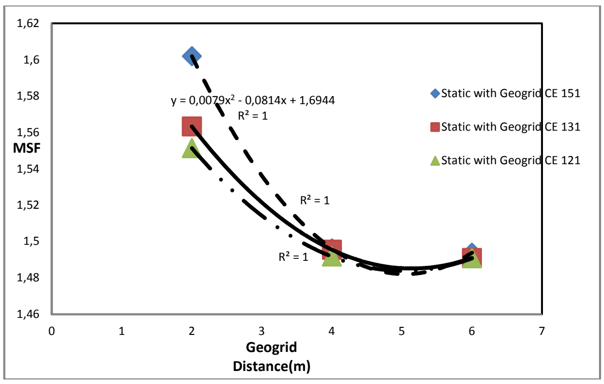


Figure 18: The effect of the distance between the geogrids in the static status

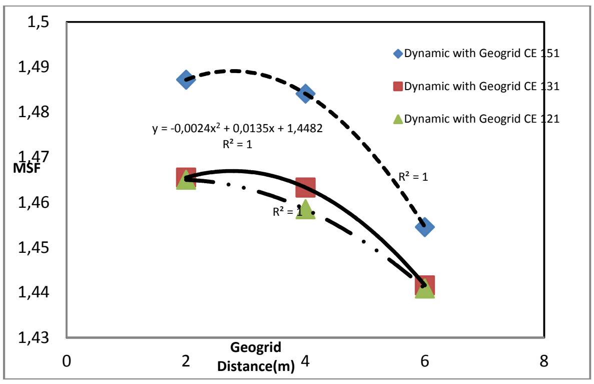


Figure 19: The effect of the distance between the geogrids in the dynamic status

Besides, figure 19 shows that by increasing the distance between the geogrid the safety factor is reduced. It is worth mentioning that considering greater number of variables as table trend as well as the effect of 3 types of the geogrids can be seen in the chart.

The stiffness effect of the geogrids in the static and dynamic statuses

In figure 20 can be seen that by increasing the stiffness of the geodrides the safety factor is increased as well as by changing the pattern the results are obtained. Likewise, as it is expected the pattern 2 is provided the better results compared to other patterns. Besides, as shown in figure 21, by increasing the stiffness of the geodrides the safety factor is increased as well as the effect of the geogrids in dynamic status is greater than the static status.

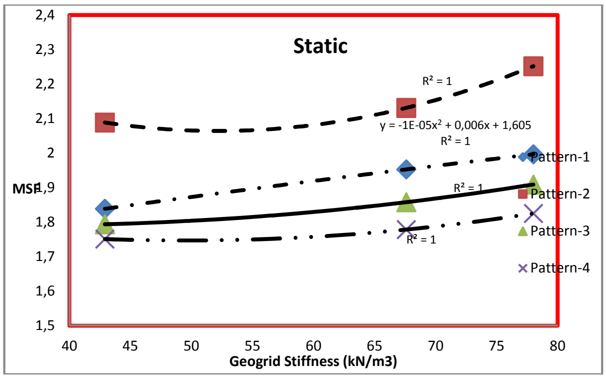


Figure 20: The effect of the stiffness of the geodrides in the static status

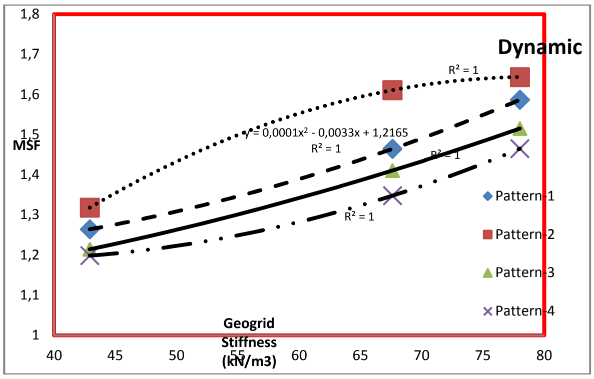


Figure 21: The effect of the stiffness of the geogrides in the dynamic status

The effect of the number of geogrids in pattern 5 in thestatic and dynamic statuses

In this pattern the number of geogrids is variable and as it is expected by increasing -the number of- the geogrides the safety factor is increased as well as by comparing the chart(s) in figure 22 can be concluded that the effect of increasing -the number of- the geogrides in dynamic status is clearer and greater than static status. Besides, the obtained results show that the effect of the geogrides in the dynamic status is clearer than the static status. Besides, the importance of the geogrid in the dynamic earthquake is clearly understandable.

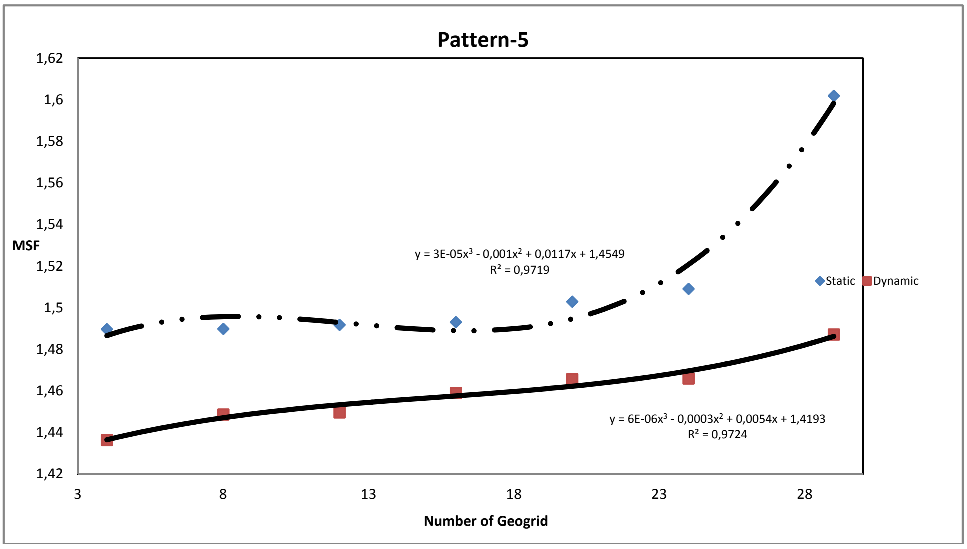


Figure 22: The effect of the number of geogrid in pattern 5 in the static and dynamic statuses

The slope angle change(s) in the static and dynamic statuses

In the next –part- by changing the slope in 6 different statuses the effect of the slope angle in the downstream of the study dam is determined. In table 7, the safety factors obtained from changing the downstream slope angle in relation to horizon in the static and dynamic statuses has been presented with and without geogrid.

Table 7: Results of the slope angle changing

Downstream slope angle (degrees)	Statical		Dynamical	Dynamical
	Without geogrid	With Geogrid	Without geogrid	With Geogrid
26.56	1.7378	2.2524	1.0795	1.6435
30.963	1.662	1.8428	0.9374	1.5338
34.992	1.5727	1.5953	0.9049	1.3473
38.659	1.3028	1.4446	0.8764	1.1038
41.987	0.9787	1.1417	0.7707	0.9743
45	0.7549	1.0311	0.5779	0.7113

As shown in figure 23, by increasing the slope angle in the static status with and without geogrid the safety factor is reduced, however, it is increased with geogrid, but after a –certain- angle it is not effective.

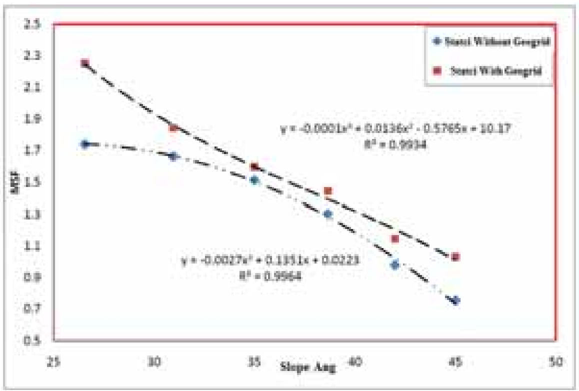


Figure 23: increasing the slope angle in the static status with and without geogrid

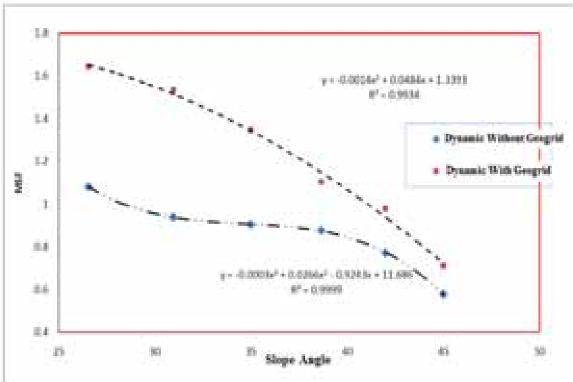


Figure 24: increasing the slope angle in the dynamic status with and without geogrid

Besides, it is shown in figure 24 that by increasing the slope angle in the dynamic status the safety factor is reduced and generally with the geogrid the safety factor is increased. However, by increasing the angle, after a –certain- angle, the slope changing has no significant effect and results in decreasing the safety factor. However, by comparing the graphs 23 and 24 can be found that the use of geogrid in the same slope angles is resulted in improving the safety factor compared to without geogrid status. Besides, this increase is particularly clearer in the dynamic status.

Conclusion

In this paper, using numerical calculations and numerical analyses in the finite element software of PLAXIS and limit equilibrium of Geo Office the stability safety factor of the earth dam was studied by doing a case study on the Alavian dam, Iran. In calculations the finite element method was used together with the dynamic earthquake. Besides, in the limit equilibrium calculations the quasi-static method was used. The main results obtained from this study can be summarized as follows:

• In the study dam when the dam is filled by water and the steady-state current is occurred in the dam, the safety factor for the dam downstream is smaller than for upstream.Since, the effective stresses in the dam downstream are greater than the dam upstream, it is expected that the safety factor be greater for downstream. In the dam upstream the water pressure resulted from the water level behind the dam can prevent the displacement and results in higher stability of the dam, thus it main creases the stability safety factor of the upstream.

• By applying the earthquake and therefore the resulting vibrations generated in the dam, the pore-water pressure would be increased. This increase needs adequate time to drain. But since the permeability coefficient of the soil layer is very inconsiderable, thus this pressure increasing will be permanent. Likewise, by applying the earthquake consecutive cycles this pressure increasing will increase cumulatively, which in turn causes to decreasing the effective stress and consequently results in decreasing the dam stability safety factor.

• The permeability of the dam clay core is lower than other layers existing in the dam, therefore, by applying the earthquake the increase of the pressure in clay core is more than other parts. Accordingly, to stabilize the earth dam a specific attention should be paid to the clay core.

• Due to the effect of the earthquake force and sudden changes in pore-water pressure, those points of the dam, which were in contact with the water, the effective stress is highly decreased in vertical and horizontal directions. Thus, in the upstream points of the shell the effective stress is severely decreased and in some places even reached to zero.

• In the dynamic analyses the dam stability safety factor is changed in terms of acceleration of the applied earthquake, which in turn can cause to create instability in the earth dams. The stability safety factor value is decreased rapidly at first; after passing a short time the changes are reduced and by reaching to the maximum earthquake acceleration damage the lowest safety factor is achieved. This shows that the conditions of dynamic stability of the study dam are not met.

The results obtained from reinforcement of the critical slope (downstream) behind the study dam can be expressed as follows:

• By increasing the stiffness of the geogrids as well as by reducing the distance between them the stability safety factor of the reinforced earth dam is increased compared to unreinforced status.

• By increasing the number and length of the geogrids the stability safety factor is increased

• By increasing the slope angle, whether in the static status or in the dynamic status, the safety factor is reduced.

• The use of geogrid in the same slope angles resulted in improving the safety factor compared to without geogrid status. This increase is particularly clearer in the dynamic status.

• By moving from the dam crest (top) towards the dam foundation (down)the displacement is reduced.

• The displacement is increased with increasing the geogrid distances compared to the height.

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