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Stability assessment of the Farrokhi earth embankment dam using the pseudo-static and deformation based methods

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Abstract

The behavior of earth dams has particular complexities against dynamic forces and its assessment requires detailed and scientific analysis. The Farrokhi earth dam is a heterogeneous type with vertical clay core, built 150 km away from Birjand and 41 km away from Qaen on the Farrokhi River. There are concerns regarding the stability of this dam in the event of a future severe earthquake especially because of its placement near the active faults of eastern Iran, a very active seismic area in the country. In this study, the pseudo-static analysis, sliding block model and Singh et al empirical relationships were adopted to evaluate safety of the Farrokhi dam against the design earthquake. Based on the findings of the present study, the stability of the Farrokhi earth dam is not guaranteed and as such, small to large size slope failures are expected especially in the upstream side. The pseudo-static analysis and the Singh et al empirical relationships revealed similar conclusions about the stability condition of this dam, while the sliding block method underestimated the earthquake related deformations.

Keywords: *Farrokhi earth dam, probabilistic pseudo-static analysis, deformation based methods, Monte Carlo simulation.*

1. Introduction

The earth dam is an artificial dense embankment and is built to store water for various agricultural and industrial purposes or for drinking. One of the destructive factors that cause failure in the body of earth dams is the vibration caused by the earthquake in surrounding area of the dam site. Although most engineers consider earth dams safer than concrete dams, there are documented reports [1-2] of the collapse of earth dams due to earthquake vibrations. Based on the aforementioned, the minor to total destruction of seven earth dams in the Indian state of Gujarat could be mentioned, which occurred as a result of the famous 7.6-magnitude Bhuj earthquake on the 26th of January 2001[1]. Therefore, due to the large volume of water behind the dam embankment and the cost of construction, stability analysis of earth dams against destructive factors, including the vibration of a possible earthquake is essential. This is especially true in the case of the Farrokhi earth dam, because of its placement near the active faults of eastern Iran, a very active seismic area in the country.

Seismic stability analysis of earth embankment dams is a complicated task and as such, various methods have been proposed. Each of these methods has its own limitations and usually, a single method is not sufficient. In general, these methods are classified into three (3):

- Deformation based Methods
- Pseudo-static analysis and
- Dynamic analysis.

In the deformation based methods, an estimation of permanent deformation during an earthquake is made and then compared to what is regarded as acceptable deformation. This is commonly carried out using Newmark's sliding block analysis method in which the potential sliding mass is approximated as a rigid body resting on a rigid sloping base. Contact between the potential sliding mass and the underlying slope is assumed as rigid-plastic [3]. Deformation based methods are straightforward but are still used for assessing the stability of earth embankments [4, 5].

Pseudo-static stability analysis is basically a static limit equilibrium analysis in which the effect of dynamic earthquake loading is replaced by a constant equivalent-static acceleration, which produces horizontal and vertical inertial forces acting on the centroid of the sliding mass. Pseudo-static analysis is widely used for seismic stability assessment of earth embankments and slopes [1, 6, 7, 8, and 9].

Dynamic analysis is recommended for important dams, of which failure may lead to high levels of risk. Dynamic analysis essentially involves the estimation of deformation behavior of an earth dam using the finite element or finite difference method [10]. An actual dynamic analysis is an exhaustive task that requires extensive database and specialized skills [5, 11].

Farrokhi earth dam is of medium size, located in a less populated arid region, for which the deformation based and pseudo-static methods are used in the present study. Considering the inherent uncertainties in geotechnical parameters and to obtain realistic results, the pseudo-static analysis is done with probabilistic approach using the Monte Carlo simulation technique. Also, the sliding block method and empirical relationships of Singh et al are used as deformation based methods. A comparison between the results of the three different analysis methods is made at the end of this study and it is shown that, while Singh et al correlations confirm the final outcome of pseudo-static stability analysis, the sliding block method underestimates the earthquake related deformations.

2. General description of the Farrokhi earth dam

The Farrokhi earth dam is a heterogeneous type with a vertical clay core, which is built150 km away from Birjand and 41 km away from Qaen on the Farrokhi River. The height of this dam from the river's bed is 19 m, with crest length of 927 m, crest width of 8 m and a reservoir size of 9 million cubic meters. The width of the river bed which is partly covered with coarse grain sediments is 462.5 m and width of terrace sediments (fine sand and silt) in the right and left sides of the dam axis is 187.5 and 95 m, respectively. The alluvium layer located below the dam embankment is mainly composed of gravel mixed with sand and silt, and its thickness is about 7 m [12].

During the construction of the Farrokhi dam, a portion of the alluvial layer was removed and the clay core was placed directly on the foundation rock. Elsewhere in the dam, the alluvial layer is present and the dam was built (Fig. 1).

A seismic study conducted in the area around the Farrokhi dam, has estimated the maximum horizontal ground acceleration (a_{max}) acting on the construction site as 0.4 g [13]. Also, due to the placement of the dam on loose and saturated granular sediments, there is a possibility of liquefaction to a depth of 3 m [14].

3. Deformation based methods

3.1. The sliding block model

In this method, the effects of earthquakes on embankment stability are assessed in terms of the deformations they produce rather than the minimum safety factor. Here, the potential sliding mass is considered as a rigid body resting on a rigid sloping base and the contact between the potential sliding mass and the underlying slope is assumed to be rigid-plastic [10]. The method suggests that whenever the down slope ground acceleration exceeds a threshold value required to cause collapse, permanent displacements will occur.

The threshold acceleration, which causes down slope permanent displacements is referred to as the yield acceleration, a_y. The yield acceleration (a_y) is the horizontal seismic coefficient, which gives a safety factor of 1 when used in a traditional pseudo-static analysis.

Experience indicates that deformation calculated along the failure plane by deformation based methods should not generally exceed 1 m [10]. By determining the a_y and the a_{max} for the embankment or dam, the permanent displacements of the embankment can be calculated using the upper-bound relationship of Figure 2 suggested by Hynes-Griffin and Franklin [15].

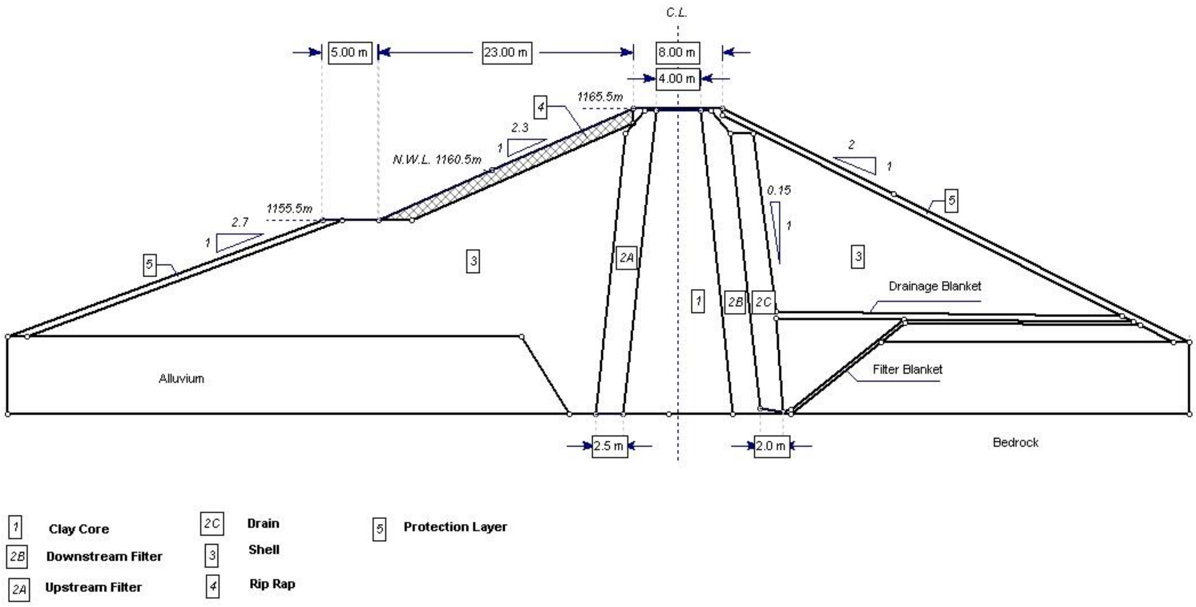

Fig. 1. Cross-section of the Farrokhi earth dam [12]

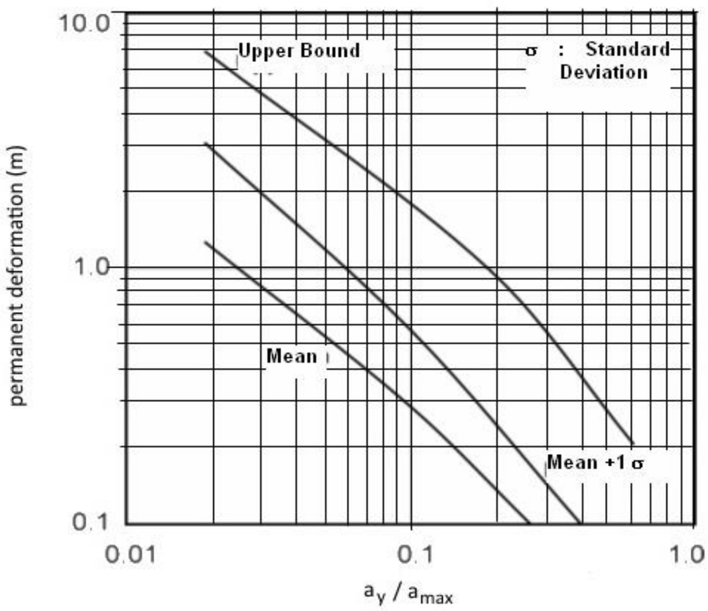

Fig. 2. Relationship between ratio of yield to peak accelerations and permanent deformation [15]

3.2. Singh et al empirical relationships

After the examination of 122 case histories on the performance of earth dams and embankments during past earthquakes, Singh et al. [16] proposed an alternative empirical framework for estimating earthquake-related permanent deformations of earth dams and embankments. Equations 1 and 2 have been proposed as mean and upper bound relationships for the estimation of earthquake-related permanent deformations in which the expected deformations are related to the ratio of yield acceleration to the peak horizontal ground acceleration (a_y / a_{max}).

$$\log D_{avg} = -1.407 - 2.773 \times \log(a_y / a_{max}) - 0.667 \times \{ \log(a_y / a_{max}) \}^2 \quad (1)$$

$$\log D_{max} = -0.694 - 2.684 \times \log(a_y / a_{max}) - 0.652 \times \{ \log(a_y / a_{max}) \}^2 \quad (2)$$

For the purpose of earth dam design, it is safer to use the upper-bound relationship.

4. Pseudo-static stability analysis

The pseudo-static analysis is a primary method for stability analysis of earth and rock-filled dams. Although dynamic analysis of embankment dams has significantly progressed in the recent years, yet the pseudo-static analysis is used in many cases owing to its simplicity and speed. In pseudo-static approach, the dynamic loads are treated as static forces acting on the mass above failure surface. The earthquake effect is considered as a horizontal acceleration caused by the earthquake. This horizontal acceleration is then multiplied by the weight of the sliding mass and when applied to the model, it increases the driving forces and reduces the safety factor. However, the vertical acceleration is waived because of its very little impact on the safety factor [10]. Therefore, the forces exerted during the earthquake are considered as an equivalent horizontal force in the potential ailure direction. During stability analysis by this method) for example, for a mass of clayey soil as shown in Figure 3 a circular slip surface is assumed and in addition to the weight force of the sliding mass which acts in the vertical direction, an equivalent horizontal force: $F = a \times W$ (a is the coefficient of earthquake acceleration) is also applied [17]. For this purpose, the safety factor of stability for sliding mass is calculated using Equation (3). Usually, a less allowable safety factor is considered in comparison with static analysis.

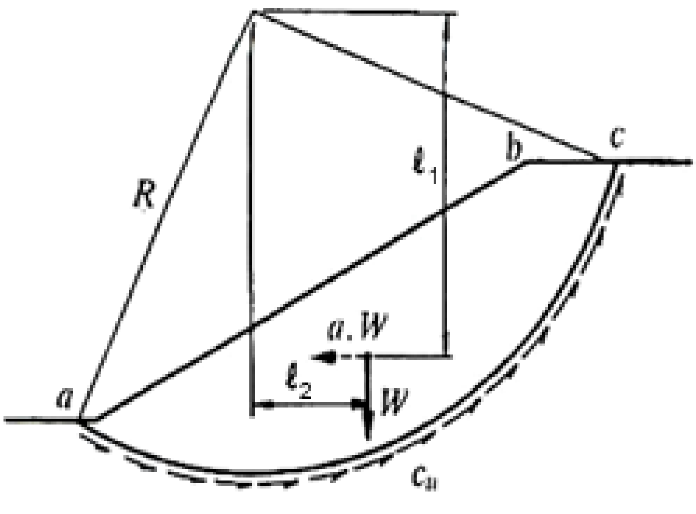

Fig. 3. Schematic diagram for pseudo-static stability analysis using total stress method (Swedish slip circle method)

$$SF = \frac{c_u \times L \times R}{W \times l_1 + a \times W \times l_2} \quad (3)$$

where L = length of a sliding circular arc, c_u = undrained shear strength of soil, R = radius of the slip circle, W = weight of mass above the sliding surface, a = coefficient of earthquake, l_1 and l_2 = distances between the center of sliding mass and the center of rotation in the vertical and horizontal directions(Fig. 3).

In pseudo-static analysis, different limit equilibrium methods can be used in the analysis of slope stability such as Fellnius [18], Bishop [19], Janbu [20], Spencer [21], Morgenstern-Price [22], etc. In this study, Bishop simplified, which is a simple but useful method, is used along with the precise and sophisticated method of Morgenstern-Price.

Usually, in all of the limit equilibrium methods ,the soil mass above the sliding surface is divided into vertical slices. The forces on each separate slice are shown in Figure 4. Here, X_i and E_i indicate the normal and shear forces acting between slices, T_i and N_i are the normal and shear forces exerted on the base of each slice, while W_i is the weight of each slice. Various limit equilibrium methods differ in terms of assumptions made to satisfy the force equations of equilibrium and moment equations of equilibrium with respect to the point O.

In Bishop simplified method, it is assumed that the shear side forces (X_i) may be ignored without introducing serious error into the analysis. The method is based on satisfying the moment equation of equilibrium and the vertical force equation of equilibrium. Safety factor is determined through a successive iteration of Equation (4) [19]:

$$SF = \frac{1}{\sum_i W_i.\sin\alpha_i}.\sum_i \frac{c_i.b_i + (W_i - u_i.b_i).\tan\varphi_i}{\cos\alpha_i + \frac{\tan\varphi_i.\sin\alpha_i}{SF}} \quad (4)$$

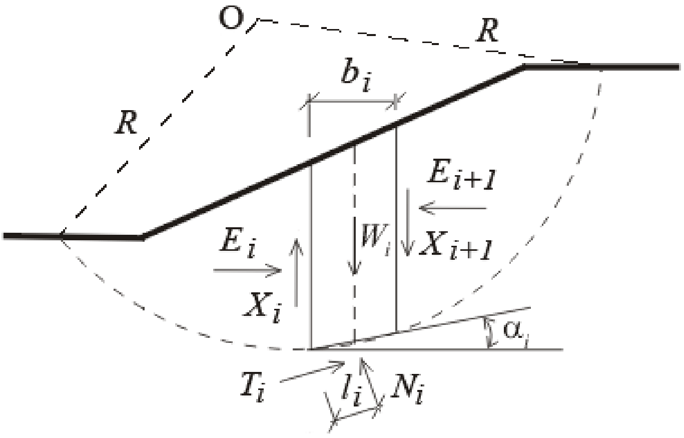

Fig. 4. Schematic diagram of static forces in a slice

where u_i is the pore water pressure in each slice, c_i and φ_i are the effective values of soil parameters, W_i is the slice weight, α_i represents the inclination of a segment of the slip surface and b_i is the horizontal width of each slice.

In the method of Morgenstern-Price, shear forces are assumed to be non-zero between slices. The resultants of shear and normal forces acting between slices have different inclinations at each slice. Morgenstern-Price is a precise method, in that, it satisfies all three equations of equilibrium: the force equations of equilibrium in the horizontal and vertical directions and the moment equation of equilibrium. The safety factor is determined through the iteration of inclination of forces acting between blocks and the safety factor [22].

5. Stability analysis of the Farrokhi earth dam

5.1. Considerations of input parameters for stability analysis of the Farrokhi earth dam

During the stability analysis of earth dams, usually, it is necessary to have a knowledge of the following details:

- The geometrical characteristics of the embankment
- The geotechnical characteristics of filled materials and foundation.
- The pore water pressure in the embankment and foundation.
- The seismicity of construction site.

At the first stage, considering the dimensions of the Farrokhi dam, two-dimensional model of the embankment and foundation was created using Slide 6.0, a commercial software developed by the Rocsience Inc. for limit equilibrium analysis of earthen slopes (Fig. 5). Then, the geotechnical parameters required for different layers of the embankment were selected including density, cohesion (c), internal friction angle (φ) and permeability coefficient (k) as shown in Table 1. The Mohr-Coulomb relationship was defined as the strength criterion for the filled materials and its parameters (c and φ) were selected according to the results of the triaxial compression tests conducted [12].

Table 1. Geotechnical characteristics of the Farrokhi earth embankment dam [13]

Layer	Natural density kN/m^3	Saturated density kN/m^3	Permeability k, m/s	Internal friction angle, $\Phi°$	Cohesion c, kN/m^2
Clay core	20.5	21.2	$1*10^{-9}$	$\Phi_{uu} = 0$ $\Phi_{cu} = 21$ $\Phi' = 22$	$c_{uu} = 42$ $c_{cu} = 45$ $c' = 45$
Shell	22	22.4	$4*10^{-4}$	41.5	0
Filter	20.5	21	$2.1*10^{-4}$	30	0
Drain	22	23	$6.5*10^{-2}$	32	0
Alluvium	20	21	$5*10^{-4}$	43.5	0
Rip rap	18	-	1	50	0

The pore water pressure in the embankment and dam foundation is an essential factor in stability analysis of dams. Therefore, stability analysis is usually performed in three different conditions:

- During construction or immediately after the end of construction;
- Steady state condition of groundwater movement;
- Rapid drawdown condition in the water level of the reservoir.

In each case, different conditions of pore water pressure occurs in the dam embankment and as such, a separate analysis is needed.

A sudden increase in pore water pressure during earthquake-induced vibration of an earth dam requires special attention. If the filled materials are granular, the impact of seismic stresses on the embankment can be analyzed in drained condition with negligible error. But if the filled materials are of fine-grained clay, usually, undrained loading conditions will be assumed. Therefore, unconsolidated undrained (UU) condition for clay core and consolidated drained (CD) condition for the remaining layers was assumed in pseudo-static stability analysis of the Farrokhi earth dam.

The maximum horizontal ground acceleration (a_{max}) for the Farrokhi earth dam has been obtained as 0.4 g, based on a seismic study conducted [13]. Hence, the horizontal acceleration coefficient (K_h) in pseudo-static analysis was assumed equal to half the maximum horizontal acceleration (a_{max}), that is, 0.2 g according to the recommendation of the U.S Army Corps of Engineers [23]. The coefficient of horizontal acceleration (K_h) is a proportion of the maximum acceleration applied to the soil structure. During an earthquake, earth materials experience vigorous shaking, its impact on compacted soils can be loosening of soil, thereby reducing the strength parameters. Therefore, it is usually recommended that soil parameters should be reduced by 20% in a pseudo-static analysis [23].

Another possible effect of earthquake on soil layers is liquefaction. According to previous studies, in the event of design earthquake, at least in part of the Farrokhi dam, an alluvial layer in the upstream side with a depth of 3 m is expected to liquefy [14]. A liquefied soil layer in the dam foundation would decline strongly in terms of strength and therefore, greatly increases the probability of embankment failure (The main cause of destruction of several earth dams in the Bhuj earthquake [1]). So, in any pseudo-static analysis where the simultaneous risk of soil liquefaction exists, the effect of liquefied soil layer on stability analysis must be considered. In steady state and rapid drawdown conditions, the effect of soil liquefaction was considered by a reduction in the strength parameters of the alluvium layer.

Since the Farrokhidam embankment is heterogeneous, and composed of different layers and materials, the non-circular slip surfaces are more likely to occur. Therefore, the Path Search algorithm, which is available in Slide 6, was employed to determine the critical non-circular slip surfaces.

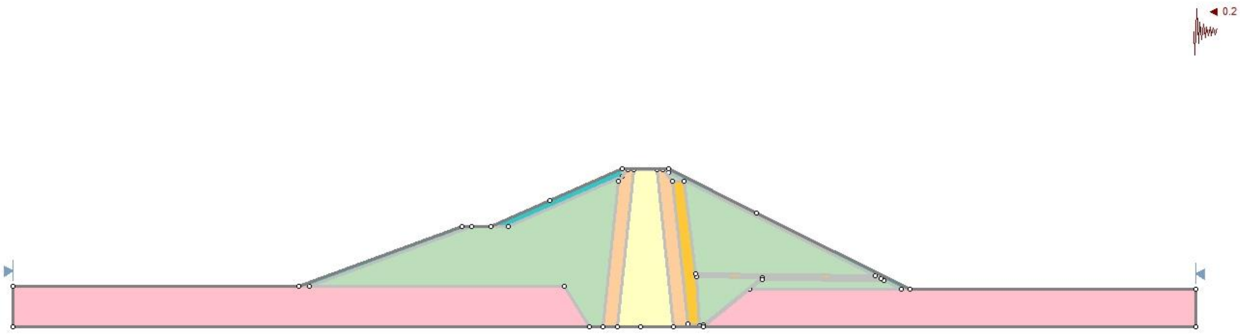

Fig. 5. Two-dimensional model of the Farrokhi earth embankment dam

5.2. Selection of the criteria for stability of the dam

The basic rule of judgment about the stability of a soil structure is the standard safety factor, a simple factor that is easy to interpret by physical or engineering concepts. In the traditional deterministic approach of safety factor in calculation such as that of Equation (3) or (4), any error or uncertainty in the soil parameters is directly reflected in the results. In the recent past, the importance of risk assessment in the dam projects has been emphasized [24], and it has become apparent that the safety factor alone is not sufficient for risk assessment. For this reason, the use of the probabilistic approach that provides additional measures related to the probability of failure and risk rate has become prevalent. For the efficient performance of a probabilistic analysis in a slope stability problem, the use of the Monte Carlo simulation is highly recommended [25]. In Monte Carlo simulation, instead of using a definite values for each soil parameter, random values generated within a variation domain are used to calculate a series of safety factors (e.g., as in Equation 4). To define a variation domain, usually a normal or lognormal statistical distribution is assumed for each soil parameter, then the experimental average of that parameter is multiplied by its coefficient of variation in order to achieve the standard deviation. Random samples required for Monte-Carlo simulation is selected from a range of average laboratory parameters plus / minus maximum three times of standard deviation.

To perform a probabilistic analysis for the Farrokhi earth dam, the strength properties of the different soil layers (that is, c and φ) and the density (γ) were defined as variable inputs. The coefficients of variation of these parameters according to laboratory experiments [26] were selected as shown in Table 2. The normal and log-normal statistical distributions were used for the input variable parameters as recommended by USACE [27].

One of the key factors in Monte Carlo simulation is the number of simulation cycles or the number of data samples. In this study, the mean safety factor for the Farrokhi dam was initially obtained by 10,000 random samples. Then, the number of random samples was increased to 15000 as far as increase in the number of samples would not cause any change in mean safety factor. In the probabilistic approach in addition to the mean safety factor (SF), the probability of failure (PF) and reliability index (RI) were also obtained. The probability of failure (PF) is defined [26] according to Equation (5):

$$PF = \frac{N_F}{N_T} \times 100 \tag{5}$$

Here, N_F is the number of samples with a safety factor below 1 and N_T is the total number of the analyzed samples. Reliability index (RI) is another commonly used criterion for evaluating the stability analysis and is calculated as follows [26]:

In case of using a normal distribution

$$\beta = \frac{\mu_{FS} - 1}{\sigma_{FS}} \tag{6}$$

and, in case of using a log-normal distribution

$$\beta_{LN} = \frac{\ln\left[\dfrac{\mu_{FS}}{\sqrt{\left(1+V^2\right)}}\right]}{\sqrt{\ln\left(1+V^2\right)}} \tag{7}$$

where μ_{FS} and σ_{FS} are the average and standard deviation of the obtained safety factor after N times of Monte-Carlo simulation, respectively. *V* is the coefficient of variation defined as the ratio of $\frac{\sigma_{FS}}{\mu_{FS}}$.

According to the U.S Army Corps of Engineers [27], after a probabilistic analysis, an earth dam could be assumed to be stable if, in addition to an appropriate safety factor, the probability of failure is between 10^{-6} and 10^{-4} and reliability index is 3 to 5.

The main criterion for judgment on the stability of a dam in different loading conditions is the calculated safety factor. In static loading conditions, the allowable safety factor depends on the type of analysis (the end of construction, steady state condition...) but it is often considered to be more than 1.2 [28]. A safety factor of 1.0 is usually considered acceptable in the pseudo-static seismic slope stability assessment [10].

Table 2. Coefficient of variation for soil properties [26]

Soil properties	**Coefficient of variation, %** $V = \frac{\sigma}{\mu}$
Unit weight, γ	7
Internal friction angle, Φ	13
Cohesion, c	20

5.3. Results of deformation based methods

As mentioned earlier, about 0.4 g was obtained as the maximum horizontal ground acceleration (a_{max}) for the Farrokhi earth dam [13]. However, about 0.15 g was estimated as the yield acceleration (a_y) after conducting a conventional limit equilibrium analysis for the Farrokhi dam in full reservoir condition (Fig. 6). Therefore the ratio of (a_y / a_{max}) is obtained as 0.375.

Using the upper-bound relationship of Figure 2, the maximum permanent deformation of the Farrokhi dam estimated by the sliding block model will be about 40 cm. This value is far below the 100 cm as the threshold; hence, the Farrokhi dam with a full reservoir is predicted to be stable in the event of design earthquake according to the sliding block model.

Substituting the values of a_y and a_{max} in the Singh et al upper-bound relationship (Equation 2), the permanent deformation of the Farrokhi dam against the design earthquake is estimated as 214 cm. Thus, according to the empirical correlation of Singh et al, the dam is expected to fail should the design earthquake occur in full reservoir condition.

The two deformation based methods adopted in this study arrived at opposite conclusions regarding the stability of the Farrokhi dam. In the next sections, after the implementation of probabilistic pseudo-static analysis, which is an elaborate and more accurate method, these two conflicting conclusions will be verified.

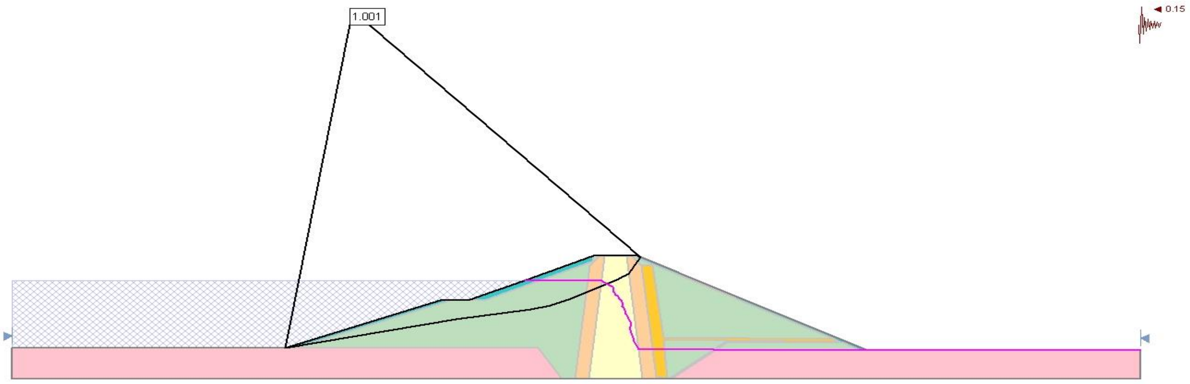

Fig. 6. Yield acceleration (a_y) estimated for the Farrokhi earth dam in the full reservoir condition

5.4. Results of pseudo-static analysis

5.4.1. End of construction stage

In the present study, stability analysis of the Farrokhi earth dam immediately after the end of construction against the design earthquake (a_{max}= 0.4g) was conducted using the Slide 6 program in deterministic and probabilistic conditions. The results of this analysis of the upstream slope are shown in Figures 7 and 8. A good agreement is seen between the two applied limit equilibrium methods, that is, Bishop and Morgenstern-Price. Based on the results, it is noted that the safety factor for the upstream slope is not enough, but close to 1. Also, the probability of failure has exceeded the recommended limit and therefore, the instability of upstream slope in the form of small slides is possible. Figures 9 and 10 show the sliding surfaces with a minimum safety factor for the downstream slope. Also, the biggest size sliding surface with a safety factor less than 1 is shown to have an understanding about the maximum size of the anticipated failures. It is seen that the small to medium-sized failures would occur in the event of design earthquake. The summary of the results of the conducted analysis at the end of the construction stage are given in Tables 3 and 4.

5.4.2. Steady state condition

By assuming a full reservoir with a height of 15 m, the flow of the infiltrated water in the dam embankment was estimated by a finite element routine built in Slide6.

As reported in a previous study [14], if the design earthquake occurs in the full reservoir condition, total liquefaction of the dam foundation in the upstream side of the Farrokhi dam will occur to a depth of 3 m. Due to soil liquefaction, the shear strength parameters of the alluvial layer would reduce sharply, and therefore, cannot be used in the analysis. The post-liquefaction shear strength of the cohesionless alluvium was estimated from Equation (8) [29]:

$$\frac{\tau}{\sigma'_v} = 0.03 + 0.0075(N_1)_{60} \tag{8}$$

where σ'_v is the vertical effective stress and $(N_1)_{60}$ is the SPT number of the soil.

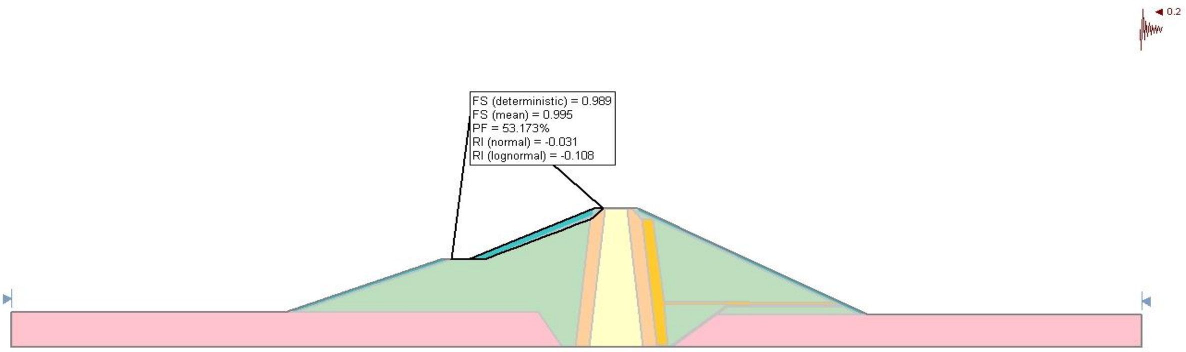

Fig. 7. Results of stability analysis of the upstream slope using Bishop simplified method (at the end of the construction stage, k_h = 0.2 g)

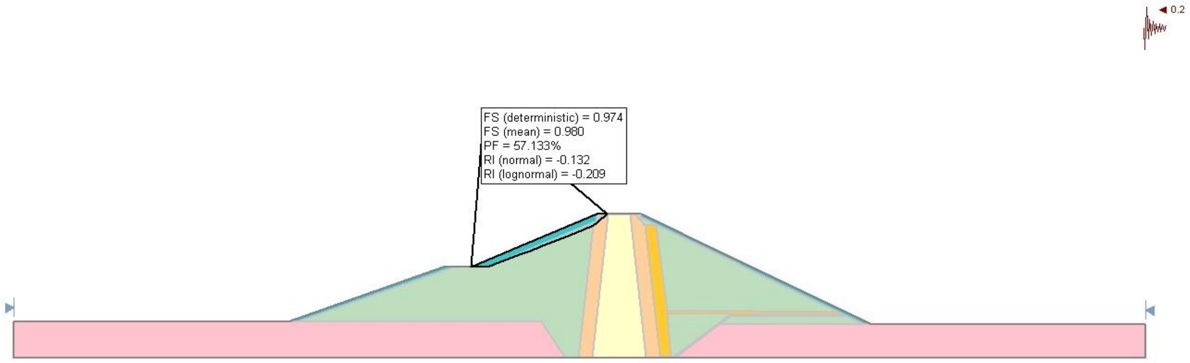

Fig. 8. Results of stability analysis of the upstream slope using Morgenstern-Price method (at the end of the construction stage, k_h = 0.2 g)

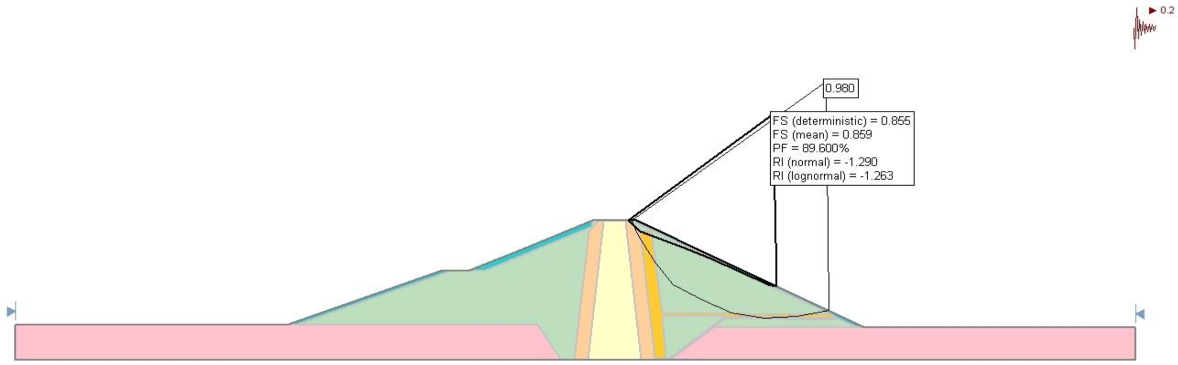

Fig. 9. Results of stability analysis of the downstream slope using Bishop simplified method. At the end of the construction stage, k_h = 0.2 g (*the biggest size failure surface is also shown*).

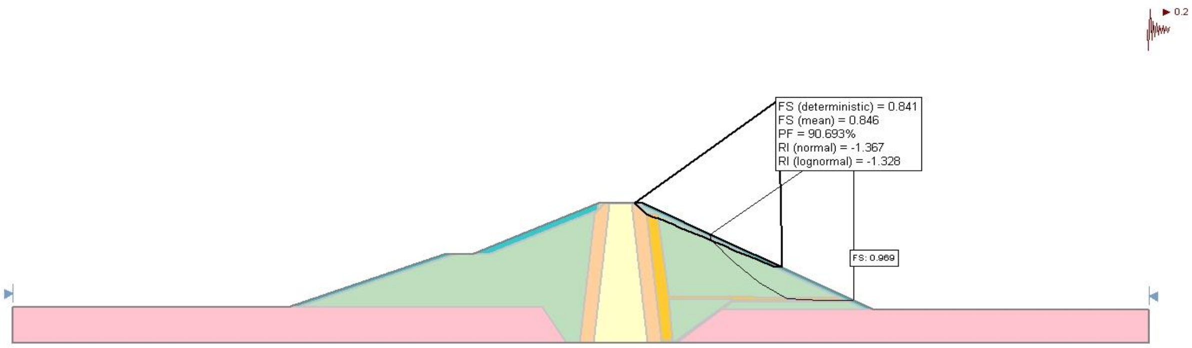

Fig. 10. Results of stability analysis of the downstream slope of the Farrokhi dam using the Morgenstern-Price method. At the end of the construction stage, k_h = 0.2 g (*the biggest size failure surface is also shown*).

Table 3. Safety factor of the upstream and downstream slopes of the Farrokhi dam for end of construction stage with K_h = 0.2 g

Slope	Limit equilibrium analysis method	Safety factor, deterministic	Safety factor,meanprobabilistic
Upstream	Bishop simplified	0.989	0.995
Upstream	Morgenstern-Price	0.974	0.980
downstream	Bishop simplified	0.855	0.859
downstream	Morgenstern-Price	0.841	0.846

Table 4. Statistical results of the upstream and downstream slopes of the Farrokhi dam for end of construction stage with K_h = 0.2 g

Slope	Limit equilibrium analysis method	Probability of failure, PF %	Reliability index, RI (normal)	Reliability index, RI (lognormal)
Upstream	Bishop simplified	53.17	-0.031	-0.108
Upstream	Morgenstern-Price	57.13	-0.132	-0.209
Downstream	Bishop simplified	89.60	-1.290	-1.263
Downstream	Morgenstern-Price	90.69	-1.367	-1.328

By applying the new values for shear strength of the liquefied layer, the pseudo-static analysis was performed. The potential failure surfaces and related safety factors are shown in Figures 11 and 12 for the upstream slope of the Farrokhi dam. Also, the biggest size sliding surface with a safety factor less than 1 is given. As shown in these figures, the probability of failure is 100%, indicating that the upstream slope is indisputably unstable should the design earthquake occur. As shown in Figures 11 and 12, the sliding surface will pass through the liquefied layer beneath the dam. In fact, the existence of a weak liquefiable layer in the dam foundation substantially increases the possibility of large soil slides in the upstream slope.

Figures 13 and 14 show the stability condition of downstream slope in steady state condition. Compared to the upstream side, the downstream slope has a higher safety factor; nevertheless, in the event of design earthquake, it is unstable with a high probability of failure. In the absence of liquefaction layer, there is a reduced change in the stability condition of the downstream side in comparison with the end of construction state. This, in turn, emphasizes the great effect of the liquefaction on the overall stability of earth dams.

The summary of the results of the pseudo-static steady state analysis are given in Tables 5 and 6. The results indicate a serious situation for the Farrokhi dam in the event of a severe earthquake and a full reservoir.

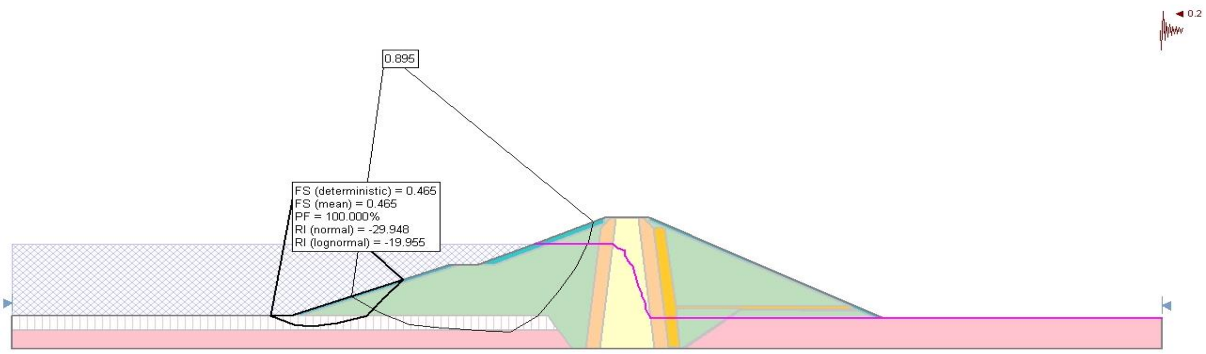

Fig. 11. Results of stability analysis of the upstream slope using Bishop simplified method for steady state stage with k_h = 0.2 g (*the biggest size failure surface is also shown*)

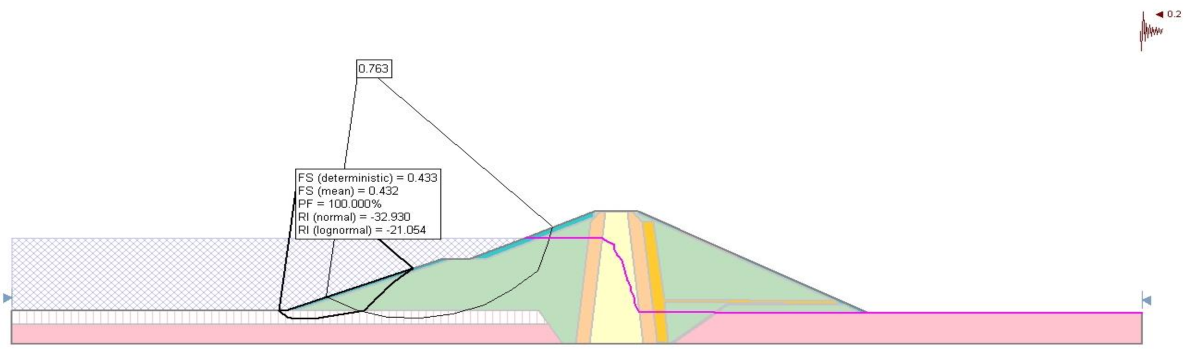

Fig. 12. Results of stability analysis for the upstream slope using Morgenstern-Price method. for steady state stage with k_h = 0.2 g (*the biggest size failure surface is also shown*)

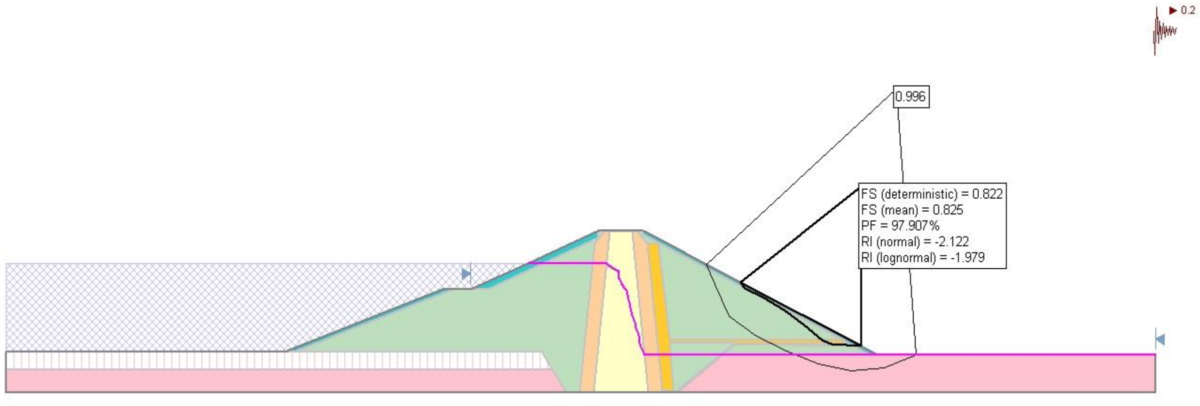

Fig. 13 Results of stability analysis of the downstream slope using Bishop simplified method for steady state stage with k_h = 0.2 g (*the biggest size failure surface is also shown*)

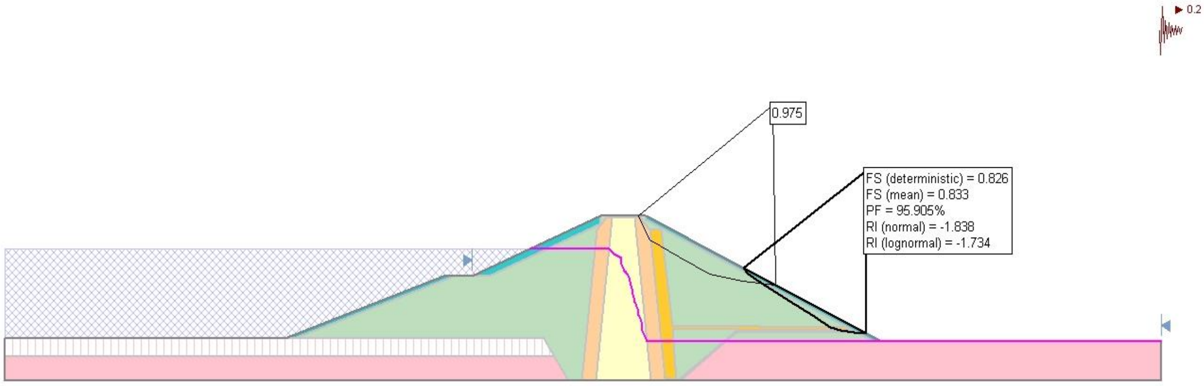

Fig. 14. Results of stability analysis of the downstream slope using Morgenstern-Price method for steady state stage with k_h = 0.2g (*the biggest size failure surface is also shown*)

Table 5. Safety factor for the upstream and downstream slopes of the Farrokhi dam for steady state stage with K_h = 0.2 g

Slope	Limit equilibrium analysis method	Safety factor, deterministic	Safety factor, meanprobabilistic
Upstream	Bishop simplified	0.465	0.465
Upstream	Morgenstern-Price	0.433	0.432
downstream	Bishop simplified	0.822	0.825
downstream	Morgenstern-Price	0.826	0.833

Table 6. Statistical results for the upstream and downstream slopes of the Farrokhi dam for steady state stage with K_h = 0.2 g

Slope	Limit equilibrium analysis method	Probability of failure, PF %	Reliability index, RI (normal)	Reliability index, RI (lognormal)
Upstream	Bishop simplified	100	-29.948	-19.955
Upstream	Morgenstern-Price	100	-32.930	-21.054
Downstream	Bishop simplified	97.91	-2.122	-1.979
Downstream	Morgenstern-Price	95.91	-1.838	-1.734

5.4.3. Rapid drawdown condition

This condition occurs when the water level in the reservoir reduces quickly, but the earth materials of the upstream slope do not drain simultaneously. In this case, due to the removal of the water pressure behind the dam, the pressure equilibrium (in the upstream) between the pore water and the water behind the dam would be lost. In such a situation, due to saturation of the slope materials, the water seepage would be directed to the upstream side shortly. This will create a critical condition in the upstream slope stability. This condition is usually avoided by putting a thick layer of coarse materials on the upstream slope.

To conduct a stability analysis of the Farrokhi dam in rapid drawdown condition, it is assumed that the dam reservoir is initially full, at a height of 15 m, and then reduce quickly to a height of 4 m. The stress analysis is done with effective stress values using B-bar method with the B-bar coefficient specified as 1.

Similar to steady state condition, the effect of liquefaction on the foundation soil and subsequent reduction in shear strength parameters were considered in the analysis. Figures 15 and 16 show the stability condition of the upstream slope in rapid drawdown condition. Also, the biggest size sliding surface with a safety factor less than 1 is given. As shown in these figures, the upstream slope of Farrokhi dam would be unstable in rapid drawdown condition should the design earthquake occur. Similar to steady state condition, the probable failure surface passes through the liquefied layer of the dam foundation. The summary of the results of pseudo-static rapid drawdown analysis are shown in Tables 7 and 8.

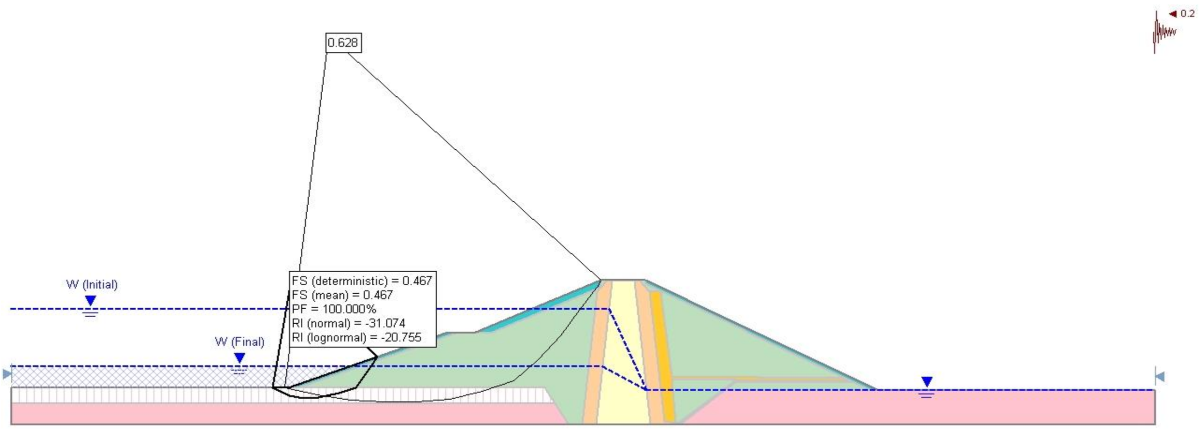

Fig. 15. Results of stability analysis of the upstream slope using the Bishop simplified method for rapid drawdown stage with k_h = 0.2 g (*the biggest size failure surface is also shown*)

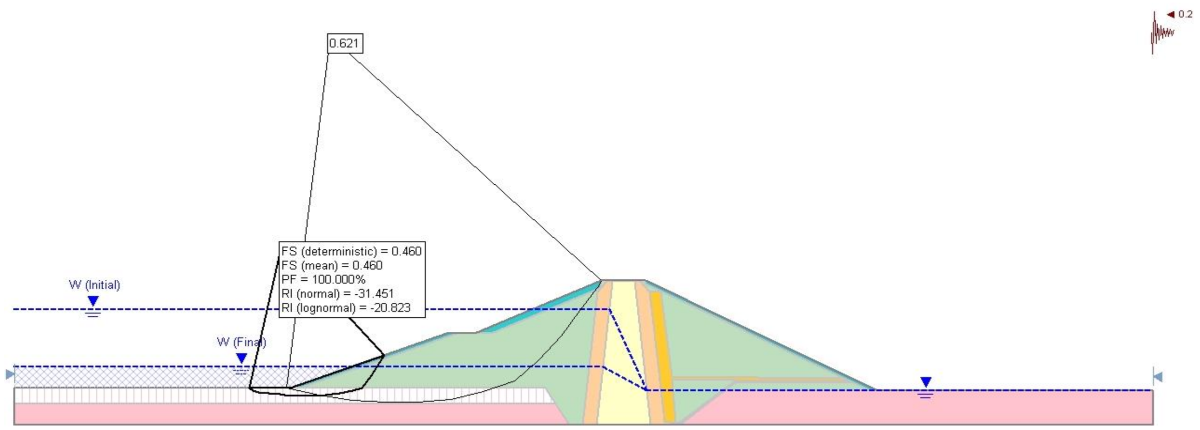

Fig.16. Results of stability analysis of the upstream slope using Morgenstern-Price method for steady state stage, with k_h = 0.2 g (*the biggest size failure surface is also shown*)

Table 7. Safety factor for the upstream slope of the Farrokhi dam for rapid drawdown stage, with K_h = 0.2 g

Slope	Limit equilibrium analysis method	Safety factor, deterministic	Safety factor, meanprobabilistic
Upstream	Bishop simplified	0.467	0.467
Upstream	Morgenstern-Price	0.460	0.460

Table 8. Statistical results for the upstream slope of the Farrokhi dam for rapid drawdown stage with K_h = 0.2 g

Slope	Limit equilibrium analysis method	Probabilty of failure, PF %	Reliability index, RI (normal)	Reliability index, RI (lognormal)
Upstream	Bishop simplified	100	-31.074	-20.755
Upstream	Morgenstern-Price	100	-31.451	-20.823

6. Conclusions

In this study, the stability condition of the Farrokhi earth dam against the design earthquake (a_{max}= 0.4g) was studied using the pseudo static method, sliding block model and Singh et al empirical correlations. In the full reservoir condition, both pseudo static analysis and Singh et al relationship confirmed the major instabilities in the dam embankment, while the Sliding block model underestimated the dam deformations and showed a stable condition.

Pseudo-static analysis of the Farrokhi earth dam was performed by adopting a horizontal ground acceleration coefficient (K_h) of 0.2 g and a 20% reduction (due to liquefaction) in the shear strength parameters of dam foundation in the three following cases as thus explained:

End of construction or empty reservoir stage

In this case, the safety factor for the upstream slope is not enough, but close to 1. Also, the probability of failure exceeded the recommended limit and therefore, the instability of the upstream slope in the form of small slides is likely. The downstream slope is unstable in terms of safety factor and other statistical parameters. Nevertheless, the expected failures in the dam embankment are of small to medium size.

Steady state stage

In this case, the upstream slope of the Farrokhi dam is unstable if a severe earthquake (a_{max}= 0.4 g) occurs in the vicinity. It should be noted that the probable failure surface will pass through a liquefied layer in the dam foundation. Although the downstream slope has relatively better situation, it is still unstable and the possibility of failure is high. In general, the existence of a weak liquefiable layer beneath the dam significantly increases the possibility of large soil slides.

Rapid drawdown in the reservoir level

Similar to the steady state condition, the upstream slope is unstable and the probable failure surface would be of large size and passes through the dam foundation.

In general, the stability of the Farrokhi dam against a severe earthquake, such as historical earthquakes of the region is not guaranteed and as such, small to large size slope failures are expected especially in the upstream side. With regards to this risk, the following actions are suggested to prevent a serious problem in the future:

- Strengthening the alluvial foundation layer particularly under the upstream slope using soil nailing and cement grouting to decrease the possibility of liquefaction and to increase the overall safety factor of the dam stability.
- Estimation of the potential risk of the downstream side in the event of a possible dam failure and adopting the precautionary procedures.

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