DYNAMIC ANALYSIS OF EARTH DAM USING NUMERICAL METHOD – A CASE STUDY: DOYRAJ EARTH DAM

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Abstract
The precise study of the response of earth dams to earthquakes is one of the most complex issues in the field of soil structures. In this research, dynamic analysis of earth dam structures (a case study: Doyraj dam in the west of Iran) have been performed using 2D Finite Difference Method (2D F.D.M.). The aim of this study is to investigate accelerations, lateral (horizontal) and vertical displacements (i.e. settlements) due to earthquake occurrence. The results of dynamic analysis indicate that the performance of the dam is satisfactory for each one of the seismic scenarios considered in this investigation. The maximum settlements at the dam crest is considerably smaller than that of the dam freeboard, with maximum value of 540 mm, which is comparable to recommendation of the Department of Safety of Dams (DSOD). Depth of sliding surfaces is better shown in the Finn model, and the settlements based on the Finn model is about 2.5 times higher than that of Mohr model. In contrast to what is commonly accepted

DINAMIČNA ANALIZA ZEMELJSKE PREGRADE Z UPORABO NUMERIČNE METODE: ŠTUDIJA PRIMERA ZEMELJSKE PREGRADE DOYRAJ

Ključne besede
študija praktičnega primera, dinamična analiza, 2D metoda končnih razlik zemeljska, pregrada

Izvleček
Eno najbolj zapletenih vprašanj na področju zemljinskih konstrukcij predstavlja natančna študija odziva zemeljskih pregrad na potres. V tej raziskavi je bila izvedena dinamična analiza konstrukcije zemeljske pregrade (študija primera: jez Doyraj na zahodu Irana) z uporabo 2D metode končnih razlik (2D FDM). Celj te študije je raziskati pospeške, bočne (vodoravne) in navpične premike (tj. posedke) zaradi pojava potresa. Rezultati dinamične analize kažejo, da je zmogljivost pregrade zadovoljiva za vsakega od potresnih scenarijev, obravnavanih v tej preiskavi. Največje posedanje na grebenu je znatno manjše kot pri prosti višini jeza, z največjo vrednostjo 540 mm, kar je primerljivo z predlogom Ministerstva za varnost jezov (DSOD). Globina drsnih površin je bolje prikazana v Finnovem modelu, posebno, ki temeljijo na Finnovem modelu, pa so približno 2.5-krat večji kot pri Mohrovem modelu. V nasprotnem so splošno sprejetimi vrednostmi potresnih pospeškov (povečanje
about earthquake acceleration (the increase in earthquake acceleration from the base to the top of the dam), it cannot generalize to all cases, and it can be limited to very strong dams or can be related to poor earthquakes.

1 INTRODUCTION

Damage and loss of life caused by earthquakes are immense. This is amplified when accompanied by the collapse of essential infrastructures, such as a dam or a power plant, which have the potential of destroying the entire cities. The water and power supplied by dams are essential for the survival of a community. However, when a dam fails, the destruction is often deadly, causing irreparable damage to the land, the people, and to the economy [1]. In fact, the deformations resulting from the earthquake may cause overtopping of water from the dam, leading to severe damages [2]. Earth dams are widely used throughout the world due to the availability of suitable materials and their flexibility. There are numerous earth dams in each country, including Iran. Dams are used for irrigation, flood mitigation, and hydroelectric power generation purposes.

Due to soil nature and its flexibility, the earth dams have better seismic performances in comparison to concrete dams. However, many earth dams were damaged during strong earthquake, and even collapsed in some serious cases [3]. It is essential to carry out post-earthquake investigation and analysis on the earth dams; however, researches have been focused in this field [4-7]. As Iran lies in one of the world’s most seismically active areas, the main issue in dam management and construction is seismic safety. Therefore, to assure dam safety, proper evaluation of dynamic analysis is crucial.

In general, the assessment of the seismic stability of new or existing dams can be performed via (a) pseudo-static analysis [8], (b) displacement-based (Newmark or sliding block) methods [9-11], and (c) dynamic stress-deformation numerical analysis [12]. These above-mentioned analyses provide insight into the seismic response of zoned earth dams and homogeneous embankments and ascertain the relative significance of various parameters. Such parameters included excitation characteristics (intensity and frequency content), dam geometry (height and existence of stabilizing berms), foundation soil conditions, and the dam’s operation phase (“end of construction” and “steady-state seepage” conditions). Note that in addition to the excitation characteristics, the other parameters under study are crucial to the static stability of earth dams, but their significance in terms of seismic loading is unknown [13].

Although dam failures are rare, studies have been conducted based on such events to understand the causes of those failures. One example is the failure of the Teton dam, an earth dam located in Idaho, the United States. The dam failed on June 5th in 1976, as it was being filled for the first time, owing to internal erosion known as “piping”. The failure caused a huge flood that damaged the city downstream, which cost about 2 billion US$ [14]. The Lower San Fernando dam, which was a 40-meter-high hydraulic-fill earth dam located in San Fernando, California, failed on February 9th in 1971 [3]. In Japan, the Aratozawa dam is a rock-fill imperious-core dam with the height of 74.4 m, located in Kurihara. The Iwate–Miyagi Nairiku earthquake in 2008 caused huge landslides occurred in the left bank of the reservoir from the dam. This caused settlement of the core zone about 20 cm. There was no evidence of severe damage to the dam structure, but it was taken out of operation because of safety concerns [15]. Tschuschke, et al. [16] investigated quality control for construction of tailings dam, they also examine the effects of the applied technology on the condition of the natural environment.

Other researchers have also evaluated the safety of earth dams. For example, Soralump, et al. [17] conducted a dynamic response analysis on the Srinagarind dam by using 213 records of 35 earthquake events and the equivalent linear method for the nonlinear behavior of dam materials. Similarly, Fallah and Wieland [18] conducted an evaluation on earthquake effects and the safety of the Koman concrete-faced rock-fill dam in Albania, by using a 2D F.E. models for the maximum cross section. Their study was undertaken using the equivalent linear method. The dam was checked for the safety evaluation against earthquake with a peak ground acceleration of the horizontal component of 0.45g.

This study aims to gain insight into the behavior of the Doyraj dam (a case study in Iran) in case of the earthquake by using the 2D F.D.M. numerical modeling. In order to select the input motion, seismic hazard analysis was performed by deterministic and probabilistic methods of probable horizontal and vertical acceleration in the dam area. Then, the nearest earthquake record for the region has been selected from the earthquakes that occurred in all around the world as a baseline earthquake. By completing modeling and performing dynamic analyzes, the results have been discussed. The
importance of such research is not only to examine the behavior of a dam or the level of damage it can sustain, but also to preserve it against future earthquakes.

2 NUMERICAL MODELLING

2.1. Analysis Approach

During a seismic event, stress waves propagate through soils and attenuate with distance. Energy dissipation, volume changes, and stiffness degradation of the materials are the factors, which affect this attenuation. During shaking, soils exhibit continuous hysteresis modulus degradation resulting in increasing levels of damping, which in turn decrease the amplitudes of the stress waves. The representation of this material behavior is important in seismic analysis of embankment dams.

Two approaches are conventionally used in simulation of inelastic characteristics of soils subjected to cyclic loading: Equivalent Linear Methods (E.L.M.) and nonlinear numerical methods [19]. In the E.L.M. nonlinear behavior of soil is simulated by adjusting the shear modulus and damping ratio as functions of maximum shear strain in the soil. On the other hand, nonlinear methods use nonlinear constitutive models to represent the nonlinear behavior during cyclic loading. Constitutive equations used in predicting inelastic cyclic behavior of soils can become quite complex and may require various material parameters. Alternatively, simple elastic–plastic constitutive models may be used with additional damping added, to represent inelastic damping behavior [19]. The latter method has been adopted in the present study.

Damping in soils is primarily hysteretic, since energy dissipation occurs when grains slide over one another [20]. During dynamic analysis, as effective stresses decrease with increase in pore pressure, the soil begin to yield and increments of permanent deformation are accumulated. Simultaneous coupling of pore pressure generation with non-linear, plasticity based, stress analysis produces a more realistic dynamic response than that can be achieved with the equivalent linear method [20]. The above-described approach has been verified in the literature through analysis of well-documented case histories [21] and was adopted herein for the dynamic analysis of embankment dams with a center core.

In the present study, a bilinear elastic–perfectly plastic stress–strain relationship with a Mohr–Coulomb failure criterion has been used in the dynamic analyses. In this model, energy dissipation is achieved by plastic flow when shear stresses reach the yield strength. For cycles generating shear stress levels remaining in the elastic range, energy dissipation is achieved by viscous damping. Rayleigh damping consisting of two viscous elements is generally used in the numerical analyses. The two elements of Rayleigh damping are both frequency dependent; one increases linearly with frequency (stiffness damping as a function of strain rate) and the other decreases exponentially with increase in frequency i.e. mass damping as a function of particle velocity [20]. The finite difference grid dimensions were selected taking into account the maximum frequency, $f_d$ of the shear wave that the model could respond to during earthquake loading [22].

2.2. Doyraj Earth Dam

Doyraj Reservoir Dam is located on the river Doyraj at about 13 km north of Moosian and southwest of Ilam province, in Iran. The main purpose of Doyraj dam construction was irrigation of about 10,000 acres of agricultural land in Moosian Plain. The normal level of operation in this dam is 226.5 meters and the level of the river bed is 176 meters above sea level. Seismic monitoring equipment has been located in different parts of the Doyraj dam. The location of the project, and Doyraj dam layout is shown in Figure 1. The maximum height of the Doyraj dam is 58 meters and the operational reservoir
surface is 50 meters. The length of the dam is 1160 m and the crest width is 10 m. In numerical modelling, the dam body was modeled in 8 layers of the same thickness, so that the conditions for the construction of the dam can be simulated.

2.3 Geometry and properties of Doyraj dam

Two-dimensional plane strain models of the embankment were considered in the parametric analyses of this study. The geometry of the model is shown in Figure 2. Doyraj Dam is mainly composed of silty soil and sand. The filter and drainage layers are composed of fine-grained sand for drainage of water leakage and variable deformations between the core and the crater embankment. Suitable materials were used for the core

<table>
<thead>
<tr>
<th>Materials</th>
<th>$\gamma_{dry}$ (kN/m$^3$)</th>
<th>$k_z$ (m$^2$/s)</th>
<th>$E$ (MPa)</th>
<th>$\nu$</th>
<th>$c$ (kPa)</th>
<th>$\phi$ ($^\circ$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Foundation Rock Layer</td>
<td>22.0</td>
<td>$1.0\times10^{-9}$</td>
<td>300.0</td>
<td>0.3</td>
<td>0.0</td>
<td>32</td>
</tr>
<tr>
<td>Foundation Clay Layer</td>
<td>20.0</td>
<td>$3.9\times10^{-5}$</td>
<td>200.0</td>
<td>0.25</td>
<td>0.0</td>
<td>28</td>
</tr>
<tr>
<td>Gravel Shell</td>
<td>21.0</td>
<td>$1.0\times10^{-7}$</td>
<td>100.0</td>
<td>0.33</td>
<td>0.0</td>
<td>38</td>
</tr>
<tr>
<td>Gravel Drainage</td>
<td>19.5</td>
<td>$1.0\times10^{-3}$</td>
<td>90.0</td>
<td>0.25</td>
<td>0.0</td>
<td>39</td>
</tr>
<tr>
<td>Clay Core</td>
<td>20.5</td>
<td>$1.0\times10^{-9}$</td>
<td>75.0</td>
<td>0.25</td>
<td>50.0</td>
<td>13</td>
</tr>
<tr>
<td>Filter</td>
<td>19.5</td>
<td>$1.0\times10^{-5}$</td>
<td>35.0</td>
<td>0.3</td>
<td>0.0</td>
<td>35</td>
</tr>
</tbody>
</table>

Table 1. Properties of materials used in 2D F.D.M. analysis.
and cutting off the clay and concrete. In fact, the area of gravel levee and embankment were constructed with the excavated materials near site of the project. The particle size distribution for different materials of the Doyraj dam is shown in Figure 3. Physical and mechanical properties of the embankment and foundation soils used in the numerical models, are presented in table 1.

2.4. Finite difference modelling

The technique of layered construction was employed in the static stress analyses. Since pore pressure generation during construction was a concern, the following procedures were used to model the static effective stress conditions:

(a) Embankment materials were divided into eight layers and placed sequentially. Following the placement of each layer, other layers were added to the layer and the model was set to run for a time period equal to the estimated time required for actual construction of the layer under field conditions. The time needed was calculated by assuming an embankment construction with rate of 1000 m$^3$/day and multiplying this by the total fill volume of each layer.

(b) The water level was then raised to full the pool. Boundary water pressures were applied along the interior dam surface and the reservoir bed to account for water pressures.

(c) Seepage analysis was performed to achieve steady state conditions within the embankment and the foundation.

(d) Mechanical adjustment of stresses was allowed by performing mechanical calculations in which, flow calculations turned off, and forcing the model to reach equilibrium.

Once initial stresses for the steady state condition were achieved, the following steps were taken to prepare the model for dynamic analyses:

(a) Apply dynamic boundary conditions. In order to enforce free field conditions in the numerical boundaries of the discretized half space foundation, free field boundary conditions available in the code were adopted for the lateral boundaries. For the horizontal boundary, the simulation of outward propagating waves in the foundation was achieved by employing the absorbing boundary conditions which are also available in the finite difference code. Absorbing boundaries have been shown to be effective in absorbing outward propagating waves, and hence simulating half space conditions. The viscous boundaries developed by Lysmer and Kuhlemeyer [23] are adopted in the finite difference code, and were employed in the analyses herein. One restriction in applying absorbing boundaries in numerical modeling for dynamic analysis is that such boundary conditions cannot be simultaneously applied to a model, where acceleration or velocity input is applied; since prescribing acceleration or velocities to a boundary would nullify the effect of the absorbing boundary. In such situations it is necessary to convert acceleration or velocity inputs into stress waves, as described below.

(b) Prepare input motion and apply seismic loading to the numerical model. Horizontal component of the acceleration record from Loma Preita earthquake was applied to the base of the model. Since quiet boundaries were already attributed to the horizontal boundary, the acceleration time history of the input motion was converted to shear stress waves, as described above. A nearly perfect match between the input acceleration time history and the time history recorded after applying the shear wave time history, confirmed the validation of the process employed in this paper. After preparing the input motion by filtering the acceleration time history and converting it to a shear stress time history, the resulting shear stress time history was prescribed to each of the

Figure 3. Grain size distribution of Doyraj dam materials.
grid points at the horizontal boundary in which, absorbent boundary conditions were previously prescribed. 
(c) Rayleigh damping was assigned to each element of the model in the mid-range between the natural frequency of the model and the predominant frequency of the input motion. The value of Rayleigh damping was chosen based on shear strains recorded from undamped analysis, by taking 65 % of the peak recorded strain. The damping ratios obtained by this procedure ranged from 3 % to 7 %. 
(d) Dynamic analysis was performed for the duration of the earthquake, and results were extracted for interpretation and further assessment.

It should be mentioned that for simplification purposes, the hydrodynamic interaction effects of the dam reservoir were neglected. Furthermore, the vertical components of the seismic input were not considered in the seismic loading, since the present study aims to provide qualitative results of the behaviour of a dam under seismic loading conditions. The maximum shear modulus of materials used in the dam body, derived from empirical relationships. In these relationships, the shear modulus can be determined by the porosity of the materials. The relationships to determine the $G_{max}$ for materials of different areas of the dam body used in this research, are presented in table 2.

In order to determine the most probable horizontal and vertical acceleration due to the earthquake in the study area, two approaches of determination of the earthquake magnitude have been considered. 1) Definitive determination and 2) probabilistic determination. In the definitive determination of the earthquake magnitude based on seismic studies and earthquake studies, the most important seismic spring relative to the site of the Doyraj Dam is the hidden fault of the Black Sea Anticline "SZ4". This fault has the maximum seismicity ($M_s = 7.0$) at 42 km north of the dam site and approximately 13.6 km of the earth surface; with the maximum horizontal and vertical acceleration of 0.3 and 0.19 g, respectively. As compared to other seismic springs, that is the highest one.

In the probabilistic method for determining the earthquake magnitude, a large return period has been estimated for the radial range of 100, 150 and 200 kilometers around the Doyraj Dam site using Gothenburg-Richter preparatory method, the final fit method, and the maximum likelihood estimator or probable maximum (KIKO) method. By comparing the various methods presented for determining the maximum return magnitude, it is concluded that the method of estimating the maximum exponential (KIKO method) is closer to reality because this approach has been used besides the use of the Guthenberg-Richter cumulative distribution function of historical earthquakes. Figure 4 shows the graphs of the return periods of surface magnitudes in the studied areas. Finally, the results of calculations based on a range of 100 km with a conservative view and appropriately fitted to seismic springs for earthquake hazard analysis are attributed. In this regard, an earthquake event with a magnitude of $M_s = 5.5$ has a return period of about 100 years, an earthquake with magnitude given at $M_s = 6.1$ has a return period of about 500 years, an earthquake with $M_s = 6.3$ has a return period of about 1000 years, and an earthquake with $M_s = 6.6$ will have a return period of about 2000 years. However, seismicity exceeding $M_s = 6.6$ corresponds to a return period over 2000 years.

By studying maximum acceleration of probabilistic method for the 1000-year return period, the maximum horizontal and vertical acceleration were projected to be 0.24 and 0.16 g, respectively. In order to study the seismic behavior of earth dams in this study, the time history of Loma Prieta earthquake acceleration was selected as the input movement in the dam foundation section. Horizontal time histories of the desired earthquake were considered as a two-way input to the dam base for a period of 40 seconds in the dynamic analysis. In the Loma Prieta earthquake, the maximum horizontal and vertical acceleration at high level of the MCL design for this acceleration is 0.367 and 0.28 g, respectively. This historical history has the closest horizontal and vertical acceleration to the most probable predictable earthquake in that area. Record of the earthquake input is shown in Figure 5.

<table>
<thead>
<tr>
<th>Materials</th>
<th>$G_{max}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Foundation Rock Layer</td>
<td>1.62 GPa</td>
</tr>
<tr>
<td>Foundation Clay Layer</td>
<td>1.62 GPa</td>
</tr>
<tr>
<td>Gravel Shell [24]</td>
<td>$G_{max} = 13000 \frac{(2.17 - \varepsilon)^2}{(1 + \varepsilon)} (\sigma)$</td>
</tr>
<tr>
<td></td>
<td>Kokusho &amp; Esachi (1981)</td>
</tr>
<tr>
<td>Gravel Drainage [24]</td>
<td>$G_{max} = 8400 \frac{(2.17 - \varepsilon)^2}{(1 + \varepsilon)} (\sigma)^{0.55}$</td>
</tr>
<tr>
<td></td>
<td>Kokusho &amp; Esachi (1981)</td>
</tr>
<tr>
<td>Clay Core [25]</td>
<td>$G_{max} = 3270 \frac{(2.973 - \varepsilon)^2}{(1 + \varepsilon)} (\sigma)_{0.5}$</td>
</tr>
<tr>
<td></td>
<td>Hardin &amp; Black (1968)</td>
</tr>
<tr>
<td>Filter [26]</td>
<td>$G_{max} = 220K_{max}^{0.9} (\sigma)_{0.6}$</td>
</tr>
<tr>
<td></td>
<td>$K_{max} = 59$</td>
</tr>
<tr>
<td></td>
<td>Seed &amp; Idriss (1970)</td>
</tr>
</tbody>
</table>
Figure 4. Return period of earthquakes according to surface magnitude in the radial ranges of 100, 150 and 200 km.

Figure 5. The recorded of horizontal acceleration time history for the Loma Prieta earthquake at the base of the Doyraj dam.

Figure 6. The model of Doyraj dam in F.D.M. analysis.

The Meshing of the Doyraj dam contained 15360 quadrangular and triangular elements. For investigating of the displacements in the static and dynamic analyses, a nonlinear model with Mohr-Coulomb failure criterion has been used to simulate the elasto-plastic behavior of the foundation material and the embankment body of the dam. For the pore water pressure analysis, the fine model is used. Figure 6 shows the model built in the F.D. environment.

3 RESULTS AND DISCUSSION

3.1 Static Analysis of Doyraj Dam

Prior to studying the dam response to earthquake loading, the construction of the earth dam was simulated to reproduce the initial state of effective stress. Actually, static analysis has been performed with the aim of
reviewing and controlling the static behavior of the Doyraj dam. The horizontal and vertical displacement of the Doyraj dam at the time of completion (10/02/2013; and before impounding) and at the first impounding is shown in Figure 7. The horizontal displacement changes of the dam is presented in Figure 7.a and after impounding is shown in Figure 7.b. The results of vertical deformations of the dam at the time of construction and after impounding are shown in Figures 7.c and 7.d, respecti-

![Figure 7](image1.png)

**Figure 7.** The amount of settlements, pore water pressure and horizontal displacements in the Doyraj dam.

![Figure 8](image2.png)

**Figure 8.** Excess pore pressures and computed settlement profiles.
tively. In Figure 7.e, the pore water pressure is shown in which, the dam located at the impounding stage.

Due to the construction process of the embankment, the clay core has vertical settlements about 530 mm. With reference to Figure 8.a, it is observed that with the different heights of the embankment in the central axis of the core, which has not yet been completed, the most settlements are located at about one third of the center of the clay core, which is approximately 540 mm. However, there is not much change in the amount of settlements after impounding and before impounding in the clay core. In Figure 7.b, it is clear that the amount of horizontal displacement variations has doubled compared to before impounding, increasing from 60 mm to 120 mm. In fact, the increase in horizontal displacements in the dams is due to increased reservoir pressure. In the case of a reservoir drop stage, the elastic deformation of the dam material because of impounding can be improved due to discharge of the reservoir. Figure 8.b shows the consolidation process at different times in the downstream vertical axis. As it can be seen, the amount of change in the consolidation settlements with the increase in time is not much noticeable and the validity of this issue can be seen in Figure 8.c, so there is not much excess pore pressure in the dam. Furthermore, as shown in this figure, the excess pore water pressure is reduced over time.

In the analysis stage of the permanent leakage of the dam, it is assumed that the reservoir water height has its maximum value (approximately 51 meters). Therefore, an analysis of the steady state seepage was performed for the water level of 51 m. Steady state seepage analysis was performed separately from the mechanical analysis. Therefore, according to the patterns, the maximum sum of the settlements is located at one third of the dam height from the foundation, with maximum value of 540 mm. The maximum pore water pressure is also obtained at around 700 kPa.

### 3.2 Dynamic Analysis of Doyraj Dam

Dynamic analysis performed to investigate the probable behavior of the dam during an earthquake. Using the initial stress state obtained from the previous static analysis, the dam has been analyzed using the Loma Prieta earthquake history as an input quake. The earthquake time incoming on the dam was about 40 seconds.

#### 3.2.1 Acceleration Distribution Inside the Dam Body

Acceleration contours showed that the maximum acceleration at the end of the earthquake was about 2 m/s². But the history of acceleration at the crest and the maximum acceleration at the crest of the analysis in 8 seconds step, was calculated about 5.3 m/s².

![Figure 9. Horizontal and vertical acceleration changes on upstream and downstream slopes.](image-url)
History of acceleration time for 6 points from the upstream slope and downstream slope of the dynamic finite difference analysis indicated in Fig. 9 (Point d-f and a-c). Peak acceleration at 6 points in front of its height is depicted in Figure 9, showing that from the base of the dam to its crest, it first increased and then slightly decreased. As it can be seen, the amount of horizontal and vertical acceleration changes is higher than the rest of the points in the middle third part of the upstream and downstream shells; this is also indicative of the possibility of slipping wedges in these areas. In addition, the horizontal acceleration is higher than the vertical acceleration peak in the dam. Given that the motion of the bed is stronger in the vertical direction, it is clear that the dam is substantially strengthened in the direction of the horizontal motion of the earth. Peak acceleration direction at 6 points is also shown in Figure 9. Peak acceleration in the upstream of slope is mainly downward to upstream, while in the downstream of slope, it is often upward. Finally, at the same elevation, the peak acceleration in the upstream of slope is larger than the downstream of slope indicating that there is more seismic deformation on the upstream side of the dam. Another matter that can be stated is that, the acceleration at the point H located at the bed of the foundation is slightly higher than the rest of the points, and the cause can be attributed to the effect of the shear force caused by the earthquake on the base of the foundation and where the earthquake force it enters. So, with respect to this, we can say that Finn model is not very successful in acceleration analysis in the model based on performance of this model which proves the opposite belief about this model. As shown in Figure 10, in relation to the acceleration on the crown and the base of the foundation, the two Finn and Mohr models have been the opposite of each other.

3.2.2 Deformations

Deformations can be used to assess the safety of the dam due to the loss of free height. After dynamic analysis, the calculated changes for comparing the damping behavior presented in Figure 11, which show horizontal and vertical contours, respectively. Deformations contour indicate that the maximum settlement at the dam was 61 cm. It is obvious that the settlements in the clay core have continued to the crest. However, horizontal deformations in the clay core are less than the upstream and downstream shells. Patterns show that the maximum horizontal displacements occur in the downstream shell.
which is approximately 60 cm in size. Of course, the depth of horizontal displacements is not much on the downstream slope. The results of this section are based on Mohr model.

Figure 12 show the location of the monitoring points. As shown in this figure, these points have experienced the modest changes in horizontal displacements. Figure 12.a shows the history of horizontal displacement during dynamic loading based on the Mohr model in the monitoring point. Figure 12.c also shows the history of horizontal displacement during dynamic loading based on the Finn model in the monitoring point. It is observed that the most horizontal displacement in the Mohr model occurred about 38 cm at the point "f". However, in the Finn model, the maximum horizontal displacement of about 46 cm occurred at the same point. Figure 12.b and 12.d show the history of the vertical displacements at the monitoring points based on the Mohr and Finn models, respectively. It is observed that the maximum settlements at the point "f" occurred on both models. But the difference between the amounts of settlements in two models is considerable. The settlement based on the Finn model is about 2.5 times higher than that of Mohr model. This proves that the Finn model shows the displacements more than the predicted value. In general, according to the obtained results, the lasting displacement in the side of the crest of the dam has its highest value.

3.2.3 Strains

Shear strain provide information for understanding the locations inside the dam that may have been damaged during severe earthquake stimulations. Shear contours

![Figure 12](image-url)
of dynamic analysis for different models are shown in Figure 13. Most parts of the dam body have experienced a small amount of shear strain. Shear strains can be seen in the upper part and close to the crest. In fact, at 6 seconds from the occurrence of the earthquake, the shear strain had more penetration in the dam body. Thereafter, the occurrences of large shear strain can be observed clearly on both sides of the dam in the middle and bottom portions of the slope downstream. As seen in Figure 13.a, in Mohr model, all the shear strains have occurred on a slope downstream, but in the Finn model (Figure 13.b.), all shear strains happened in the upper slope of the dam. This shows that the Finn model considers saturated areas more critical. It is even observed that the depth of shear strain penetration in the Finn model is more than that of Mohr model.

![Shear strain based on Mohr model.](image1)

![Shear strain based on Finn model.](image2)

Figure 13. Counter of shear strains at different times of dynamic analysis.

3.2.4 Stability of Doyraj Dam

Changes in the coefficient of slope stability of an earth dam can occur during different periods from construction to operation time; therefore, stability analysis of the upstream and downstream slopes of the earth dams in different conditions, the operation of the reservoir of dams, execution of the body at the end of construction are important. Significant parts in this matter are the design, implementation and operation of the dam body at various stages. Therefore, in the present study, static and seismic properties of the Doyraj earth dam located in Ilam province, which is in operation at the moment, have been investigated. For this purpose, F.D.M. and Newmark methods have been used. The results show that the dam has a fixed stability in the static and seismic states. FOS for upstream and downstream in F.D.M. analysis was 1.57 and 1.49, whilst for Newmark method the values were 1.44 and 1.55, respectively.

The results of deformation analysis for the Loma Prieta earthquake are shown in Figure 14. The amount of permanent deformation obtained by this analytical method is about 470 mm, which is close to the results of F.D.M. analysis. Although the Newark Slip Block Method cannot fully model the mechanism of settlements occurrence in an earth dam in the case of an earthquake, the previous studies have shown that if the appropriate assumptions were considered in the analysis, the displacement values could be in good agreement with both elasto-plastic dynamic analysis and the method used in this research (Newmark Slip Block).

![Deformation Newmark](image3)

Figure 14. Permanent deformation of the Doyraj earth dam using Newmark’s analytical method.

4 CONCLUSION

Based on 2D F.D. analysis of a case study earth dam located in the west of Iran (Doyjay earth dam) the following results obtained:

In the static state, major settlements occur at the center of the core. The maximum settlement of the dam body is 530 mm at the end condition which occurs in the middle of the core. The maximum horizontal displacement at the end of the construction is about 6 cm. In fact, the
increase in horizontal displacements in the dams is due to increased reservoir pressure, since the amount of horizontal displacement variations has doubled compared to before impounding. The impounding does not change the amount of settlements, but the horizontal displacement of the dam reaches 120 mm. In this investigation for the settlements of the crest dam, the assumption of a sliding circles runway from the middle section of the upstream and downstream slopes (Middle Dam), are more critical. The maximum amount of sustained settlements by the Newmark sliding block method is 47 cm which is far less than the Free Board, accelerated magnification values at the crest of the dam in the seismic analysis for accelerated mapping used between 2 and 5, indicating significant acceleration in the dam body. Nonlinear analysis showed that the amount of cyclic shear strains inside the core are very small compared to other parts of the dam. Deformations contour indicate that the settlements in the clay core have continued to the crest. However, horizontal deformations in the clay core are less than the upstream and downstream shells. The static stability coefficients of the downstream slope are close, in all cases. This demonstrates that in a static state, the safety of embankment is not affected by changes in water level in the reservoir. The maximum shear strains distribution in the dam body, especially on the upstream and downhill slopes, are obtained by the accelerograms, which can represent a lower coefficient of certainty against the failure of susceptible levels of slippage. Sliding surfaces is shown deeper in the Finn model. Contrary to what is commonly accepted (the increase in earthquake acceleration from the base to the dam is always expected), this is not a general phenomenon and it can be limited to very strong dam (whose behavior remains earthquake-resistant, i.e. elastic) or related to poor earthquakes. Conversely, due to strong earthquakes, some parts of the dam may reach plasticity. Earthquake acceleration does not increase along the altitude. However, at the same time, the body of the dam gradually surrenders and may collapse.

REFERENCES


