Seismic Analysis of Rockfill Dam, a Case Study of Ghoocham Dam

Hooman Rezaee[1], Hamidreza Hajihassani[2], Shahram Mahzarnia[3]

1- Rock mechanics engineer of Tahkimkav Consulting Engineers Company
2- Project Manager of Tahkimkav Consulting Engineers Company
3- Project Manager of Iran Water and Power Development Company

Email: Hooman.rezaee@mi.iut.ac.ir

Abstract

Ghoocham dam is 45 m height ECRFD type with a vertical clay core and soft alluvial and rock foundation under construction in west of Iran. The study of the seismic response of rockfill dams is a complex problem that generally requires the use of dynamic methods of analysis. The non-linear analysis is established to calculate the permanent deformation caused by the earthquake. In this paper, the seismic analysis of Ghoocham dam based on non-linear approach is presented. The dynamic analyses were conducted as non-linear two-dimensional analyses using the Fast Lagrangian Analysis of Continua (FLAC) computer code. The initial condition of stress and pore pressure is performed for the steady state condition. The initial condition of stress and pore pressure calculated in construction stage was performed for analysis at steady state condition. Time-acceleration histories of two different earthquakes with a magnitude of 6.9 in Richter scale and the maximum peak acceleration (PGA) of 0.47g were applied in analysis. The maximum deformation and the failure pattern are obtained through the non-linear analysis. The analysis results show that permanent deformation is in the acceptable range and stability of dam is secured after the earthquake.

Keywords: Rockfill dam, seismic analysis, non-linear analysis, numerical analysis.

1. INTRODUCTION

Dams play a significant role in fulfilling the increasing demand of water for municipal and agricultural purposes. Embankment dams have become very popular among dam engineers since available materials of different types at site could be used in appropriate zones of dam. Safety of rockfill dams depends on the proper analysis, design, construction and monitoring of actual behaviour during the construction and the operation of the structure. The stability of dam during and after earthquake is a very important safety issue. The solution of geotechnical engineering problems is complicated. One of the reason is the geotechnical problems generally involve large nonhomogeneous structures. The solution of dynamic problems is complicated because soil exists in general as a multi-phase solid-fluid system. Further, the behaviour of the soil is highly nonlinear, anisotropic and hysteretic [1-2]. Under these conditions, the analytical solutions of geotechnical problems have necessarily involved many simplifying assumptions. However, the development of computing facilities has now made it possible to solve these complicated problems more exactly using numerical methods. Nowadays the non-linear dynamic analysis approach is common regarding the development of computational methods and advanced numerical analysis. In this category of analysis, an appropriate constitutive model can be assigned to soil materials and, moreover, a real acceleration-time history is used to calculate the seismic deformations. Considering these factors, the results of the non-linear method are supposed to reflect the real dynamic behavior of soil materials. Ghoocham dam is an ECRFD type which is under construction in west of Iran. In this paper, to obtain the seismic response of Ghoocham dam during and after earthquake, the seismic analysis was done based on non-linear approach.

2. GEOMETRY

Ghoocham dam is an ECRD with vertical clay core material with 45 m height. The length of dam crest is 1820 m at the elevation 1856 m.a.s.l. The slopes of both dam abutments are gentle, around 12%-15% and 600 m of the central part of dam foundation is almost flat. Upstream and downstream slopes of rockfill shell are 1:1.7 and 1:1.5 (V:H), respectively. The slope of central core is 1:0.35(V:H).

Due to weak rock foundation, two stabilizing berms at upstream and downstream of dam with 16 & 50 m width have been designed in central part of dam. At maximum height section of dam, two 3 m and 2 m width fine filter and transition zone are considered at both upstream and downstream of core with the same slope of central core. Figure 1 shows the typical cross section of Ghoocham dam. Shell materials are obtained from two limestone and andesite-bazalt quarries close to the dam site. Materials for zones 4, 5A, filter and transition are obtained from limestone quarry, and material for zones 4A and 4B are from andesite-bazalt quarry. Zone 7 material is from mandatory excavation of fine alluvial and rock foundation of dam body. Foundation of dam consists of 4-14 m thickness alluvial material and weak rock of tertiary quaternary unit of sanandaj-sirjan zone underneath.

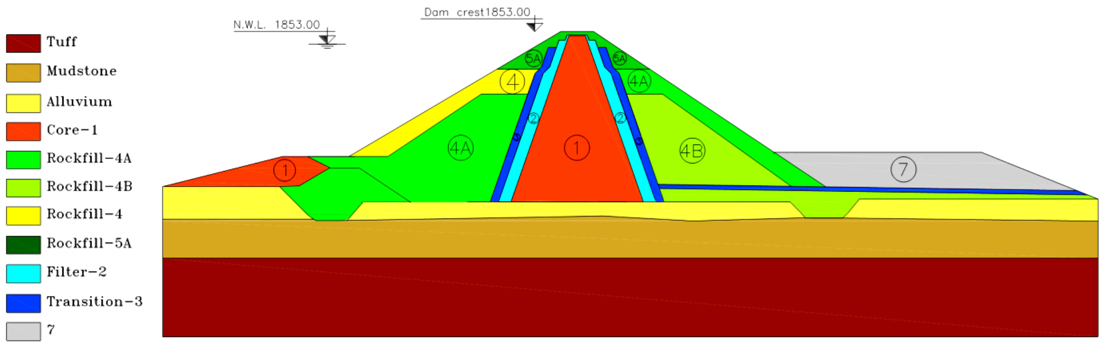

Fig 1. Typical cross section of Ghoocham dam

3. Foundation And Dam Body Material

3.1. Overburden and foundation

The overburden of Ghoocham site with 4-14 m thickness consists of alluvial, slopewash and residual soils. The type of soil is majority silt, clay with a thin layer of sand and gravel. The Consistency of overburden is classified as stiff to hard based on standard penetration test (SPT). A complete set of site and laboratory tests have been performed on these materials. Summary of overburden material parameters is shown in table 1.

Table 1. Geomechanical parameters of overburden.

Materials	d (kg/m3)γ	E (MPa)	υ	C (KPa)	φ	k (cm/sec)
Alluvium	1800	10 - 40	0.25	34	19	3×10^{-7}

Rock types of foundation at Ghoocham dam site are mudstone, tuff and weak conglomerate. The average of rock quality designation (RQD) of rock is 81. The different types of foundation rock parameters are shown in table 2. Based on this table, the cohesion of rock is 40-50 KPa, internal friction angle is 22-25 degree and deformation modulus ranges between 470-800 MPa. The permeability of the rock is around 3×10^{-7} cm/s.

Table 2. Geomechanical parameters of dam foundation.

Materials	d (kg/m3)γ	E (MPa)	υ	C (KPa)	φ	k (cm/sec)
Tuff	2400	470	0.3	40	22	3×10^{-7}
Mudstone	2180	800	0.3	50	25	3×10^{-7}

3.2. Dam body material

Clay material of central core has been obtained from 0.5-1.5 Km upstream borrow areas in dam reservoir. The core material is classified as CL based on unified classification, average PI of material is 22 and compacted with 2-3 percent moisture more than optimum water content. The maximum size of core material aggregates is 25 mm. fine and coarse filters at both upstream and downstream of core zone are processed from limestone quarry. D_{15} of fine filter is 0.45 mm with 10 mm maximum size aggregate (MSA). The MSA of coarse filter is 50 mm. The upstream outer shell zone (zone 4) and upper elevation shell (5A) are from limestone quarry. The other shell zones (4A & 4B) are andesite-bazalt type rock from basalt quarry. The layer thickness of zones 4, 4A and 4B is 60 cm. The maximum percent of passing sieve no.200 for zones 4 and 4A materials is 5 percent and for zone 4B material is 15 percent. The filling of shell zones has been done with wet procedure. A complete series of laboratory tests including large scale triaxial, shear and index tests performed on shell materials. Geomechanical parameters of different zones of dam body are listed in table 3.

Table 3. Geomechanical parameters of dam body

Materials	**$_d$ (kg/m3)γ**	**υ**	**C (KPa)**	**φ**	**k (cm/sec)**
Core	1650	0.35	41	28.8	$4×10^{-7}$
Filter & Drainage	1900	0.25	0	38	$1×10^{-3}$
Rock fill(4)	2150	0.25	38	43	$1×10^{-5}$
Rock fill(4A)	2150	0.25	0	43.2	$1×10^{-5}$
Rock fill(4B)	2150	0.25	0	43.2	$1×10^{-5}$
Rock fill(5A)	2150	0.25	38	43	$1×10^{-5}$
Zone7	1900	0.25	30	30	$1×10^{-5}$

Adequate information on dynamic soil properties, including dynamic shear modulus, damping ratio, pore pressure response and cyclic strength, are more essentially considered in ground response and soil–structure interaction problems. Dynamic behaviour of soils under cyclic load is different from static behaviour. The soils have a non-linear behaviour under cyclic load. As shown in Fig. 2 the shear modulus (G_{max}) of soil in cycle number 1 is decreased to (G_1) and (G2) in cycle number 2 and 3 [1-2]. An accurate dynamic analysis needs to obtain the soils parameters such as shear modulus, damping and etc. The shear modulus can be determined from field measurement techniques, such as down-hole, up-hole, cross-hole, seismic refraction surveys or from the tests such as resonant column and ultrasonic wave propagation in the laboratory. Cyclic triaxial, cyclic simple shear and hollow cylindrical simple shear tests on the other hand, are suitable for the strain levels on the order of 10^{-3} to 10^{-2} [1]. Many researchers have suggested equations based on laboratory and field measurements, to evaluate the maximum shear modulus [1-3-4]. Table 4 shows the equations which used in this paper to calculate the maximum shear modulus of dam body and foundation. Dynamic response of earth and rock fill structures is very dependent on an amount of energy internally dissipated through the soil during shaking. This mechanism of energy dissipation is commonly referred to as damping. Damping in soil is caused by viscous properties and plastic deformation during the shaking and it is to cause energy losses of dynamic load. In general, the initial damping ratio of soils material is assumed between 2% to 5%.

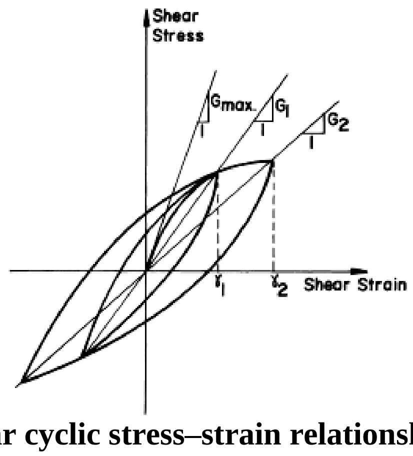

Fig 2. Nonlinear cyclic stress–strain relationship of soils [1]

Table 4. Maximum shear modulus equations of materials

	Materials	Shear Modulus
(1)	Rockfill	$G_{max} = 13000\,\frac{(2.17-e)^2}{1+e} \times (\sigma'_{ave})^{0.55}$ [3]
(2)	Core	$G_{max} = 3270\,\frac{(2.97-e)^2}{1+e} \times (\sigma'_{ave})^{0.5}$ [3]
(3)	Filter & Drainage	$G_{max} = 220 \times 60 \times (\sigma'_{ave})^{0.5}$ [4]
(4)	Alluvium	$G_{max} = 220 \times 90 \times (\sigma'_{ave})^{0.5}$ [4]

4. Numerical Analysis

Numerical analyses were conducted using the finite difference program FLAC^{2D} based on a continuum finite difference using the Lagrangian approach. The maximum cross section of the dam was selected to analyze. Fig. 3 shows the finite difference mesh of Ghoocham dam. The boundary condition of the model is fixed at the base at which the seismic loading is applied ,and is free field at laterals to prevent wave reflection from the sides .Kuhlmeyer and Lysmer (1973) showed that for an accurate representation of the wave transmission through the soil model, the spatial element size ΔL, must be smaller than approximately one-tenth to one-eighth of the wavelength associated with the highest frequency component of the input wave [5]. In the numerical model the element size is about 2m.

$$\Delta L \leq \lambda/10 \qquad (1)$$

λ is the wave length associated with the highest frequency component that contains appreciable energy.

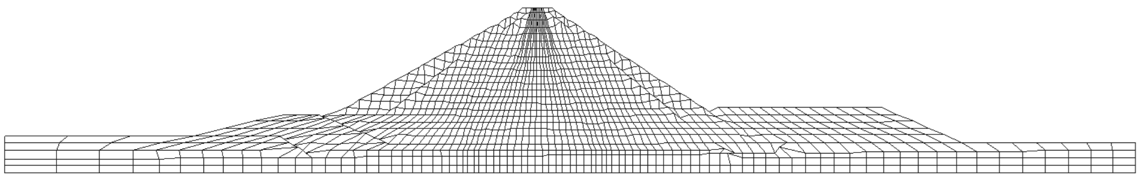

Fig 3. Finite difference mesh

The initial condition of stress and pore pressure is performed for the steady state condition. The steady state condition is reached after a layer by layer construction analysis by a combination of non-linear elastic Duncan and Chang model and elastic-perfectly plastic Mohr-coulomb constitutive model following by an impounding analysis [6]. The nonlinear stress-strain behavior of materials can be represented more accurately by cyclic nonlinear models that follow the actual stress-strain path during cyclic loading. After completion of the static stages, the dynamic analyses were performed. The elasto-plastic model (Mohr-Coulomb) was used for all materials incorporated in of the dam body. These analyses were performed for maximum credible earthquake (MCE) by two acceleration time histories, Khoorgo and Cape Mendocino with the maximum peak ground acceleration of 0.47g with the duration of 41 and 36 sec. The acceleration time histories of the earthquakes are illustrated in Fig. 4 and Fig. 5.

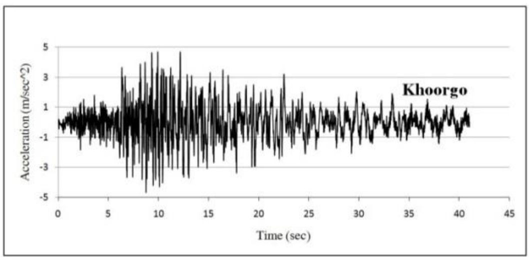

Fig 4. Khorgoo input acceleration time history

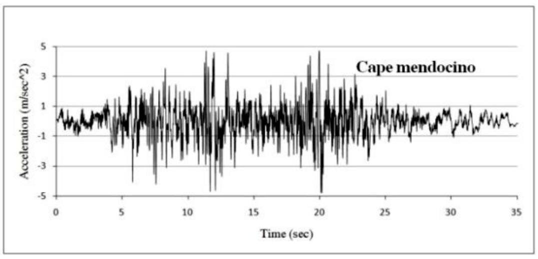

Fig 5. Cape Mendocino input acceleration time history

To dynamic analysis of dam needs to calculate the shear modulus and damping. Many researchers have been proposed equations and curves to estimate the shear modulus and damping based on shear strain [4-7]. In the first step, the analysis was done by elastic model and the shear modulus and damping was assumed as table

4 and 5%, respectively. The maximum shear strain of each element was recorded during the analysis and the shear modulus and damping ratio are updated with respect to the maximum shear strain of each element. In this paper was used the curves proposed by Seed et al. 1984. Fig. 6 shows the shear modulus and damping ratio curves. This procedure was repeated until the shear strain converged to a constant value. In final step, the analysis was conducted by elastic-perfect plastic Mohr-Coulomb constitutive model and final shear modulus and damping to estimate permanent deformation in dam body.

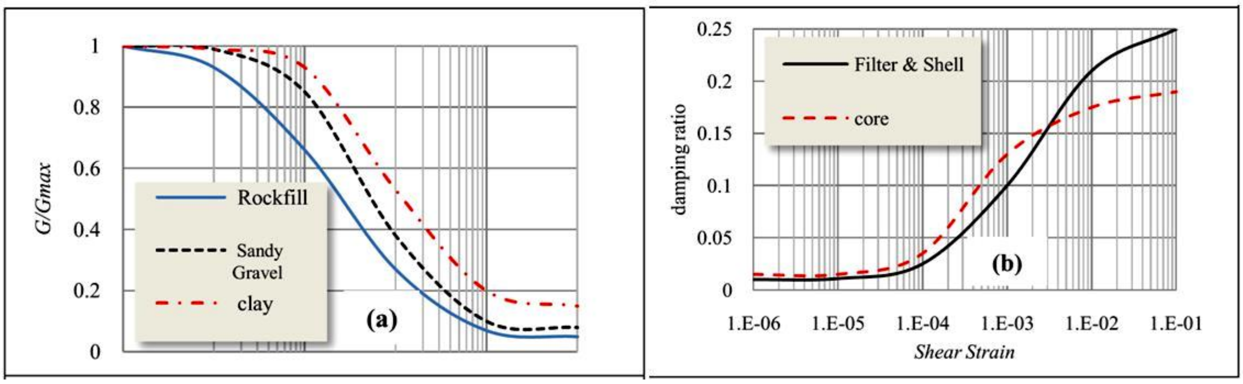

Fig 6. a) shear modulus curves of dam body materials, b) damping ratio curves of dam body materials

Dynamic loading from an earthquake changes the stress states within an embankment, causing permanent deformation. The maximum displacement vector of Khoorgo earthquake is illustrated in Fig. 7 and it is equal to 1.155 m. As shown in Fig. 8 and Fig. 9 the maximum vertical and horizontal displacements at the end of Khoorgo earthquake are 1.1 m and 60 cm, respectively. Fig. 10 shows the dam crest acceleration history. As shown the maximum crest acceleration is 5.96 m/s^2 equal to 1.29 resonance coefficient.

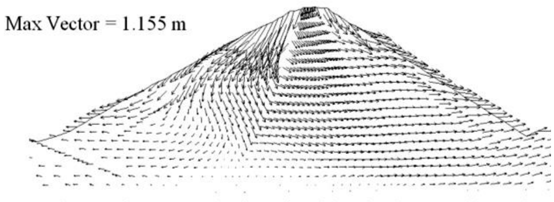

Fig 7. Maximum displacement vector of Khoorgo earthquake

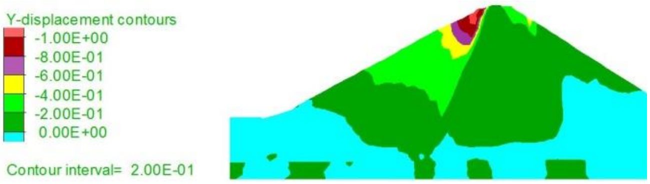

Fig 8. Vertical displacement contours of Khoorgo earthquake

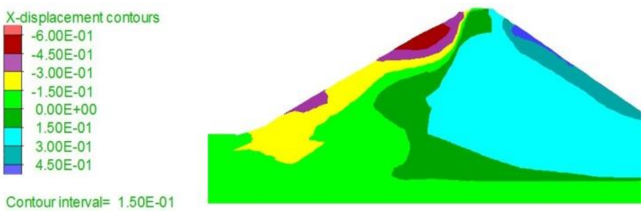

Fig 9. Horizontal displacement contours of Khoorgo earthquake

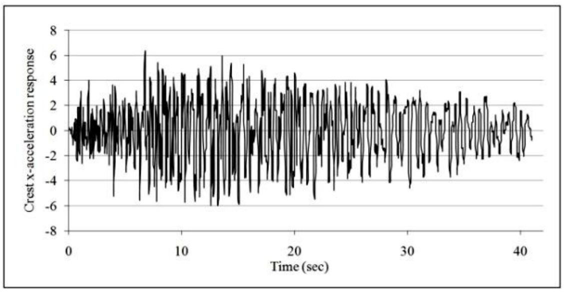

Fig 10. Crest acceleration time history of Khoorgo earthquake

Fig. 11 shows the maximum displacement vector of Cape Mendocino earthquake and it is equal to 0.582 m. As shown in Fig. 8 and Fig. 9 the maximum vertical and horizontal displacements at the end of Cape Mendocino earthquake are 55 cm and 35 cm, respectively. Fig. 14 shows the dam crest acceleration history. As shown the maximum crest acceleration is 5.2 m/s^2 equal to 1.14 resonance coefficient.

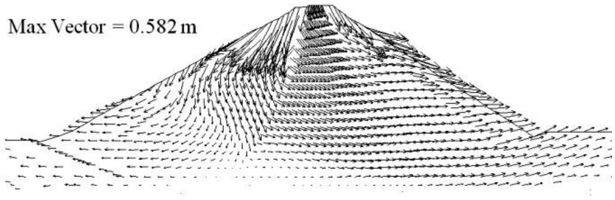

Fig 11. Maximum displacement vector of Khoorgo earthquake

Fig 12. Vertical displacement contours of Khoorgo earthquake

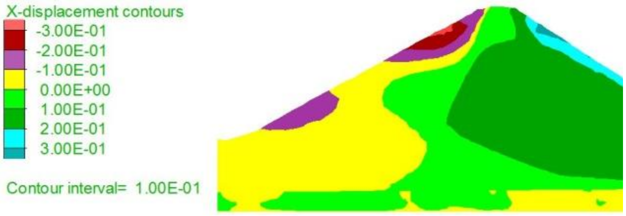

Fig 13. Horizontal displacement contours of Khoorgo earthquake

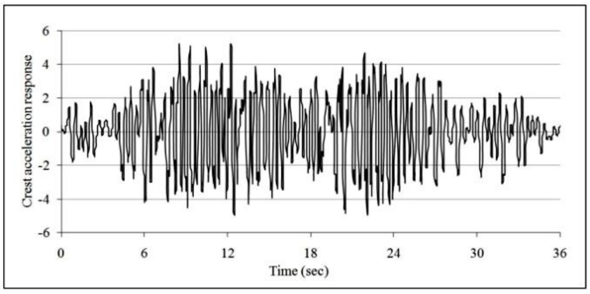

Fig 14. Crest acceleration time history of Khoorgo earthquake

5. CONCLUSIONS

This paper presents nonlinear dynamic behavior of rockfill dams using finite difference method. A simple elastic-perfectly plastic constitutive model with Mohr-Coulomb failure criterion is used to describe the stress-strain response of the soil. The analysis was done by earthquake loading condition for MCL by two acceleration time histories, Cape Mendocino and Khoorgo earthquake with the peak ground acceleration of 0.47g. Based on engineering codes such as ICOLD, USBR and DSOD a general criterion to evaluate damage to embankment dam is the proper continuity and workability of different part of dam such as filters, core, freeboard and etc. after the earthquake. The analysis results show maximum displacement of Khoorgo and Cape Mendocino earthquake are 1.15 m and 58 cm. The upstream and downstream filters are first line of defense against leakage through the dam. The results show the deformation in filters zones is quiet small therefore, the continuity of filters is appropriate. Regarding analysis results, the maximum settlement in dam body is 1 m therefore, the height of freeboard (3m) is adequate. As a conclusion, the non-linear dynamic analysis of Ghoocham dam shows after the earthquake performance of different parts of dam is suitable.

6. REFERENCES

1. Yener Ozkan. M. (1998), "*A review of considerations on seismic safety of embankments and earth and rock-fill dams*," Soil Dynamics and Earthquake Engineering, 17, PP. 439–458.
2. ABOUSEEDA. H. and DAKOULAS. P. (1998), "*Non-linear dynamic earth dam-foundation interaction using a BE-FE method*," Earthquake Engineering & Structural Dynamics, 27, PP. 917—936.
3. Kokusho, T. and Esahsi, Y. (1982), "*Cyclic triaxial testing on sands and coarse materials*," Proc. 10th Int. Conf on Soil Mechanics and Foundation Engrg., 1, PP. 673–676.
4. Seed, H. B., and Idriss, I. M. (1970), "*Soil Moduli and Damping Factors for Dynamic Response Analysis*," Earthquake Engineering Research Center, University of California, Report No. EERC 75-29.
5. Kuhlmeyer, R. L. and Lysmer, J. (1973), "*Finite Element Method Accuracy for Wave Propagation Problems,*" J. Soil Mech. Foundations Div., ASCE, 99(SM5), PP. 421–427
6. Duncan, J.M., and Chang, C. Y (1970), "*Nonlinear analysis of stress and strain in soil. Journal of the Soil Mechanics and Foundation Division*," ASCE, Vol. 96. No. SM5, PP. 1629-1653.
7. Seed, H.B., Wong, T.W., Idriss, I.M. and Tokimatsu, K. (1986), "*Moduli and damping factors for dynamic analyses of cohesionless soils*," J. Geotechnical Engrg., ASCE, 112(11), PP. 1016–1032.