



Effect of the vertical earthquake component on nonlinear behavior of an arch dam having a foundation with discontinuities

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1. Introduction

Concrete arch dams are important infrastructures that are constructed for different purposes, such as irrigation, flood control and power generation. These types of structures may experience more than one earthquake in their service life. Tensile cracking, excessive contraction joint opening, and abutment movement are the main possible seismic failure modes of these types of dams [1]. Failure of these dams can result in heavy human and financial loss. Among the aforementioned failure modes, the abutment movement is the most important one. Hence, it is essential to investigate the stability of dam abutments subjected to seismic loading. The abutment stability of arch dams was examined in some research works. Londe [2] developed a limit equilibrium to assess the abutment stability by considering thrust and uplift forces. The assumptions of this approach mainly focus on the mechanics and displacement simplifications. The stability of the left abutment of Luzzone dam was studied by Sohrabi et al. [3].

Abstract:

In this study, the effects of the vertical component of ground motion on both the safety factor of wedge and response of dam having a foundation with joints are investigated. The Bakhtiari arch dam, with 6 wedges at each of its abutments, is chosen as a case study. The safety factor of dam abutments is obtained by the implementation of time history analysis and Londe limit equilibrium method. The thrust forces are calculated using ABAQUS, a commercial finite element software package. The safety factors of wedges are obtained using the code written within MATLAB. The results indicate that considering the vertical component of the earthquake decreases the safety factors of the wedges considerably. Moreover, the vertical component of ground motion plays a key role in the nonlinear behavior of the dam having a jointed foundation.

They obtained the safety factor time history of a wedge by employing the Londe conventional method in conjunction with the finite element procedure. A comparison investigation between a 3D rock wedge using the finite element and traditional Londe methods was carried out by Mirzabozorg et al. [4]. Their results indicated that the Londe method predicted the wedge displacement to be larger than that of the finite element method. In another study, Mahmoudi et al. [5] studied the influence of foundation nonlinearity on the seismic behavior of an arch dam. They showed that taking foundation nonlinearity into account had a weak influence on the results due to the special shape of the dam. Using Newmark approach, the probable wedge displacements of the Luzzone dam under seismic loading were determined by Mostafaei et al. [6, 7]. Moreover, their results illustrated that dynamic analysis was more conservative than quasi-static analysis. In another study, they indicated that the uplift pressure had a significant influence on the abutment safety factor [8].

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 Table 1 : Possible sliding mode

Case	Definition	Result
1	All of the normal forces on the planes are compressive	Wedge is completely stable
2	Normal force on the first plane is compressive	Wedge detaches from the two other planes
3	Normal force on the second plane is compressive	Wedge detaches from the two other planes
4	Normal force on the third plane is compressive	Wedge detaches from the two other planes
5	Normal force on the first plane is tensile	Wedge detaches from the first plane
6	Normal force on the second plane is tensile	Wedge detaches from the second plane
7	Normal force on the third plane is tensile	Wedge detaches from the third plane
8	All of the normal forces on the planes are tensile	Wedge is unstable

To have a proper understanding of the characteristics of vertical component of earthquake and its effects on the response of structure, related studies have been undertaken by several researchers. Attarnejad and Bagheri studied the influence of vertical component of earthquake on the hydrodynamic pressure [9]. The effects of vertical component of near-field acceleration records were investigated by Naseri and Khalkhali [10]. In their study, three empty, half-full and full reservoirs were considered. They indicated that since the vertical component of ground motion had a key role on the hydrodynamic pressure it should not be neglected in the seismic evaluation. It can be inferred that in addition to horizontal components of earthquake, the seismic behavior of dam is very sensitive to the vertical component of earthquake.

To the authors' best knowledge, there is no study in the literature that investigates the effects of the vertical component of the earthquake on the stability of arch dam abutments. In this sense, this study aims to examine the influence of vertical component of ground motion on the safety factor of arch dam abutments against instability and nonlinear behavior of a dam having a foundation with discontinuities. The Bakhtiari arch dam with a height of 325m, located in Iran, is chosen as an application. Finite element models of the dam with integrated and jointed foundations are established first. To model the nonlinear behavior of concrete material, concrete damaged plasticity (CDP) is considered in the nonlinear analyses. For the seismic investigation of the dam abutment, 10 acceleration records are chosen. Afterwards, linear and nonlinear time history analyses of the dam are performed for two cases consisting of models with and without vertical components of the earthquakes and the results are compared with each other.

2. Seismic stability of the wedge

The wedge, which is shown in Fig. 1, is defined by intersection of three probable sliding planes, which are named as sub-horizontal (P1) and sub-vertical planes (P2) and the grout curtain (P3), respectively.



Fig. 1 : Dam-reservoir-foundation, wedge and its supporting planes.

Considering the assumption of the Londe method [2], the resultant of the applied forces can be written as follows:

$$\mathbf{F}_{\text{Res}}^{\text{W}} = \mathbf{F}_{\text{W}}^{\text{W}} + \mathbf{F}_{\text{Up}}^{\text{W}} + \mathbf{F}_{\text{EQ}}^{\text{W}} + \mathbf{F}_{\text{TH}}^{\text{D}}$$
(1)

where F_w^w , F_{Up}^w , F_{Eq}^w , and F_{TH}^D are the weight of the wedge, uplift force, seismic inertial force and the thrust force, respectively, that are obtained by using a 3-D finite element dam-foundation-reservoir model.

Using equilibrium equations, the three corresponding normal forces on the planes, which are named as N_1 , N_2 , and N_3 are obtained. Subsequently, by taking the Londe assumptions that the planes can only be in compression, eight different sliding modes that are listed in Table 1 may occur.

For the case numbers 2, 3, and 4, the tensile force is replaced by a friction force F acting along the intersection line of the two planes. By using three translational equilibrium equations, the two normal forces and the single friction force are obtained. For example, if the third plane is in tension, the safety factor of the wedge can be calculated as follows:

$$SF = \frac{N_{1} \tan(\varphi_{1}) + c_{1}A_{1} + N_{2} \tan(\varphi_{2}) + c_{2}A_{2}}{F_{12}}$$
(2)

where A, φ , and c are area, friction angle, and cohesion of the planes, respectively. Moreover, F_{12} stands for the friction force acting at the intersection of planes 1 and 2. For the case numbers 5, 6, and 7, instead of the two tensile reactions, two normal components of a friction force acting on the third plane are considered. The two aforementioned forces along with a normal reaction force on the third plane can be determined by solving the equilibrium equations. The resultant of the friction force is determined to be F. For example, if N_2 and N_3 are tensile, the safety factor against sliding is obtained as follows:

$$SF = \frac{N_{1} \tan(\varphi_{1}) + c_{1}A_{1}}{F_{1}}$$
(3)

3. Description of the case study

The Bakhtiari dam is a 325m high concrete double curvature dam. The dam thickness varies from 5 m at the crest to 54 m at the base [11]. Moreover, the normal water level is 320m. Two separate finite element models (dam with integrated foundation and dam with a jointed foundation) are developed by using the eight-node brick

isoparametric element (C3D8R). Besides, the reservoir water is modeled by the acoustic element AC3D8R. A transmitting boundary condition is applied at the far-ends of the reservoir. For modeling the wave radiation, the infinite elements are employed at the foundation boundaries. Fig. 2 presents the finite element models utilized in this research.



Fig. 2 : The finite element model of the Bakhtiari dam. a) Dam with integrated foundation, b) dam with foundation having joints.

3.1 Material properties

The material properties of the concrete are taken to be: mass density $\rho_c = 2400 \ kg/m^3$, Elastic modulus $E_c = 24GPa$, and Poisson's ratio $v_c = 0.18$. Since the tension and compression stress-strain responses of the test samples are not available, mathematical models are used for this purpose [12]. The Kent and Park model is considered for the stain-stress behavior of concrete. Fig. 3 shows the constitutive relations under tensile and compressive loadings.



Fig. 3 : The Stress-strain model for the tensile and compressive loadings of concrete

The concrete damage plasticity is employed for modeling the nonlinear behavior of concrete. In this approach, the tensile cracking and compressive crushing of concrete are taken into account by using two damage variables, d_t (for tensile damage) and d_c (for compressive damage). The aforementioned variables are assumed to be functions of inelastic strain ratio that changes from zero to one showing the undamaged material and total loss of strength, respectively. The stress-strain relations under uniaxial tension and compression can be written as follows:

$$\sigma_{t} = (1 - d_{t}) E_{0} \left(\varepsilon_{t} - \varepsilon_{t}^{\Box pl} \right)$$

$$\sigma_{c} = (1 - d_{c}) E_{0} \left(\varepsilon_{c} - \varepsilon_{c}^{\Box pl} \right)$$

$$(4)$$

where E_0 is the initial (undamaged) modulus of elasticity, and σ , ε and $\varepsilon^{\sim pl}$ are the stress, strain, and inelastic strain of concrete, respectively, in tension (t) and compression (c).

In addition, the mass density, modulus of elasticity, and the Poisson's ratio of the foundation are assumed to be $\rho_f = 2600 \, kg/m^3$, $E_f = 12GPa$, and $v_f = 0.25$, respectively. The mass density and Bulk modulus of water are taken to be $\rho_w = 1000 \, kg/m^3$ and $K_w = 2.2GPa$, respectively. It is noteworthy that the damping ratio of the system is assumed to be 5% of the critical damping.

3.2 Geometric nonlinearity

The contact force between two surfaces has tangential and normal components. The stresses transmitted across the interfaces are related to each other by the Coulomb friction model. The aforementioned model is exhibited as follows: (5)

$$\tau_u = \mu \sigma$$

where τ_u is the ultimate shear stress, σ , is the normal stress and μ stands for the coefficient of friction. Moreover, hard contact condition is considered for modeling the normal behavior of contact.

3.3 Definition of the Wedges

As mentioned before, the wedges are defined by three discontinuity planes. Table 2 presents the unit normal vectors of these discontinuities at each abutment.

Six wedges have been selected to investigate the stability analysis for each abutment of the dam as presented in Fig. 4. It is noteworthy that the elevation of the horizontal plane for wedges is different.



Table 2 : The unit normal vector of the discontinuities planes [11]

Fig. 4: Geometry of the abutment wedges; (a) WL1 and WR1 at elevation 305m (b) WL2 and WR2 at elevation 275m (c) WL3 and WR3 at elevation 225m (d) WL4 and WR4 at elevation 185m (e) WL5 and WR5 at elevation 145m (f) WL6 and WR6 at elevation 85m

Wedge		Sub-H	Sub-Horizontal Plane (P1)					Sub-Vertical Plane (P2)				Plane	(P3)	Elevation	
		Uplift Force (MN)	C (MPa)	Φ (°)	Area (m ²)	Uplift Force (MN)	C (MPa)	Φ (°)	Area (m ²)	Uplift Force (MN)	C (MPa)	(°) Ф	Area (m ²)	From-to (m)	Wedge volume (m ³)
	WL1	58	0.3	45.3	3,800	253	0	32	2,273	75	0.3	45	275	305-325	62,258
k	WL2	421	0.3	44.5	4,991	1117	0.1	42	5,934	394	0.3	45	902	275-325	194,096
oan	WL3	1168	0.4	43.5	6,591	4744	0.1	42	13,754	1812	0.3	45	2451	225-325	424,886
ffl	WL4	2576	0.5	43.1	10,112	10893	0.1	42	22,655	4033	0.3	45	4103	185-325	744,914
Ľ	WL5	5724	0.5	42.7	14,784	21258	0.1	42	34,561	7460	0.3	45	6069	145-325	1,251,255
	WL6	11432	0.6	42	23,514	39453	0.1	42	54,974	14042	0.3	45	9720	85-325	2,396,494
	WR1	250	0.3	45.3	9,003	343	0	32	2,987	105	0.3	45	339	305-325	141,808
nk	WR2	1209	0.3	44.6	12,445	1447	0.1	42	7,834	623	0.3	45	1304	275-325	464,062
ba	WR3	5128	0.4	44	18,657	5543	0.1	42	17,160	3260	0.3	45	4096	225-325	1,089,998
ght	WR4	9857	0.4	43.4	24,349	11503	0.1	42	26,093	7724	0.3	45	7354	185-325	1,949,327
Riį	WR5	17212	0.6	41.4	30,584	21397	0.1	42	37,036	15002	0.3	45	11476	145-325	3,042,590
	WR6	37687	0.5	42.4	46,244	42357	0.1	42	60,159	29315	0.3	45	19407	85-325	5,317,911

Table 3 : Geometrical and mechanical characteristics of the wedges [11]

Table 3 presents the important characteristics of these planes [11]. Moreover, the uplift pressure on the grout curtain is obtained based on the area of the plane and

normal water level. The uplift force on the two other planes is assumed to be 33% of the total uplift force that is calculated according to the two aforementioned parameters.

No	Forthquaka	Station	Voor	1	Unscaled PGA (g)				
NO.	Eartiquake	Station	Tear	Stream	Cross-stream	Vertical			
EQ1	San Fernando	Pasadena - Old Seismo Lab	1971	0.095	0.205	0.089			
EQ2	Morgan Hill	UCSC Lic Observatory	1984	0.039	0.076	0.031			
EQ3	N. Palm Springs	Anza - Red Mountain	1986	0.098	0.119	0.066			
EQ4	N. Palm Springs	Santa Rosa Mountain	1986	0.114	0.086	0.049			
EQ5	Northridge-01	LA - Wonderland Ave	1994	0.103	0.159	0.105			
EQ6	Northridge-01	Vasquez Rocks Park	1994	0.151	0.139	0.091			
EQ7	Tottori Japan	OKYH07	2000	0.128	0.185	0.125			
EQ8	Iwate	AKTH05	2008	0.066	0.085	0.039			
EQ9	Iwate	MYGH04	2008	0.152	0.227	0.127			
EQ10	San Simeon CA	Diablo Canyon Power Plant	2003	0.047	0.034	0.021			

Table 4 : Characteristics of the selected earthquakes.

3.4 Time-history analysis

To study the seismic stability of the dam abutments, 10 earthquake ground motions are selected herein, as proposed by the authors in a parallel work [13]. Characteristics of the selected earthquake records are listed in Table 4.

The selected ground motions are scaled based on ASCE/SEI (2016). Fig. 5 presents the target response spectrum along with the average response spectrum of the selected records before and after scaling. It should be noted that the fundamental period of the system and the scale factor are 1.428 sec and 2.89, respectively.



Fig. 5 : Scaling of the average response spectrum.

4. Results and discussion

In this section, influence of the vertical component of the earthquakes on the stability of the abutment, and nonlinear dynamic response of the dam are investigated. It is worth stating that the analysis is performed using ABAQUS to obtain thrust forces that are applied on the wedges. Afterwards, safety factor of the wedges is calculated by inclusion of all of the existing forces (thrust, weight, uplift pressure, and inertia forces) by a procedure developed in MATLAB.

4.1 Analysis using the Londe method

Through the summation of nodal forces at the interface of the dam and the wedges, the applied thrust forces on the wedge can be obtained. Fig. 6 shows the time histories of

the thrust forces applied on WL6 wedge at the left abutment under EQ9.

Fig. 7 shows the safety factor of WL6 wedge at the left abutment under EQ9. It can be seen from Fig. 6 that the safety factor of the wedge is less than one for some periods of time, and the wedge becomes unstable. Moreover, by considering the vertical component of the earthquake, the safety factor of the wedge decreases.

To have a proper understanding of the vertical component effects on the behavior of the dam, the maximum principal stress contours of the dam at the moment that the abutment safety factor reaches its minimum values are presented in Fig. 8 for the aforementioned earthquake. It can be seen that maximum of the principal stress belongs to the case when the vertical component of the earthquake is taken into account. It is noteworthy that the crack propagation initiates from the base of the dam.

Table 5 presents the minimum of the safety factor of wedges under various earthquakes including the vertical component. The tabulated results illustrate that the location of the bed rock is of importance in the abutments stability of the arch dam. Furthermore, the safety factors of the wedges at the right abutment are higher than those of the left abutment. This is due to the fact that the wedges at the right abutment are heavier than the others. It is noteworthy that the WL6 wedge at the left abutment has the least safety factor.

Besides, the minimum safety factor of the wedges under various earthquakes without the vertical component are listed in

Table 6.

On the basis of the listed results in Tables 5-6, effect of the vertical component of the earthquakes on the safety factor of wedges is shown in Fig. 9. As expected, eliminating the vertical component of the ground motion increases the safety factor. This is due to the fact that in absence of the vertical component, the normal stresses can increase along the contact planes and result in more stability.



Fig. 6 : Time histories of the thrust force of WL6 at the left abutment under EQ9. a) Stream direction; b) Cross-stream direction; c) Vertical direction.



Fig. 7 : The time histories of safety factors of WL6 at the left abutment under EQ9.



Fig. 8 : Stress contours of the dam under EQ9. a) With the vertical component of the earthquake, b) without the vertical component of the earthquake.

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Wedge		EQ1	EQ2	EQ3	EQ4	EQ5	EQ6	EQ7	EQ8	EQ9	EQ10	Mean
	WL1	2.68	4.65	4.00	4.68	2.72	2.77	2.43	4.32	2.14	6.06	3.65
	WL2	2.00	3.59	2.65	3.27	2.19	1.68	2.00	3.24	1.34	EQ10 Mean 6.06 3.65 4.69 2.66 2.25 1.67 1.96 1.49 1.86* 1.42* 1.96 1.47 13.20 6.19 11.94 4.31 7.57 3.49 5.08 2.25 2.09 1.74 1.86 1.42	2.66
L oft honk	WL3	1.42	2.11	1.76	2.03	1.43	1.28	1.32	1.97	1.11	2.25	1.67
Left ballk	WL4	1.30	1.90	1.56	1.80	1.32	1.11	1.19	1.77	1.01	1.96	1.49
	WL5	1.26^{*}	1.85^{*}	1.48^{*}	1.71^{*}	1.25^{*}	1.02^{*}	1.14^{*}	1.70^{*}	0.94	1.86^{*}	1.42^{*}
	WL6	1.31	1.95	1.52	1.77	1.26	1.03	1.19	1.79	0.93^{*}	1.96	1.47
	WR1	3.86	9.31	4.35	7.05	4.85	3.11	3.98	9.39	2.83	13.20	6.19
	WR2	2.45	5.81	3.06	4.18	3.10	2.10	2.62	5.87	1.95	11.94	4.31
Diaht hank	WR3	2.17	5.41	2.80	3.47	2.63	1.97	2.28	4.87	1.78	7.57	3.49
Right bank	WR4	1.86	4.28	2.47	2.77	2.18	1.71	1.96	3.70	1.60	5.08	2.76
	WR5	1.62	3.32	2.24	2.29	1.90	1.58	1.79	2.87	1.51	3.36	2.25
	WR6	1.40	2.49	1.96	1.77	1.52	1.28	1.47	2.07	1.31	2.09	1.74
Minimum		1.26	1.85	1.48	1.71	1.25	1.02	1.14	1.70	0.93	1.86	1.42
*The critical	l wedge											

Table 6 : Minimum safety factor of the wedges under various earthquakes without the vertical component.

Wedge	e	EQ1	EQ2	EQ3	EQ4	EQ5	EQ6	EQ7	EQ8	EQ9	EQ10	Mean
	WL1	2.78	4.79	4.78	5.49	3.16	2.79	3.00	4.36	2.57	6.83	4.06
	WL2	2.05	3.67	2.91	3.93	2.54	2.02	2.11	3.46	2.00	5.15	2.98
Laft hank	WL3	1.47	2.22	2.04	2.30	1.68	1.35	1.44	2.11	1.50	2.52	1.86
Lett Dalik	WL4	1.34	1.99	1.73	2.06	1.52	1.24	1.29	1.90	1.41	2.23	1.67
	WL5	1.29	1.89	1.67	1.98	1.41	1.19	1.27	1.81	1.35	2.14	1.59
	WL6	1.34	1.96	1.75	2.03	1.42	1.22	1.37	1.83	1.28	2.25	1.64
	WR1	3.92	9.59	5.52	8.71	6.20	3.49	4.22	9.96	3.61	16.82	7.20
	WR2	2.56	6.00	4.17	5.74	4.36	2.61	2.67	6.74	2.76	14.44	5.21
Diaht hank	WR3	2.29	5.48	3.91	5.11	3.12	2.36	2.37	5.90	2.73	9.50	4.28
Right bank	WR4	1.98	4.35	3.52	4.33	2.57	2.09	2.21	4.90	2.51	6.06	3.45
	WR5	1.73	3.53	3.17	3.67	2.16	1.86	1.99	3.64	2.30	3.85	2.79
	WR6	1.52	2.88	2.89	3.17	1.86	1.64	1.84	2.71	2.13	2.62	2.33
Minimum		1.29	1.89	1.67	1.98	1.41	1.19	1.27	1.81	1.28	2.14	1.59
*The critical	wedge											



Fig. 9 : Comparison of the minimum safety factor of wedges with and without presence of the vertical component of the earthquakes

It should be mentioned that the averages of the minimum safety factor of wedges with and without vertical component of ground motions are 1.42 and 1.57, respectively. As the vertical component of the earthquake is considered, the average of the minimum safety factor is reduced by 9.45%.

4.2 Finite element analysis including joints

The results shown in Table 5 indicate that WL6 at the left abutment has a minimum safety factor for the EQ9 earthquake.



Fig. 10 : Displacement contours of the dam under EQ9



Fig. 11 : Tensile damage contours of the dam under EQ9

The joints of the aforementioned wedge are modeled in the foundation, and the nonlinear response of the arch dam is achieved for both the integrated and jointed foundations as well as the cases of with and without vertical component of the earthquakes. Influence of the vertical component on the displacement contours of the dam is shown in Fig. 11. It should be noted that the displacement contours are shown at the moment that the safety factor of the wedge calculated by the Londe method attains its minimum. The obtained results show that considering the joints of WL6 wedge causes the dam leans on the left abutment. The maximum displacements of the dam with integrated and jointed foundations are 0.277m and 0.834m, respectively. Moreover, when the vertical component of the earthquake is neglected, the corresponding values are 0.202 and 0.380. By comparison between the aforementioned results, it is revealed that the effects of vertical components of ground motion on the displacements of the dam with a foundation having joint is more considerable than that with integrated foundation.

In addition, effects of the vertical component of ground motion on the tensile damage contours of the dam with integrated and jointed foundations are presented in Fig. 11. The higher tensile damages observed in the dam body are due to considering the wedges, while the assumption of integrated foundation is the main cause of marginal damage spread near the heel of the dam. In cases that wedge joints are modeled, the tensile cracks initiate from the point of contact of the wedge with the dam and continue to the middle of the dam crest. Moreover, in the contact surface between the dam and its foundation, the cracks spread from upstream to downstream. Moreover, by comparing the results obtained by bearing in mind the joints in the foundation and those achieved based on the integrated foundation, it can be inferred that the influence of considering vertical component of the earthquake on the tensile damage of the dam with jointed foundation is more significant than that with integrated foundation.

5. Conclusions

The current work presented a comprehensive investigation into the safety factor of dam abutment and seismic response of dam having a foundation with joints by taking into account the effects of vertical components of earthquake. To this end, the Bakhtiari arch dam, a doubly curved arch dam, was selected as a case study and investigated using time history analysis. Presence of six wedges at each abutment of the dam was assumed for the analyses. The 3-D finite element models of the dam with integrated foundation and foundation having discontinuities were developed. Safety factor of dam abutments against sliding instability was obtained on the basis of the Londe method along with time history analysis, and the critical wedge was identified. It was found that as the vertical component of earthquake is taken into account, the average of safety factor decreases by 11.7%. Afterwards, joints of the distinguished wedge were modeled and effects of vertical component of earthquake on the response of the dam were investigated. The obtained results indicated that taking the wedge joints of the foundation into account can create tensile damages in the dam body. As a result, considering these discontinuities in the foundation at the unstable wedges is a necessary issue. Moreover, maximum displacement of dam was toward the abutment containing the wedge. Besides, considering the vertical component of earthquake can create instability in the dam abutment and result in more extensive damages in the dam body with a jointed foundation compared with the case of the dam with an integrated foundation.

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