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Seismic analysis of arch dams subjected to in-phase and anti-phase ground motions

Mehmet Akköse [a,*], Ali Aydın Dumanoğlu [b], Alemdar Bayraktar [c]

[a] *Department of Civil Engineering, Karadeniz Technical University, 61080 Trabzon, Turkey*
[b] *Department of Civil Engineering, Canik Başarı University, 55080 Samsun, Turkey*
[c] *Department of Civil Engineering, Karadeniz Technical University, 61080 Trabzon, Turkey*

A B S T R A C T

In this study, the response of arch dams is obtained for in-phase and anti-phase ground motions when there is no water in the reservoir. The material of the dam is considered to be linearly elastic, homogenous and isotropic. The foundation and banks of the dam, which are usually of hard rock, are assumed to be rigid. The S16E component of San Fernando Earthquake, February 9, 1971, has been used in the calculations. The response of arch dams determined for anti-phase dynamic effects is compared with that of in-phase (uniform) dynamic effects.

A R T I C L E I N F O

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1. Introduction

A dynamic ground motion is defined as in-phase (uniform) if the motion propagates in the same direction with the same phase and infinite velocity. However, if the motion propagates in the same direction with the opposite phase and infinite velocity, the case of the motion is defined as anti-phase (Adanur, 1998). The anti-phase ground motion may occur due to the fact that the directions of the earthquake waves are changeable. This ground motion is particularly effective in the structures, which have large structure-foundation interface or long spans. For example, bridges, long pipelines, nuclear power plants and dams. The behaviour of arch dams subjected to the anti-phase dynamic effects is the aim of this study.

In the anti-phase ground motion, quasi-static displacements occur in addition to dynamic displacements. Inertia forces cause dynamic displacements whereas relative movements of the support points according to each other at the structure-foundation interface cause the quasi-static displacements (Dumanoglu and Severn, 1984; Bayraktar and Dumanoglu, 1998). The determination of the quasi-static displacements requires the deformed shape of the structure to be calculated for a given unit displacement of each support point of structure-foundation interface. The quasi-static displacements are time-dependent and cause stresses to be added to the dynamic stresses. There are no quasi-static effects for in-phase ground motion due to the rigid body motion.

Different earthquake waves have been recorded in soil surface of each abutment of Ambiesta dam during same earthquake (Calciati, 1979). This shows that the earthquake record might be different at different points. This situation also indicates that the propagation of the earthquake waves can be different direction for both banks. Arch dams conduct reactions induced by water pressures on its surface to the foundation rock at their banks by arc effects. Therefore, a strong ground motion can cause relative movement of the abutments of arch dams. This situation can disturb the stability of the arch dams and cause their collapse.

In this study, the response of a selected arch dam is investigated by considering relative movements of their abutments according to each other when there is no water in the reservoir. The foundation and banks of the dam, which are usually of hard rock, are assumed to be

* *Corresponding author.* Tel.: +90-462-3772628 ; Fax: +90-462-3772606 ; E-mail address: akkose@ktu.edu.tr (M. Akköse)
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rigid. The possible amplitude variations of the earthquake waves are neglected. In the other words, it is assumed that the earthquake waves propagate with the same amplitude. The response of the dam determined for anti-phase dynamic effects is compared with that of in-phase (uniform).

2. Equations of Motion

The equation of motion for dynamic effects is given by the equation,

$$M\ddot{v} + C\dot{v} + Kv = F \ , \tag{1}$$

where M, C and $K$are mass, damping and stiffness matrices, respectively. $\ddot{v}$, $\dot{v}$ and v are total acceleration, velocity and displacement vectors, respectively. F is external load vector. The degrees of freedom of the system may be separated into two groups. First group is associated with degrees of freedom of the structure-foundation interface. Second group is related to degrees of freedom of the structure. The former will be denoted as the vector v_g and the latter as v_r. Here, suffix g denotes "ground degrees of freedom" and suffix r denotes "response degrees of freedom". According to this explanation, Eq. (1) can be expressed as follows;

$$\begin{bmatrix} M_{rr} & M_{rg} \\ M_{gr} & M_{gg} \end{bmatrix} \begin{Bmatrix} \ddot{v}_r \\ \ddot{v}_g \end{Bmatrix} + \begin{bmatrix} C_{rr} & C_{rg} \\ C_{gr} & C_{gg} \end{bmatrix} \begin{Bmatrix} \dot{v}_r \\ \dot{v}_g \end{Bmatrix} + \begin{bmatrix} K_{rr} & K_{rg} \\ K_{gr} & K_{gg} \end{bmatrix} \begin{Bmatrix} v_r \\ v_g \end{Bmatrix} = \begin{Bmatrix} 0 \\ 0 \end{Bmatrix} . \tag{2}$$

In the absence of external forces applied directly to the structure, F becomes zero-vector as shown in Eq. (2). It is also possible to separate the total displacement vectors as quasi-static, v_s, and dynamic, v_d, into two groups which are shown as;

$$\begin{Bmatrix} v_r \\ v_g \end{Bmatrix} = \begin{Bmatrix} v_{sr} \\ v_{sg} \end{Bmatrix} + \begin{Bmatrix} v_{dr} \\ v_{dg} \end{Bmatrix} . \tag{3}$$

In the Eq. (3), it is clear that v_{dg} is zero. So, v_{sg} is equal to v_g. Substituting Eq. (3) into Eq. (2), the equation of motion of the dynamic component of the response degrees of freedom can be written as;

$$M_{rr}\ddot{v}_{dr} + C_{rr}\dot{v}_{dr} + K_{rr}v_{dr} = -M_{rr}R_{rg}v_{sg} \ , \tag{4}$$

where R_{rg} contains r vectors (ground displacement shape vector) that describe the displaced shape of the structure when a unit displacement is given to the single ground degrees of freedom while all after ground degrees of freedom are held fixed. r vector owing to in-phase motion is obtained for positive unit displacement assigned to the ground degrees of freedom where in-phase motion effecting. However, r vector owing to anti-phase motion, is obtained for positive and negative unit displacements assigned to right and left bank ground degrees of freedom, respectively. From Eq. (4), dynamic components of the total displacement vectors are calculated by mode superposition technique as

$$v_{dr} = \textstyle\sum_i \phi_i Y_i(t) \ , \tag{5}$$

where i is mode number, ϕ_i is ith mode vector and $Y_i(t)$ is the time-dependent modal amplitude of the response. Quasi-static components of the total displacement vectors are obtained as

$$v_{sr} = r_1 v_{1g}(\tau_1, t) + r_2 v_{2g}(\tau_2, t) + \cdots \ , \tag{6}$$

where v_{ig} is referred to displacements of the acceleration records of ground motion. This is calculated from twice integration of the acceleration records. τ_i is referred to arrival time from a certain reference point of ground motion to ith support point. Total displacements, v_r, are obtained by summing up quasi-static and dynamic displacements as follows;

$$v_r = v_{sr} + v_{dr} \ . \tag{7}$$

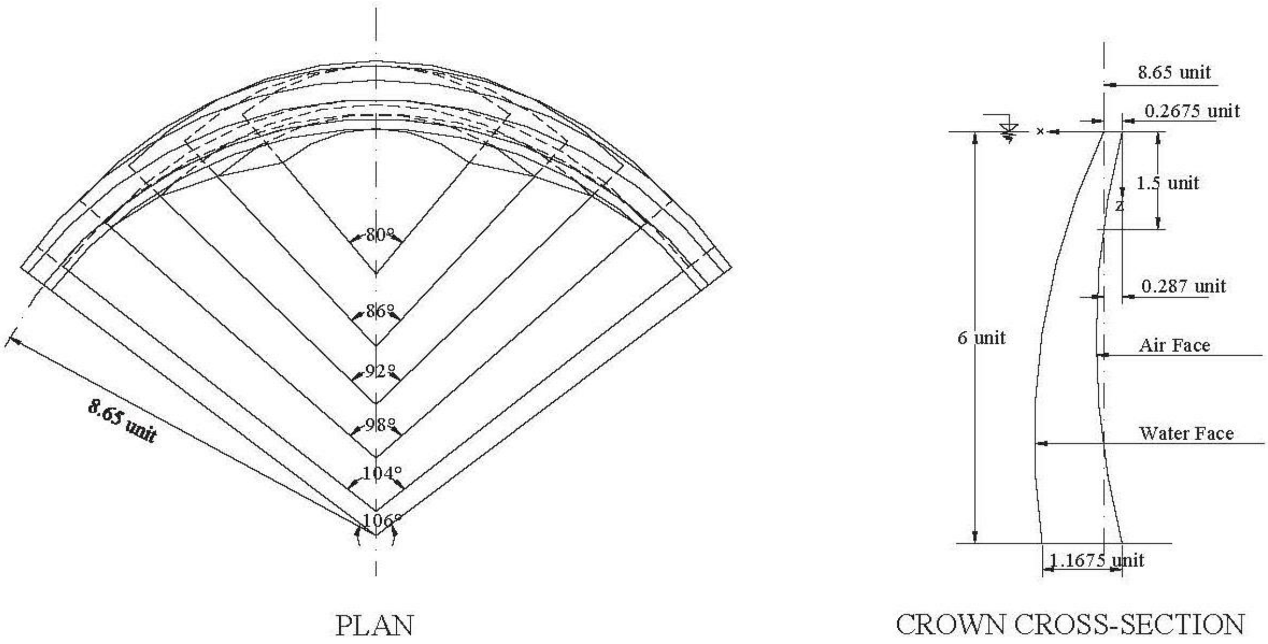
3. Numerical Example

In this study, arch dam Type 5, which was suggested in the symposium on Arch Dams (Arch Dams, 1968), is selected as an example for anti-phase dynamic effects. This dam is a standard double curved structure. The view in plan and the vertical crown cross-section of the idealised arch dam is shown in Fig. 1. The dimensions of the arch dam are in unit. The height of the dam is chosen as 120 m to obtain realistic results. The other dimensions of the dam are determined according to this size. An axonometric view of the finite element mesh of the whole dam is shown in Fig. 2. In the finite element mesh of the arch dam, 164, eight-node, 3D solid elements are used.

The material of arch dam is assumed to be linearly elastic, homogenous and isotropic. The elasticity modulus, mass density and Poisson's ratio of the dam are taken as $2\text{x}10^{10}$ N/m^2, 2446.48 N/m^3 and 0.15, respectively. The solution time step is 0.001. The damping ratio is chosen as %5. The program MULSAP (Dumanoglu, 1988) is employed in the response calculations. Pacoima Dam record S16E component recorded during the San Fernando Earthquake, February 9, 1971 shown in Fig. 3 is chosen for the ground motion. The component considered is applied in the upstream-downstream direction for both in-phase and anti-phase situation as shown in Fig. 4.

The response of the arch dam chosen for both situations is examined. The absolute maximum nodal displacements are obtained at arch sections. Fig. 5 shows reference lines of the arch sections. Figs. 6 and 7 give the displacements at the arch sections. The displacements calculated in the middle plane of the dam for anti-phase dynamic effect are considerably greater than those of in-phase dynamic effect.

The absolute maximum stresses obtained on the direction *x-x*, *y-y* and *z-z* at section I-I shown in Fig. 5 are given in Table 1 for upstream face and Table 2 for downstream face. These tables contain the response of the dam for both in-phase and anti-phase dynamic effects. The stresses are given at the centroid of the elements. The stresses calculated for anti-phase dynamic effect are different from those of in-phase. In particular, the stresses on the direction *x-x* have its maximum value around the axis of symmetry of the dam.



Level	0	1	2	3	4	5	6
Radius (units)	3.67	5.29	6.64	7.72	8.54	8.80	8.50
Central Angle	80°	86°	92°	98°	104°	106°	106°

Fig. 1. The view in plan and the vertical crown cross-section of arch dam Type 5.

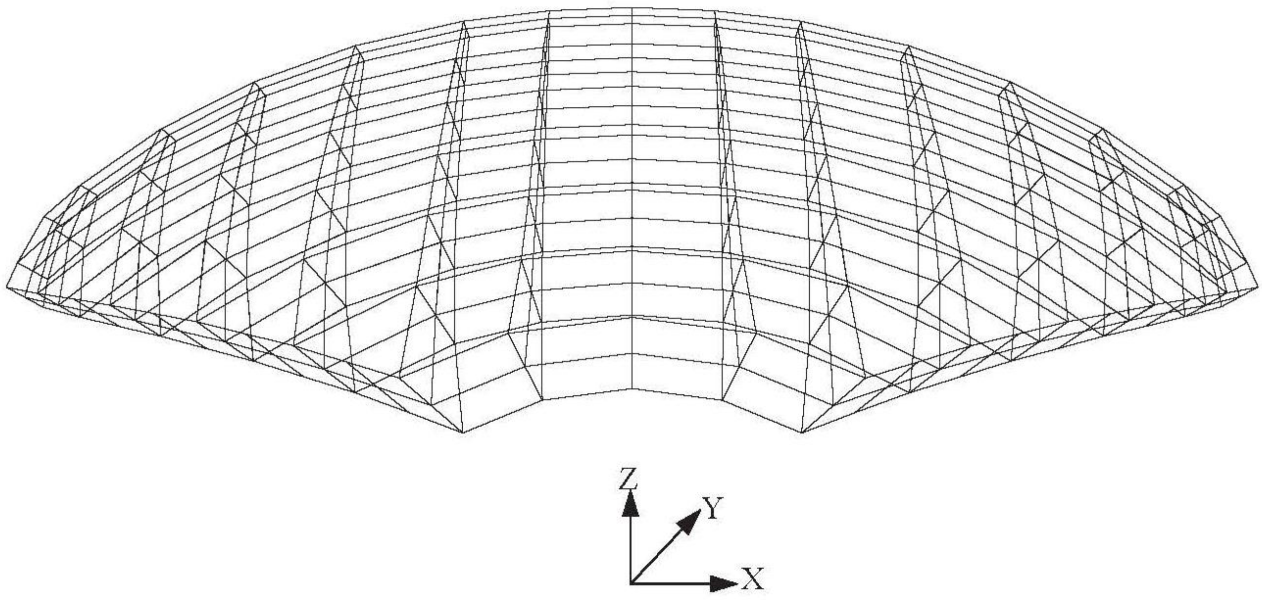


Fig. 2. The axonometric view of 3D finite element mesh of arch dam Type 5.

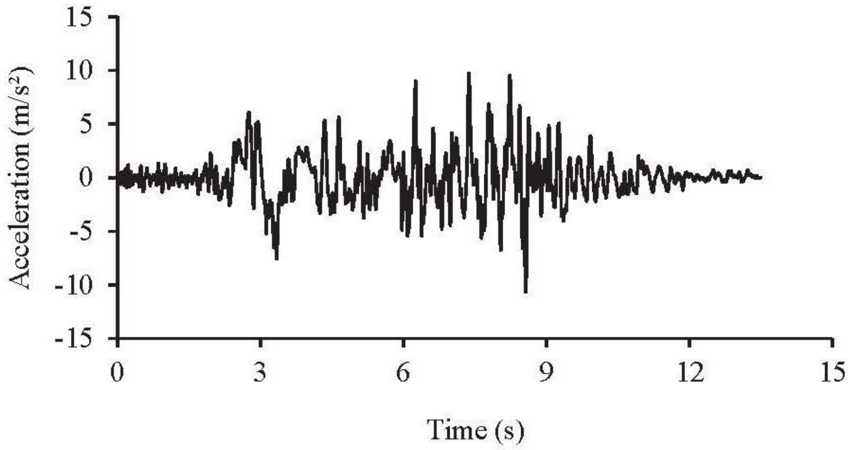


Fig. 3. S16E San Fernando Earthquake, February 9, 1971 (Pacoima dam record).

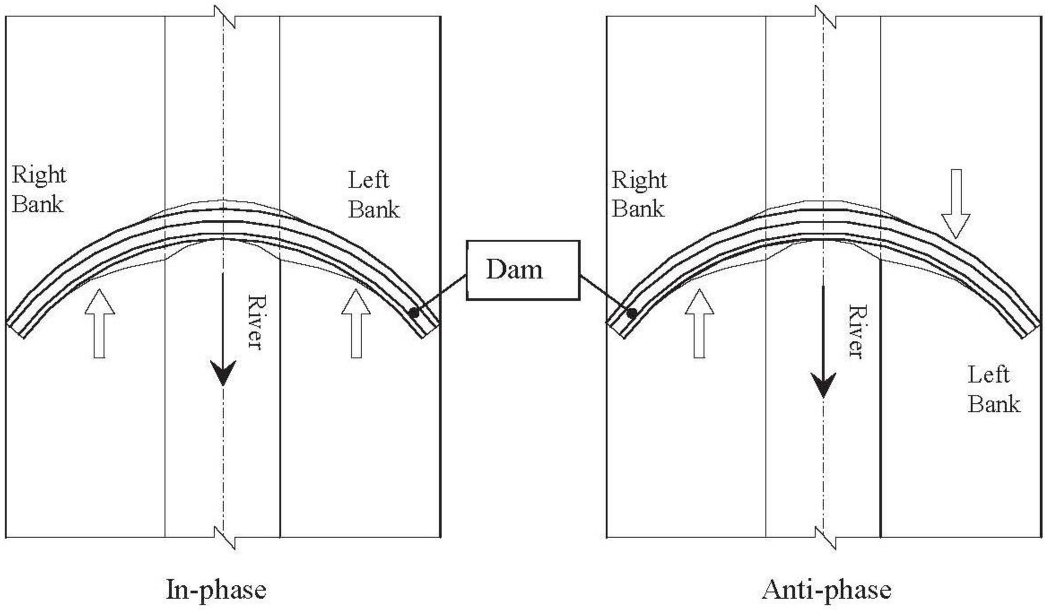


Fig. 4. Arch dam Type 5 subjected to in-phase and anti-phase ground motions in plan.

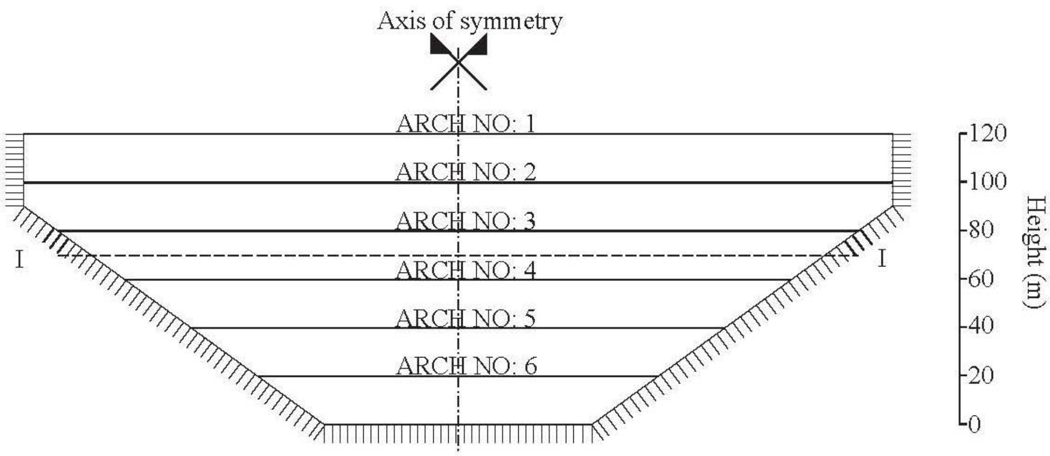


Fig. 5. The arch reference lines of arch dam Type 5.

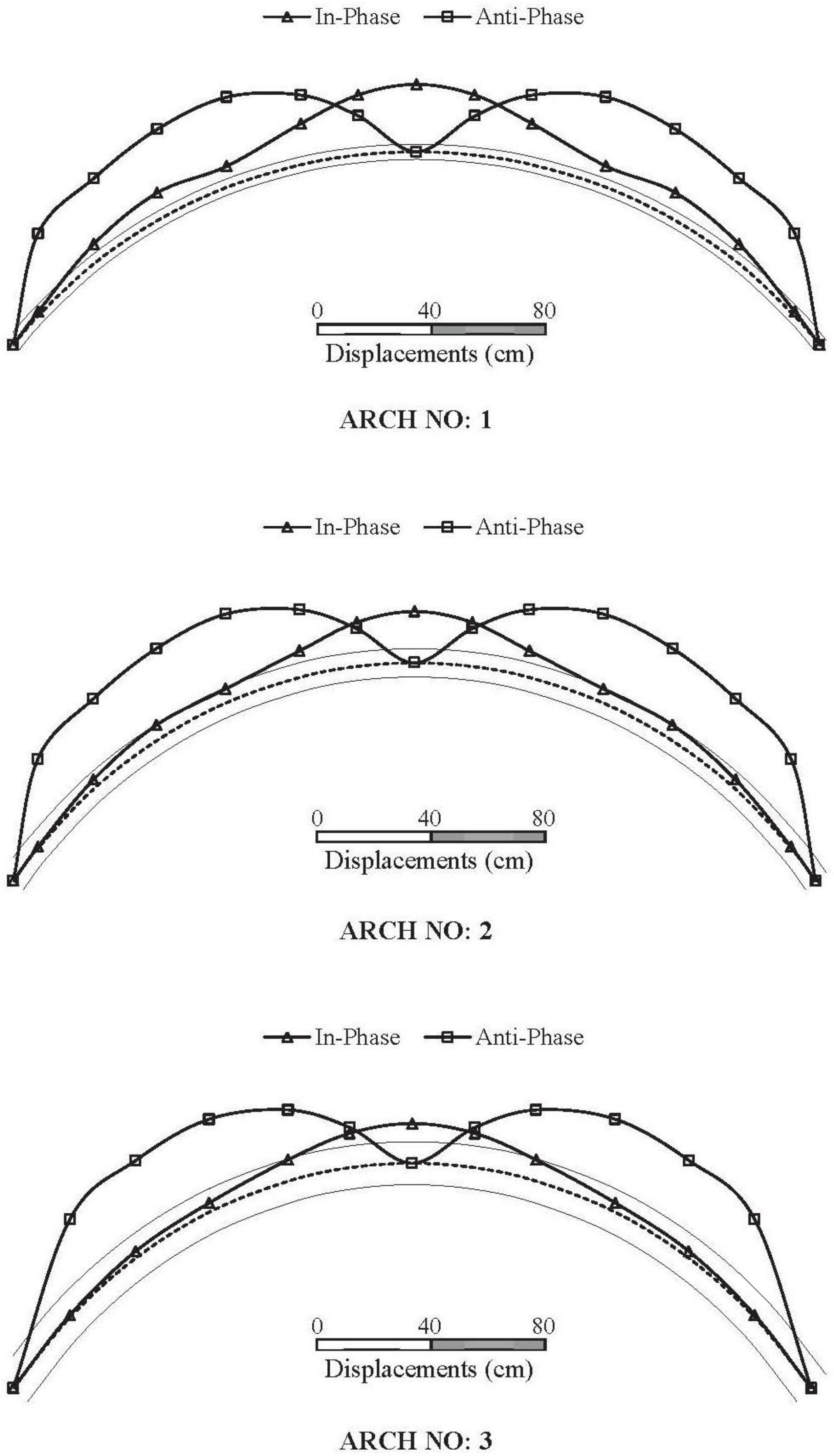


Fig. 6. The absolute maximum nodal displacements at arch section 1, 2 and 3.

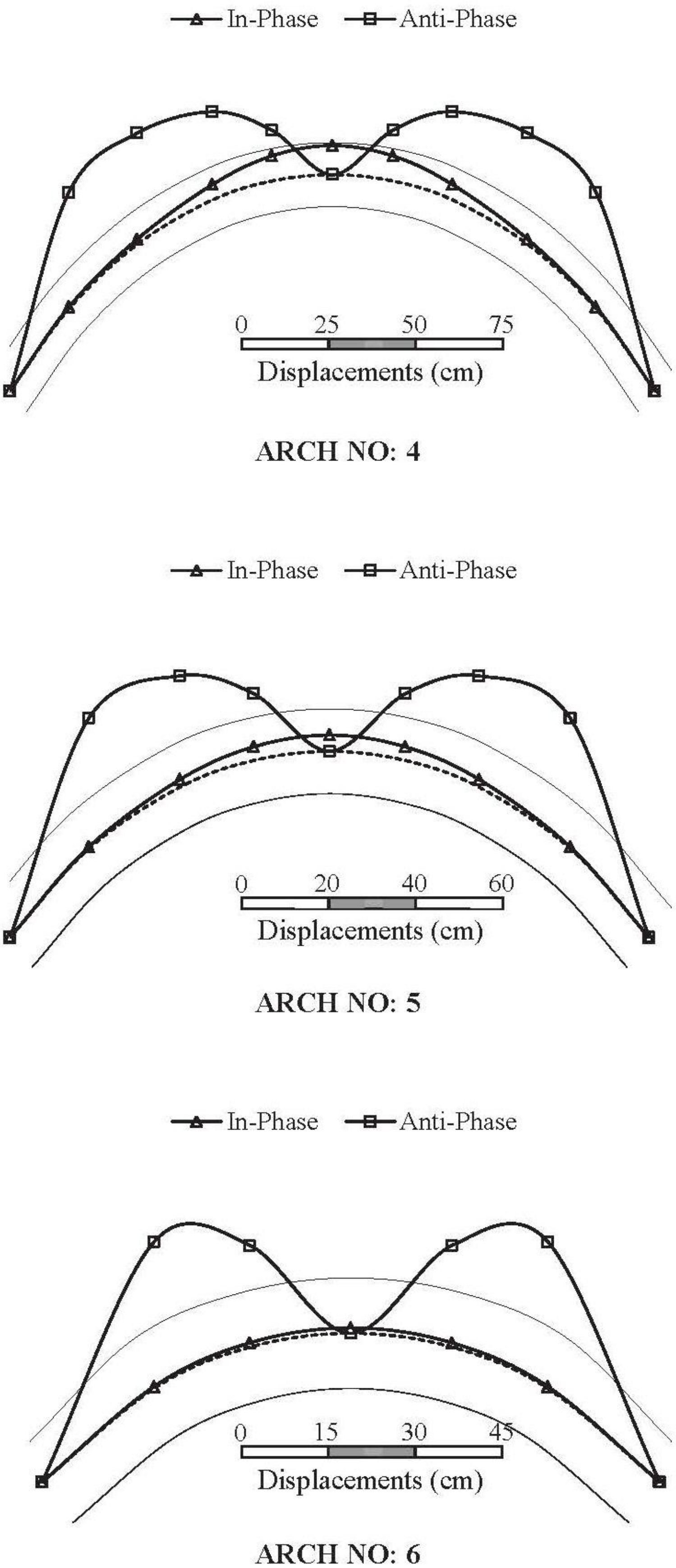


Fig. 7. The absolute maximum nodal displacements at arch section 4, 5 and 6

Table 1. The absolute maximum element stresses on upstream face at section I-I.

The Absolute Maximum Element Stresses (MPa)						
Element Number	σ_{xx}		σ_{yy}		σ_{zz}	
	In-Phase	Anti-Phase	In-Phase	Anti-Phase	In-Phase	Anti-Phase
10	3.39	1.48	0.81	0.29	3.96	2.21
16	1.69	0.36	0.71	0.39	2.62	1.79
24	1.18	1.80	0.69	0.90	2.08	2.19
34	2.30	5.77	1.07	0.26	1.57	2.03
46	5.10	6.49	0.39	0.25	3.15	1.92
58	7.72	3.02	0.39	0.29	3.93	0.80
70	7.72	3.02	0.39	0.29	3.93	0.80
82	5.10	6.49	0.39	0.25	3.15	1.92
94	2.30	5.77	1.07	0.26	1.57	2.03
106	1.18	1.80	0.69	0.90	2.08	2.19
116	1.69	0.36	0.71	0.39	2.62	1.79
124	3.39	1.48	0.81	0.29	3.96	2.21

Table 2. The absolute maximum element stresses on downstream face at section I-I.

The Absolute Maximum Element Stresses (MPa)						
Element Number	σ_{xx}		σ_{yy}		σ_{zz}	
	In-Phase	Anti-Phase	In-Phase	Anti-Phase	In-Phase	Anti-Phase
9	1.27	0.82	3.05	2.05	1.88	0.86
15	2.01	1.62	1.62	1.97	2.76	1.84
23	4.21	2.50	2.03	1.45	1.57	2.00
33	4.17	4.30	1.39	1.54	1.81	1.69
45	2.38	4.31	0.10	0.60	2.34	1.01
57	2.68	1.54	0.48	0.25	2.59	0.29
69	2.68	1.54	0.48	0.25	2.59	0.29
81	2.38	4.31	0.10	0.60	2.34	1.01
93	4.17	4.30	1.39	1.54	1.81	1.69
105	4.21	2.50	2.03	1.45	1.57	2.00
115	2.01	1.62	1.62	1.97	2.76	1.84
123	1.27	0.82	3.05	2.05	1.88	0.86

4. Conclusions

The response of arch dams is presented for anti-phase dynamic effects when there is no water in the reservoir. The dynamic effect can cause relative movements of the banks of the arch dams according to each other. This situation can disturb their stability and cause their collapse. The quasi-static displacements occurred during the anti-phase ground motion change significantly the response of the arch dams. On the numerical results presented, it is seen that the displacements increase due to the anti-phase dynamic effects and the stresses have the maximum value around the axis of symmetry of the dam. Consequently, it may be stated that the response of arch dams itself should be considered but not overlooked for anti-phase dynamic effects. However, it should be investigated the effects taken into considering fluid-structure interactions in further studies.

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