The influence of bridge–tower interaction on the dynamic behavior of intake–outlet towers

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Abstract

The dynamics of a coupled system of concrete bridge–intake tower interacting with reservoir water and foundation rock is numerically studied considering an intake tower submerged in reservoir and an access bridge connected to the top of the tower. The system is investigated in the frequency domain using frequency response functions of the tower. Various cases are generated and the analyses are conducted under horizontal harmonic excitation. The coupled system is three-dimensionally modeled using the finite element method. Then the system is analyzed in the time-domain considering the horizontal component of El-Centro earthquake. It is shown that the presence of the bridge significantly influences the dynamic response of the tower. It is concluded that the effects of bridge interaction are of much importance in the response of tall slender towers when they are inside the reservoir.

Keywords Intake–outlet towers · Bridge–tower interaction · Structural dynamics · Frequency response functions

1 Introduction

Concrete dams and their intake–outlet towers have important rule in water supply for the public utilities [1]. After preparing the general layout of the dam and its intake–outlet towers, operational and maintenance provisions should be developed. Where it is necessary, a single lane access bridge extends from valley or top of dam to intake tower to provide access for the operation, repair and maintenance of equipment. The access bridge has to be designed for all possible loading scenarios including AASHTO standard truck and off-highway loads due to cranes lifting, transporting gates and equipment into or out of intake tower [2]. The intake towers usually located close to dams, and in general cases, there are more than one tower in reservoir [3]. For example, in Hoover dam site, shown in Fig. 1, there are two towers along with their

access bridges that connect these towers to each other and to the main dam (Hoover Dam [4]).

Most previous researches on intake towers have considered towers that are anchored to the supporting foundation or rock, but they were freestanding towers without any structural connection to the dam or the valley [3, 5]. The dynamic response of dams, intake towers and their interaction with reservoir water and foundation rock has been studied in the literature [3, 5–16]. Considerable studies have been carried out on the subject of soil-structure and water-structure interaction of towers. Most recent works have focused on the dynamic behavior of hollow intake towers considering many affecting parameters such as rigidity and flexibility of foundation, foundation-tower interaction, and also various geometric shapes ranging from circular to non-circular cross-sections [3]. It has been showed that surface waves and water compressibility effects are negligible in the dynamic and frequency

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Fig. 1 Hoover arch-gravity dam and its intake towers with access bridges

response functions of submerged towers [5]. In real systems, the intake tower may be located in vicinity to the dam or valley, therefore, it has been shown that the wave reflection from the dam may change the natural frequency and seismic response of intake towers [1]. However, based on the authors' knowledge, investigating the dynamic response of intake tower interacting with its access bridge during strong earthquakes is very rare; it could be the subject of further study.

The objective of this paper is to numerically investigate the dynamic behavior of intake tower in presence of an access bridge considering their interaction themselves, with surrounding reservoir and inside water, and foundation rock. For this purpose, various simplified problems are considered: (1) a square hollow concrete intake tower in the presence of three different span access bridges, (2) inside and surrounding water, and finally (3) in the presence of surrounding water and foundation. The coupled system is modeled using the finite element method. The steady state dynamic response of the tower–bridge system to harmonic horizontal excitation is presented in the form of frequency response functions. The time history response of the system under a real earthquake is compared in various case studies.

2 Numerical modeling

The considered system, in general, contains four different components: (1) the intake tower, (2) the access bridge (with or without mid-span column), (3) the reservoir water, and (4) the foundation rock, that is shown in Fig. 2. The

governing differential equation of the structural components, including the tower, the access bridge, and the foundation, in the displacement-based Lagrangian formulation, assuming no static gravity, is:

$$\nabla \sigma - \rho_s \ddot{u} = 0 \quad (1)$$

where σ is the Cauchy stress tensor, u is the displacement vector, ρ_s is the solid mass density and ∇ indicate the Del operator Sommerfeld [17]. The governing equation of the fluid domain utilizing the pressure-based Eulerian formulation, considering that the fluid (water) is linearly compressible, neglecting its internal viscosity and having small amplitude irrotational motion, can be represented as:

$$\nabla^2 p - \frac{1}{c^2} \ddot{p} = 0 \quad (2)$$

where p is the hydrodynamic pressure in excess of hydrostatic pressure, c is the acoustic wave speed in the fluid and the ∇^2 represents the Laplacian [7]. The main boundary conditions are shown in Fig. 2 which are: (1) the fluid free surface, (2) the fluid–structure interaction, (3) truncated far-end and (4) lateral boundaries. Because the effects of surface waves can be neglected, therefore the free surface of water can be assumed as zero-pressure boundary: $p=0$ [18]. As the result of the assumption that there is no flow in solid–fluid surface, the governing equation can be represented as:

$$\frac{\partial p}{\partial n} = -\rho_f \ddot{u}_n \quad (3)$$

where ρ_f is the fluid density and n is the normal vector on different surfaces in all types of interfaces [19].

Fig. 2 The coupled system of tower–bridge–water and its boundary conditions

The solid faces of all water–structure interfaces are assumed as impermeable; hence the wave absorption is neglected. The reservoir is assumed infinite, but in the finite element method it should be truncated in suitable distance from the fluid–solid interface. In this case, the far-end boundary condition (or transmitting boundary condition) has to absorb pressure waves going away from system; therefore, utilizing the well-known Sommerfeld boundary condition is suggested [17]:

$$\frac{\partial p}{\partial n} = -\frac{1}{c} \dot{p} \quad (4)$$

The last two equations represent the natural boundary conditions of the fluid domain.

3 Analysis cases and numerical examples

The complete system is shown in Fig. 3 The intake tower structure in this research is a square hollow slender one with the total structural height of $H_t = 85$ m with a footing submerged in water. The tower cross-section is a hollow prismatic square section with the outside dimension of $b_t = 8$ m and the section thickness of $t_t = 0.8$ m. The footing is a square cube with the height of $H_f = 5$ m and section dimension of $b_f = 20$ m. In addition, the model consists of a reservoir and an access bridge which has rigid connection to the top of the intake tower. All of structural interactions are tied to each other. The access bridge is fixed in all degrees of freedom to the intake tower, the intake tower is also fixed to the foundation rock at the tower's base, and the mid-span column is fixed in all degrees of

freedom to the access bridge at its top. The base of the mid-span column, similar to the intake-tower, is assumed fixed in all degree of freedom to the foundation rock. The surrounding reservoir is assumed infinite from each side of the intake tower with the depth of $H_i = 75$ m. The water inside the hollow tower is considered with the same depth as the surrounding (outside) water. The above system is placed on a horizontal surface of a deformable rock foundation. The height of foundation rock is 80 m.

The access bridge has a box section which is suggested by AASHTO for mid-span bridges (see Fig. 3) AASHTO [20]. Total width of the bridge is $b_b = 5$ m, height of the box section is $h_b = 1.9$ m and the thickness of the box is uniformly assumed as $t_b = 0.25$ m.

The concrete material of the intake tower, the bridge, and the footing is assumed to be homogeneous, isotropic, linear elastic with these dynamic properties: elastic Young's module: $E_c = 31$ GPa, mass density: $\rho_c = 2430$ kg/m³ and Poisson's ratio: $\nu_c = 0.15$. The effects of reinforcing steel are assumed to be neglected Goyal and Chopra [19]. The unit mass of water is $\rho_w = 1000$ kg/m³; the water is assumed to be compressible with bulk modulus of $K_w = 2.07$ GPa. The whole system is three-dimensionally modeled using the finite elements method by Eulerian–Lagrangian approach. The system is linear without any joint or geometric non-linearity. Each component is discretized into several eight-node hexahedral elements. The finite element mesh is shown in Fig. 4. The lateral faces of the foundation have free slip boundary conditions with zero transverse displacement. The reservoir lateral boundaries, parallel to the direction of applied input motion, are assumed to be rigid, i.e. $\partial p / \partial z = 0$ where z is the perpendicular direction to the

Fig. 3 The general layout of the system **Fig. 4** The finite element meshes of the system

lateral boundaries Millan et al. [1]. The ABAQUS finite element software is chosen for the modeling and analysis Abaqus [21].

To investigate the effects of the presence of the access bridge with various lengths, the reservoir water and the

foundation rock, on the dynamic response of the intake tower, several analysis cases are considered as tabulated in Table 1. From this table, in TB group, the effects of the access bridge with various spans are studied. In this group, first the intake tower is modeled solely, after that, a 20 m, 40 m and 80 m bridge is added to the model. In the case of 80 m bridge, a mid-span column is also modeled with the height of 83.1 m (Fig. 3). In TBR group the reservoir is added to the TB group models. It should be noticed that the surrounding and inside water is considered in this group. In TB and TBR groups, the foundation is assumed to be rigid. In TBRF group, the foundation rock is added to models. It is assumed to be massless and deformable with elastic modulus same as the concrete, i.e. $E_f/E_c = 1$, with Poisson's ration of $\nu_f = 0.5$. It has been demonstrated that other foundation rock elastic modulus (i.e. $E_f/E_c = 0.5$) slightly influenced the frequency response of solitary intake-towers, specifically in the first two modes of vibration [3]. The connection of the bridge to the tower and the mid-span column is fixed in all degrees of freedom.

The analysis cases are first analyzed in frequency-domain computing their frequency response functions under the harmonic horizontal and vertical excitations. After that, the time domain analysis of all cases is done considering the horizontal and vertical components of 1940 Imperial Valley earthquake recorded at El-Centro station (Fig. 5). In the models with rigid foundation, the free-field earthquake record is applied to the base of the model and the end of the

Table 1 All cases analyzed in this research

Group Name	Case	20 m Bridge	40 m Bridge	80 m Bridge with mid-span column	Surrounding water	Inside water	Foundation condition	E_f/E_c
TB	1	x	x	x	x	x	Rigid	∞
	2	√	x	x	x	x	Rigid	∞
	3	x	√	x	x	x	Rigid	∞
	4	x	x	√	x	x	Rigid	∞
TBR	5	x	x	x	√	√	Rigid	∞
	6	√	x	x	√	√	Rigid	∞
	7	x	√	x	√	√	Rigid	∞
	8	x	x	√	√	√	Rigid	∞
TBRF	9	x	x	x	√	√	Flexible	1
	10	√	x	x	√	√	Flexible	1
	11	x	√	x	√	√	Flexible	1
	12	x	x	√	√	√	Flexible	1

Fig. 5 The 1940 Imperial Valley earthquake at El-Centro station; horizontal component

access bridge. In the models with massless foundation, the earthquake record is simultaneously applied to the outer boundary of the foundation and the end of access bridge. The earthquake record is applied in two main directions i.e. perpendicular and along the bridge axis separately and all of the results are obtained in both directions. In the time-history analysis, the Rayleigh method is adopted such that it produces 5% critical damping in the first two vibration modes of the models. The damping is considered in the time-history analyses using the Rayleigh damping method with mass and stiffness proportional factors to produce 5% critical damping in the first two vibration modes of the coupled system.

Fig. 6 The first mode shapes of cases 3 and 4

Table 2 Comparison of the first two modes (in Hz) in longitude and lateral direction

Group name	Case	First mode, ω_1 (ω_1/ω_1^D)	Second mode, ω_2 (ω_2/ω_2^D)
TB	1	0.88 (1.00)	0.88 (1.00)
	2	1.42 (1.61)	3.09 (3.52)
	3	1.00 (1.14)	2.85 (3.25)
	4	0.76 (0.86)	2.73 (3.11)
TBR	5	0.64 (0.73)	0.64 (0.73)
	6	1.00 (1.14)	1.72 (1.95)
	7	0.70 (0.80)	1.72 (1.95)
	8	0.64 (0.73)	1.66 (1.89)
TBRF	9	0.64 (0.73)	0.64 (0.73)
	10	1.00 (1.14)	1.72 (1.95)
	11	0.70 (0.80)	1.66 (1.89)
	12	0.64 (0.73)	1.60 (1.82)

4 Frequency response functions (FRFs)

The steady-state dynamic response of all analysis cases are calculated under unit harmonic free-field ground motions, with the frequency range from 0.1 up to 15 Hz. The first mode shapes of the coupled tower–bridge system in cases 3 and 4 are shown in Fig. 6. In general, the first mode in the presence of the bridge shows lateral displacement which is perpendicular to bridge axis.

The natural frequencies of the lowest two modes of the analysis cases in two different directions, i.e. perpendicular (first mode) and along the bridge axis (second mode), are compared in Table 2. When the tower is solely analyzed without bridge, these two frequencies, which are named ω_1^D or ω_2^D , are the same because of complete symmetry.

The other frequencies are normalized with respect to $\omega_1^D = \omega_2^D$ from Table 2, in lateral direction (perpendicular to the bridge axis), adding the 20 m and 40 m bridges increases the natural frequency of the tower, but 80 m bridge reduces it. That is because of a tradeoff between the lateral stiffness and the mass of the bridge which is added to the system. The water generally decreases the first natural frequency of the system, as expected. But it also decreases the frequency shift when the bridge is coupled to the tower. In this direction, the foundation has negligible effect on the frequency as this is the case for slender high towers.

In longitudinal direction (bridge axis), adding of the bridges essentially increases the second natural frequency of the system because of high axial stiffness of the bridge. Same as the first mode, shorter bridge causes more change in the frequency. In this direction, the foundation flexibility has slight effect on the frequencies specifically for longer bridges.

4.1 TB group (tower–bridge)

The frequency response functions (FRFs) of the displacement of the top of tower in TB group cases are illustrated in Fig. 7. The absolute values of displacements normalized by the fundamental resonant amplitude of the tower only, without reservoir on rigid foundation (case 1) u_1 , are plotted against the frequency parameter ω . The FRFs are separately plotted in the lateral (perpendicular to the bridge axis) and the longitudinal (along the bridge axis) directions.

Although the cross-section of the bridge is smaller than the tower, but the bridge–tower interaction substantially affects the frequency response functions specifically in the

Fig. 7 FRFs of TB group in **a** lateral direction (perpendicular to the bridge axis), and **b** longitudinal direction (along the bridge axis)

Fig. 8 Frequency response functions of TBR group in **a** lateral direction (perpendicular to the bridge axis), and **b** longitudinal direction (along the bridge axis)

longitudinal direction. The tower that has the same behavior in both directions when it is alone, now shows very different behaviors when interacts with the bridge. The shorter bridge has more lateral stiffness; hence its effect is higher. In the longitudinal direction, the presence of the bridge has much more influence than the lateral direction even for higher natural modes. This shows that the bridge–tower interaction should be taken into account in the dynamic analysis of the intake towers with access bridges.

4.2 TBR group (tower–bridge–reservoir)

In this group, the reservoir is added to the model and its effects are investigated. As it is represented in Fig. 8, the reservoir (both surrounding and inside water) totally changes the FRFs compared with Fig. 7. In the presence of the reservoir, the bridge has slight effect on the FRFs in the lateral direction; just the 20 m bridge changes the fundamental frequency of the tower about 40%. The changes are again significant in the longitudinal direction because of high axial stiffness of the bridge, specifically for the lower modes. Some single peaks are altered to double peaks in the presence of the bridge. All second modes of models have same shape with relatively close frequency value. The second mode shape of the case 7 in longitudinal direction with the frequency of 1.72 rad/s is depicted in Fig. 9. Because the first few modes have the most contribution in the dynamic response of slender towers, the complete interaction of the tower, bridge and reservoir should be considered in the seismic analysis.

4.3 TBRF group (tower–bridge–reservoir–foundation)

The FRFs of this group is shown in Fig. 10. Comparing this figure with Fig. 8 reveals that the flexibility of the foundation has negligible effects on the governing frequencies of the tower–reservoir system regardless of the presence of the bridge. It has been previously shown for single tall slender towers Goyal and Chopra [19]; Bosler and Lopez [22], but here it is extended for the coupled tower–bridge systems. However, the foundation flexibility may increase

Fig. 9 The second mode shape of the case 7 in longitudinal direction

Fig. 10 Frequency response functions of TBRF group in **a** lateral direction (perpendicular to the bridge axis), and **b** longitudinal direction (along the bridge axis)

the amplitudes of the FRFs specifically for the 80 m bridge with the mid-span column.

5 Time-history seismic response

Due to the differences in the FRFs of the tower in both directions when interacts with the bridge, the cases are now analyzed in the time-domain. Seismic responses of the analysis cases under the horizontal component of the El-Centro earthquake are separately computed in both lateral and longitudinal directions. When the earthquake is applied in the lateral direction, the time history of the

relative displacement of the top of the tower with respect to its base in this direction is shown in Fig. 11 for various groups. The longitudinal displacement of the tower (along the bridge axis) in this situation is negligible in all groups regardless of the presence of the bridge. In the lateral direction, the 80 m bridge with the mid-span column does not considerably change the response of the tower in all groups. But the 20 m and 40 m bridges would change the response both in magnitude and the frequency content. It should be noticed that the obtained results are earthquake record-dependent.

According to Fig. 11, in the presence of the bridge in cases 2 to 4, it is observed that shorter bridge causes lower

Fig. 11 Time-history of the relative displacement of the tower in the lateral direction under the horizontal component of the El-Centro earthquake: **a** TB and **b** TBR group

Fig. 12 Comparison of the seismic response of the tower in interaction with various length bridges

tower displacement. Due to the stiffness of the bridge, the frequency content has increased. In case 4, it is observed that the maximum displacement is identical with the conditions of which the tower has been modeled without the access bridge. As a result, it can be shown that increasing the bridge length or in the other words, reducing the lateral stiffness neglects the effect of the access bridge.

To study the effects of the tower–bridge interaction in various groups, the seismic response of the tower only, tower + 20 m bridge, tower + 40 m bridge, and tower + 80 m bridge and the mid-span column is compared in Fig. 12 with or without the reservoir and foundation interaction. In the cases 5 to 8, compared to the

cases 1 to 4 the reservoir increases the dynamic displacement amplitude. Similar to the results of case 4, in case 8 in comparison with case 5, the displacement amplitude and frequency content are the same, which indicates that in the presence or absence of the reservoir for the bridge span length above 80 m, the access bridge has limited effect on the dynamic responses. Figure 12 indicates that in shorter bridge span length, the effect of the reservoir is more significant than the larger bridge span length and the displacement amplitude is increased in higher range.

If the earthquake record is applied in the longitudinal direction, the lateral displacement of the tower in all groups is zero because of complete symmetry. However,

the longitudinal displacement of the tower is negligible because of high axial stiffness of the bridge. It is the case for all bridges with various span lengths.

6 Concluding remarks

Dynamic Response of the intake tower interacting with various access bridges considering the reservoir and foundation interaction is investigated. The system is three-dimensionally modeled using the finite elements by Eulerian–Lagrangian approach. An 80 m high intake tower is modeled in an infinite reservoir with an access bridge which is loaded and designed according to AASHTO regulations.

To study the effects of the span length of the bridge, three bridges with spans of 20, 40 and 80 m is added to the top of the intake tower. The effect of the presence and absence of the reservoir, the rigidity and flexibility of the foundation rock and the presence and absence of the access bridge on the tower is represented.

Frequency response functions of displacement of top of the tower under horizontal ground motion are provided for different cases. The stiffness of foundation rock is considered to be the same as the stiffness of the concrete, and in the other case it is assumed to be rigid that in these two cases, the change in the frequency response functions of the tower is negligible. It is also shown that the interaction of the reservoir-structure reduces the frequency response of the tower. On the other hand, the interaction between the reservoir and structure increases displacement amplitude significantly, and the results of the time history analysis indicate this fact. In vertical excitation, due to the symmetry of the model, no significant change is observed in the tower's displacement, and with presence of the access bridge, there is no significant change in the displacement and the frequency content of the tower.

The results show that in the direction of perpendicular to the bridge axis, as much as the span length is shorter, the frequency response functions are reduced, and in the 80-meter span length model, the effect of the bridge is negligible. In the direction of the longitudinal axis of the bridge, the results indicate that the displacement amplitude of the tower due to the presence of the bridge is approximately zero, and the frequency response in this direction has been tremendously reduced. With the presence of the reservoir, as the bridge length is shorter, the increase in the displacement amplitude has amplified and with the increase in the span length of the bridge, the displacement has also decreased. The results of the time history analysis under the El-Centro earthquake confirm the

previous conclusions and demonstrate the importance of the interaction between the tower and the bridge in seismic analysis.

Compliance with ethical standards

Conflict of interest The authors declare that they have no conflict of interest.

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