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Experimental study of dynamic interaction between group of intake towers and water

Haibo Wang[*1], Deyu Li[1] and Bihua Tang[2]

[1]State Key Laboratory of Simulation and Regulation of Water Cycle in River Basin,
China Institute of Water Resources and Hydropower Research, Beijing 100048, China
[2]Hydrochina Chengdu Engineering Corporation, Chengdu, 610072, China

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Abstract. Dynamic test with scaled model of a group of intake towers was performed to study the dynamic interaction between water and towers. The test model consists of intake tower or towers, massless foundation near the towers and part of water to simulate the dynamic interaction of tower-water-foundation system. Models with a single tower and 4 towers were tested to find the different influences of the water on the tower dynamic properties, seismic responses as well as dynamic water-tower interaction. It is found that the water has little influence on the resonant frequency in the direction perpendicular to flow due to the normal force transfer role of the water in the contraction joints between towers. By the same effect of the water, maximum accelerations in the same direction on 4 towers tend to close to each other as the water level increased from low to normal level. Moreover, the acceleration responses of the single tower model are larger than the group of towers model in both directions in general. Within 30m from the surface of water, hydrodynamic pressures were quite close for a single tower and group of towers model at two water levels. For points deeper than 30m, the pressures increased about 40 to 55% for the group of towers model than the single tower model at both water levels. In respect to the pressures at different towers, two mid towers experienced higher than two side towers, the deeper, the larger the difference. And the inside hydrodynamic pressures are more dependent on ground motions than the outside.

Keywords: dynamic interaction; intake tower; shaking table test; massless foundation; hydraulic structure

1. Introduction

In the event of an earthquake, it is vitally important to prevent the catastrophic failure of a dam and subsequent uncontrolled release of the reservoir. For most embankment dams, and some concrete dams, the release of water is controlled through concrete intake towers. Damage to or failure of an intake tower may result in a reduced ability to lower the reservoir, which is critical to minimize the risk of catastrophic failure of the dam immediately after an earthquake, or the disruption of water supply following an earthquake. Therefore, intake towers in seismically active regions should be designed to withstand earthquakes using rational analytical methods based on a thorough understanding of the dynamic behavior of such towers (U.S. Army Corps of Engineers 2003). Seismic safety of the intake towers of hydro projects attracted many research works in

[*]Corresponding author, Professor, E-mail: wanghb@iwhr.com

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recent years. Sabatino (2008) carried out a series of tests aimed at investigating the seismic performance of typically reinforced, non-seismically designed towers. Millán et al. (2009) investigated the seismic responses of the tower affected by the presence of the dam by coupled boundary element and finite element method in the frequency domain. Vidot and Suárez (2004) examined the effect of different ground accelerations at the supports of the tower and an access bridge to the seismic response of intake-outlet towers. Salah-Mars (2011) investigated the potential failure modes of the intake tower and Borel conduit at Lake Isabella auxiliary dam. Chen (2010) investigated the behaviors of intake towers under strong seismic excitation by response spectrum analysis with three-dimensional finite element model. Cocco (2010) applied different static nonlinear procedure to intake tower structures to develop nonlinear seismic response capacity spectrum.

The dynamic response of an intake tower during an earthquake may present quite complex characteristics due to many factors. One is the structure-water dynamic interaction, which is important to all kinds of hydraulic structures (Calayir 2005, Maity 2005). Many numerical methods (Lin 2007, 2012, Wang 2011) based on finite element method used on the dam-reservoir are applicable to intake tower as well, although the numerical models may be more complicated as there may be too many interfaces between the tower and water. Goyal and Chopra (1989 a,b,c,d) developed a simplified procedure to represent the hydrodynamic interaction effects due to horizontal seismic motion with added mass distributed over the height of the tower, based on the assumption that a rigid tower is surrounded by incompressible water of uniform depth extending to infinite in horizontal. Daniell and Taylor (1994) conducted in situ forced vibration test on a 50 m high intake tower at Wimbleball dam in the U.K. to verify the hydrodynamic interaction effects on the vibration mode of the tower.

Intake towers can be free-standing or partially supported against the rock abutment. For big reservoirs, a group of towers are necessary to accommodate flood operation demands, to fulfill water supply or power generation operation. Most of research works were focused on the seismic responses of a free-standing single tower, which may quite different from the seismic responses of a group of towers. This research is the dynamic test with scaled tower models of a series of experimental and numerical investigations into the seismic behavior of a group of intake towers, including the dynamic interaction between water and towers, between towers and foundation as well as between neighboring towers. The dam selected for this special case study was Changheba Dam, which is a rockfill dam with a gravel-clay central core. The highest dam section is 240 m in height and the crest length is 497.94 m, with the reservoir storage of 1.075 Gm^3 at normal operation water level. The dam site is located in a seismically active region of Sichuan province, China, where a very destructive earthquake of magnitude 8.0 occurred in May 12, 2008. The power station is constructed underground with four diversion penstocks and four 75 m high intake towers at upstream end aligned on the left bank of the river.

2. Description of the group of towers and test model

The total width of the four towers is 137.2 m in the direction perpendicular to flow and the width parallel to flow is 30m, as shown in Fig. 1. The drawings of horizontal and vertical cross-section of the middle towers are given in Fig. 2. The height is 75 m, with their top 9 m higher above the reservoir. Each tower consists of a service gate slot, an emergency gate slot and an air vent. The trash rack is set at the upstream side of each tower, with five openings forming by

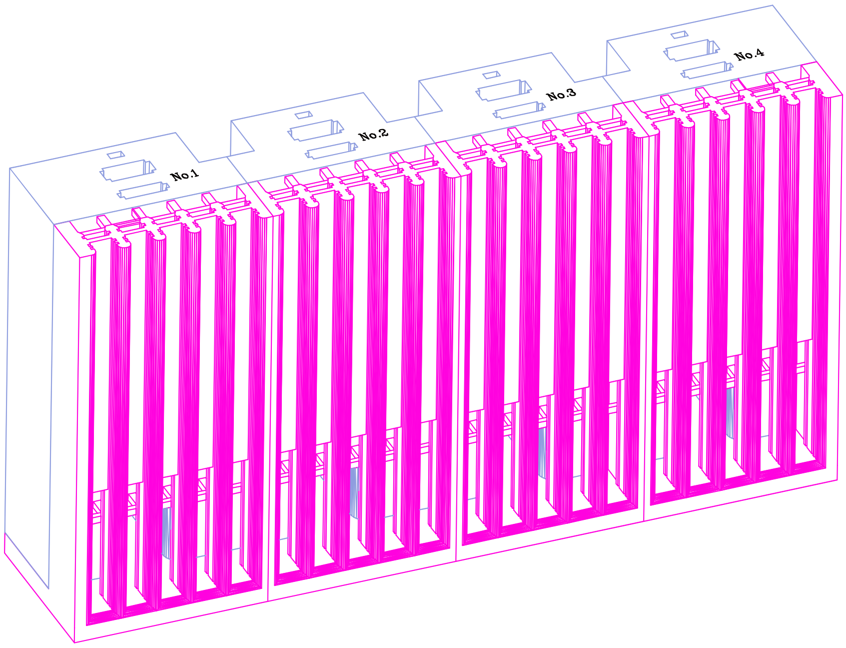

Fig. 1 Sketch of the group of towers of Changheba Dam project

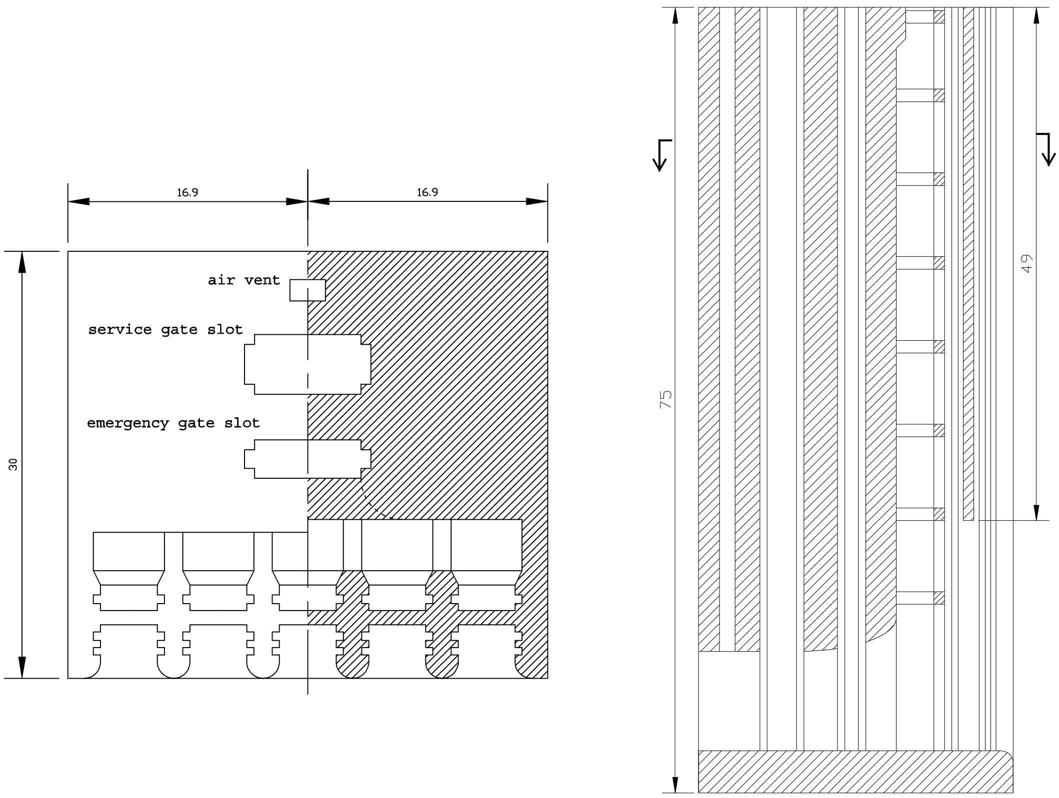

Fig. 2 Drawing of horizontal and vertical cross-section of the middle towers

four central piers and two side piers. There are slabs and beams at an 8 m interval in elevation, above the entrance to connect the piers with each other and the piers with each tower. The submerged portion of the contraction joints between the towers is filled with water as well. The modulus of elasticity of the foundation rock is about 9 GPa and that of the concrete is 28 GPa.

A 1/90-scale elastic model was used for the dynamic test, which includes a model of the towers, as shown in Fig. 3, the foundation near the towers and part of water to simulate the dynamic interaction of tower-water-foundation system. The whole model extends 3.8 m in both directions perpendicular to and parallel to flow, representing a range of river bank and water for 342 m by 342 m, as shown in Fig. 4. The depth of the foundation below the tower base is 0.6 m in the model,

Fig. 3 A close-up photo of the group of intake towers

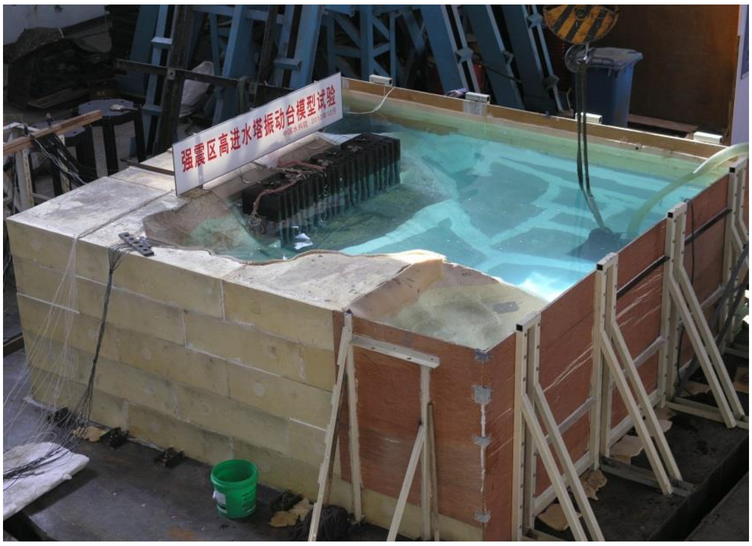

Fig. 4 The arrangement of the model on the shaking table

or 54 m in prototype. The towers were glued on the foundation with epoxy resin at bottom and rock abutment on the downstream side. The topography of the riverbank near the towers was closely simulated to account for its influence on the hydrodynamic pressure. The deepest water reached about 120 m for the prototype reservoir.

Due to the restriction of the maximum payload of the shaking table, a kind of polyurethane foam of 200 kg/m^3 in density and 100MPa in modulus of elasticity was used to build the model foundation, which was reasonably considered as a massless foundation, an assumption commonly adopted in numerical analyses to reflect the elasticity of the foundation. The material of the tower model was a specially made rubber of 2400 kg/m^3 in density and 311MPa in modulus of elasticity, with its strength higher than the value that meets the similarity factor in strength, since the test was focused on the responses with a linear material property only. It is a difficult task to develop a proper model material for concrete structures using a very small scale model that can be submerged under the water. Wang (2006, 2007) has tried a synthetic concrete with very low strength and low modulus for dynamic model tests of concrete arch dams, but it was used in the test separated from the water with a very thin membrane. It is realistic to leave the material non-linearity to the model test with a scale close to prototype intake tower structure, which

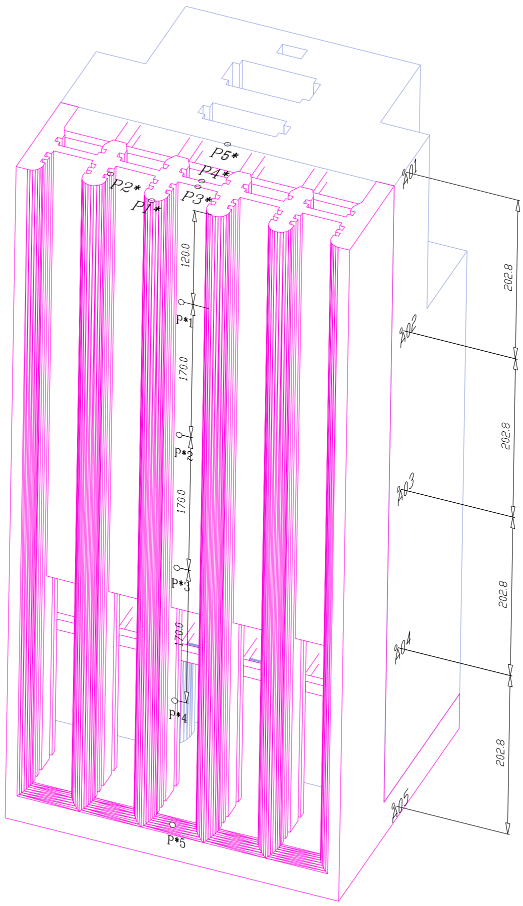

Fig. 5 Hydrophones and accelerometers on the No. 2 tower

consists of many beams and piers. It is much easier to find the proper model material meeting the requirement of similarity in strength with a reasonable large scale model.

The water in the model test is the same as in the prototype reservoir. Therefore its modulus of elasticity is 90 times higher than the value meeting the similarity requirement, which implies that a nearly incompressible fluid was used in the test.

To compare the different responses between a single tower and a group of towers, the first test was performed with a single tower, the No. 2 tower in Fig. 1, and the second test was performed with four towers. Both tests were conducted at no water, normal water and low water level conditions. The low water level is 18.3 m below the normal one.

3. Dynamic test equipment

The shaking table at China Institute of Water Resources and Hydropower Research (IWHR), Beijing, is a six-degree-of-freedom digital-controlled servo-hydraulic system with a platform of 5 m × 5 m and maximum payload of 20,000 kg. The working frequency band is from 0.1 to 120 Hz. Then the corresponding upper frequency can reach 12.6Hz for the prototype structure in the present test, where the scale in frequency is $\sqrt{90}/1.0 = 9.487$. The maximum accelerations, velocities, and displacements are 1.0g, ±400 mm/s and ±40 mm in horizontal directions, 0.7g, ±300 mm/s and ±30 mm in vertical direction, respectively.

A total of 82 channels of data were measured in the tests, including accelerations, dynamic strains, hydrodynamic pressure and opening of contraction joints during the test with LVDT. The sampling frequency of data acquisition was 1000 Hz. Most of sensors were set on tower No. 2 that was used in both the single tower test and four towers test, as illustrated in Fig. 5 for hydrophones and accelerometers. The hydrophones were named with an initial letter P plus two numbers, representing the position in the cross section and elevation, respectively. For example, P13 was located 460 mm below the top of the model tower on the third pier from the left. P3* and P4* were used to measure the outside and inside pressure of the slab between two central piers, and P5* were installed on the tower body. The accelerometers were embedded on the right side of the tower, referred as A01 to A05 from the top to the bottom in Fig. 5.

4. Test results

4.1 Dynamic characteristics

To measure the dynamic characteristics of the towers, white noise excitations of 0.1g were applied in the direction parallel to flow, perpendicular to flow and vertical direction separately, for every water level. The transfer function at every measured point of acceleration on the towers to the shaking table was calculated through its acceleration response to that on the table. Every experimentally determined transfer function was approximated by a pair of numerator and denominator polynomials through curve-fitting. Furthermore, the resonant frequencies and damping ratios were calculated from the roots of the polynomial denominator, and mode shape coefficients calculated from the residues of the transfer function at the corresponding resonant frequencies.

The fundamental resonant frequencies of a single tower at different water levels are listed in

Table 1. The frequencies decrease with the increase in water level as expected. The difference is smaller in the direction parallel to flow and larger in the vertical direction. Since water interacts with the tower on both sides in the direction perpendicular to flow, the drop in frequency in that direction is more than in the direction parallel to flow. As to the vertical direction, the hydrodynamic pressure on the flat foundation surface which was left for three additional towers is far greater than that in horizontal direction, therefore causes the largest drop in the frequency.

The fundamental resonant frequencies of the group of towers at different water levels are listed in Table 2Table . The resonant frequencies of the towers show small difference under no water condition, with maximum values of 3.7%, 2.1% and 1.1% in directions parallel to flow, perpendicular to flow and vertical, respectively. The differences are 2.0%, 5.1% and 0.89% at low water level and 0.96%, 0.72% and 0.66% at normal water level in the corresponding directions. The location of and the topography near the tower are the main contributors to the difference in the fundamental resonant frequencies although there is small structural variation for the middle and side towers. The fundamental resonant frequencies decrease from left to right in the direction parallel to flow since the left tower is the closest to the high rising riverbank. The fundamental resonant frequencies of two middle towers in the direction perpendicular to flow are higher than two side towers because the restriction effect to the foundation from towers. And the frequencies of two side towers in vertical direction are higher than those of the middle towers due to structural variation probably. The water level has little influence on the above sequence of the resonant frequencies from high to low.

The average fundamental resonant frequencies of the group of towers decreased by 21.8%, 1.7% and 17.0%, in the direction parallel to flow, perpendicular to flow and vertical, respectively,

Table 1 Fundamental frequencies of a single tower (Unit: Hz)

Direction	No water	Low level / Ratio to no water	Normal level / Ratio to no water
Parallel to flow	44.43	42.85 / 0.964	40.80 / 0.918
Perpendicular to flow	43.68	41.01 / 0.939	38.02 / 0.870
Vertical	99.12	83.52 / 0.843	76.27 / 0.769

Table 2 Fundamental Frequencies of group of towers (Unit: Hz)

Direction	No. tower	No water	Low level / Ratio to no water	Normal level / Ratio to no water
Parallel to flow	1	35.45	33.10 / 0.934	29.07 / 0.820
	2	34.96	32.97 / 0.943	29.08 / 0.832
	3	34.37	32.59 / 0.948	28.86 / 0.840
	4	34.12	32.43 / 0.950	28.80 / 0.844
Perpendicular to flow	1	42.73	43.02 / 1.004	42.59 / 0.997
	2	42.96	44.21 / 1.029	42.76 / 0.995
	3	43.50	45.09 / 1.036	42.90 / 0.986
	4	42.59	42.83 / 1.006	42.64 / 1.001
Vertical	1	82.80	75.24 / 0.909	73.10 / 0.883
	2	82.17	74.66 / 0.909	72.62 / 0.884
	3	81.92	74.57 / 0.910	72.66 / 0.887
	4	82.34	74.82 / 0.909	73.01 / 0.887

compared to the single tower without water. The decrease is mainly due to the larger mass of the group of towers on the same foundation. In the direction perpendicular to flow, however, the restriction effect to the foundation deformation from towers eliminates almost the influence of the larger mass on the frequencies. As expected, the tower-water dynamic interaction becomes stronger in the direction parallel to flow than the single tower, resulting in larger resonant frequency drop at both water levels. But for the motion in the direction perpendicular to flow, the water within the contraction joints between towers plays a role of normal force transfer between the towers, which increases the stiffness of the group of towers and cancels the influence of the hydrodynamic pressure on two side towers from the reservoir to an extent depending on water level. The frequency drop in the vertical direction is less than the single tower at both water levels.

Fundamental mode shapes at different water levels are displayed in Fig. 6 and Fig. 7 for the single tower and group of towers, respectively. Because the difference in bending stiffness is about 6 times in the two directions, bending deformation is dominant in the direction parallel to flow, rocking motion is dominant in the direction perpendicular to flow, regardless of the case of single tower or group of towers.

4.2 Seismic responses

Seismic motions were applied in the two horizontal and vertical directions simultaneously. The peak ground accelerations were 222 cm/s^2 in the horizontal direction and 148 cm/s^2 in the vertical direction. As the intake towers are located on rock foundation, all seismic motions used here are

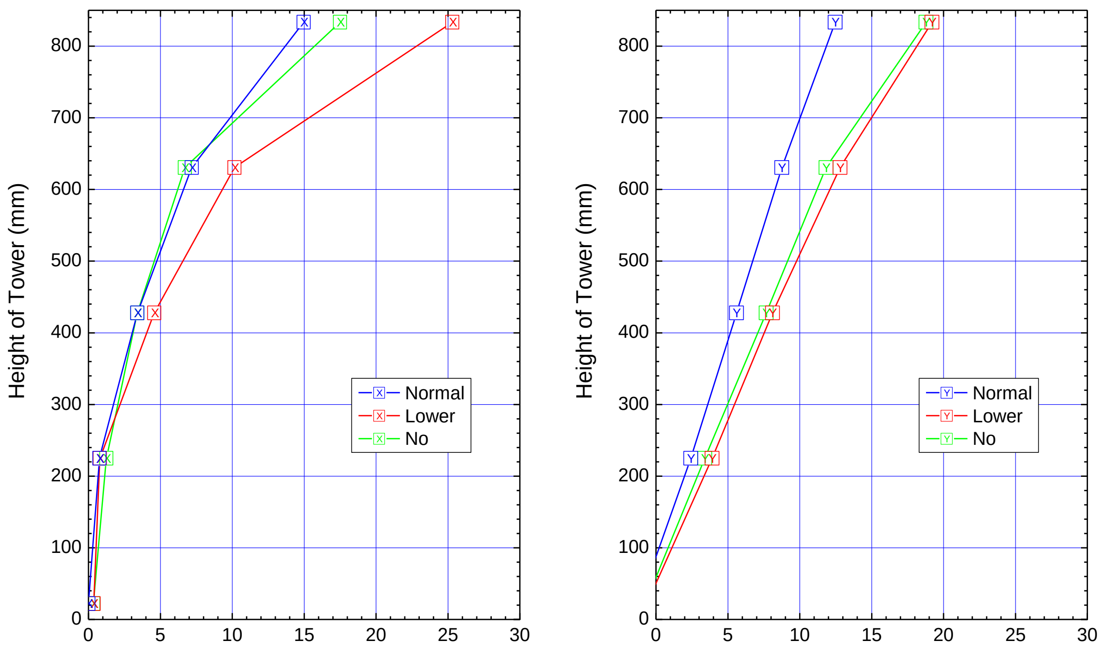

Fig. 6 Fundamental mode shapes of the single tower (left: parallel to flow, right: perpendicular to flow)

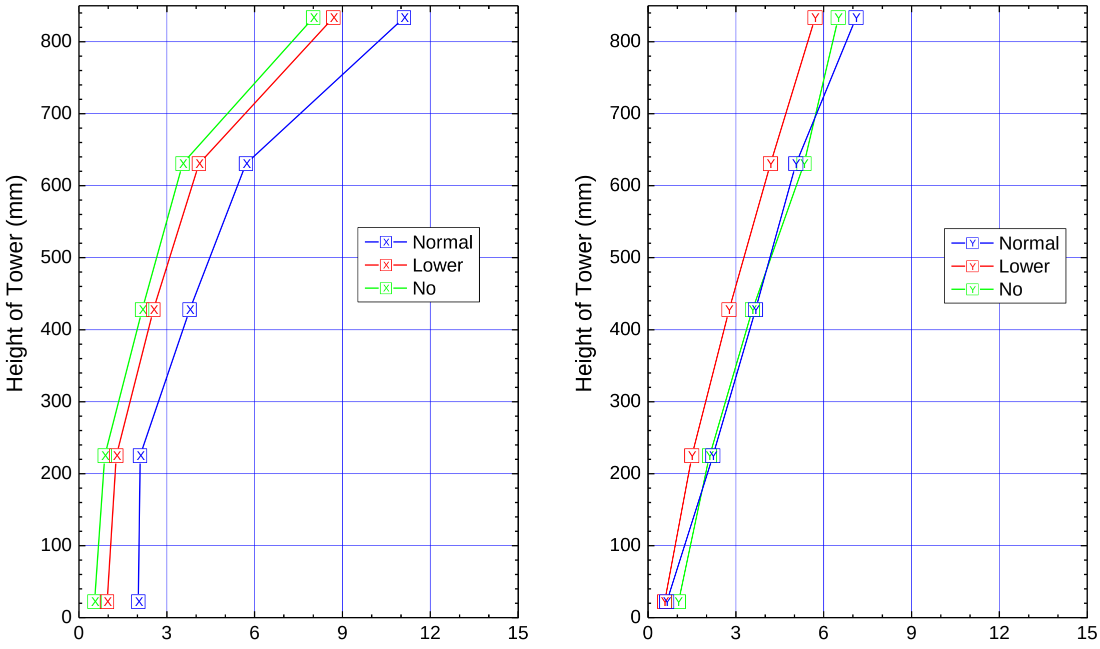

Fig. 7 Fundamental mode shapes of group of towers (left: parallel to flow, right: perpendicular to flow)

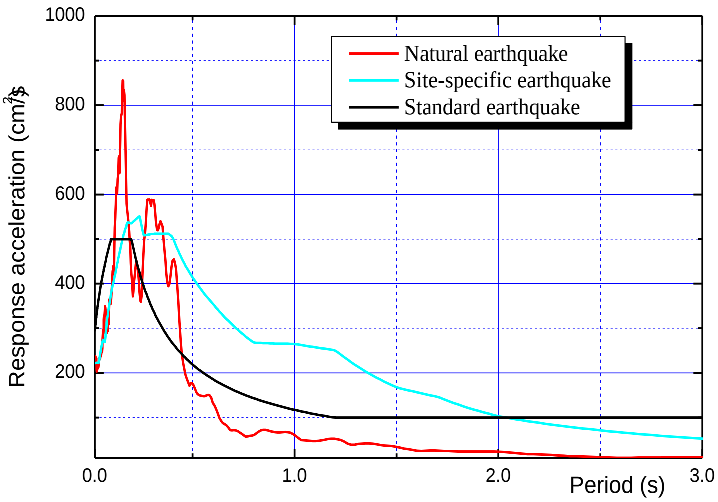

Fig. 8 Response spectra of three accelerograms in the direction parallel to flow (damping ratio of 5%)

accelerograms on the rock surface. Three sets of accelerograms were used in the test, known as standard earthquake, site-specific earthquake and natural earthquake. Their response spectra as well as the time histories in the direction parallel to flow are displayed in Fig. 8 and Fig. 9, respectively, where it is apparent the difference in these characteristics.

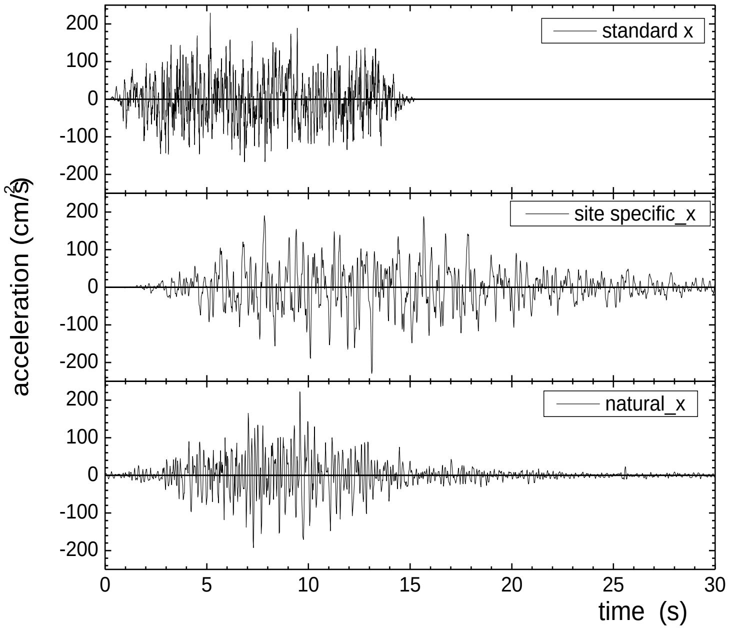

Fig. 9 Three accelerograms in the direction parallel to flow

Table 3 Acceleration responses of the single tower and acceleration on the shaking table (Unit: g)

Waves	Direction	Top of tower / Surface of the shaking table		
		No water	Lower level	Normal level
Standard	Parallel	1.51 / 0.161	1.35 / 0.158	1.27 / 0.152
	Perpendicular	1.08 / 0.240	1.24 / 0.256	0.96 / 0.302
	Vertical	0.50 / 0.096	0.49 / 0.102	0.48 / 0.113
Site-Specific	Parallel	1.74 / 0.223	1.43 / 0.226	1.44 / 0.217
	Perpendicular	2.11 / 0.262	1.49 / 0.238	0.94 / 0.168
	Vertical	0.43 / 0.114	0.44 / 0.140	0.41 / 0.146
Natural	Parallel	1.46 / 0.205	1.29 / 0.185	1.03 / 0.177
	Perpendicular	1.48 / 0.259	1.19 / 0.212	1.11 / 0.224
	Vertical	1.23 / 0.139	0.67 / 0.090	0.68 / 0.111

4.2.1 Acceleration responses

The maximum accelerations at the top of the single tower and tops of the group of towers are listed in Table 3 and Table 4, respectively, for all water levels subjected three different sets of seismic inputs. The maximum accelerations at the surface of the shaking table are given in the tables as well for every load case.

For the single tower, as shown in Table 3, the maximum accelerations at its top depend on the ground motion since both their response spectra and duration are quite different (refer to Fig. 8 and Fig. 9). The biggest was 1.47g in the direction parallel to flow under site-specific earthquake at

Table 4 Acceleration responses of group of towers and acceleration on the shaking table (Unit: g)

Waves	Direction	No. tower	Top of tower / Surface of the shaking table		
			No water	Lower level	Normal level
Standard	Parallel	1	0.862 / 0.163	0.849 / 0.143	0.819 / 0.155
		2	0.891 / 0.163	0.800 / 0.143	0.943 / 0.155
		3	0.962 / 0.163	0.793 / 0.143	0.726 / 0.155
		4	1.094 / 0.163	0.867 / 0.143	0.765 / 0.155
	Perpendicular	1	0.823 / 0.218	0.817 / 0.303	0.846 / 0.315
		2	0.762 / 0.218	0.818 / 0.303	0.845 / 0.315
		3	0.608 / 0.218	0.771 / 0.303	0.782 / 0.315
		4	0.659 / 0.218	0.850 / 0.303	0.812 / 0.315
	Vertical	1	0.450 / 0.099	0.458 / 0.102	0.414 / 0.110
		2	0.472 / 0.099	0.465 / 0.102	0.419 / 0.110
		3	0.472 / 0.099	0.445 / 0.102	0.476 / 0.110
		4	0.472 / 0.099	0.424 / 0.102	0.431 / 0.110
Site-Specific	Parallel	1	1.331 / 0.243	1.492 / 0.231	1.044 / 0.220
		2	1.118 / 0.243	1.451 / 0.231	1.234 / 0.220
		3	1.225 / 0.243	1.219 / 0.231	0.990 / 0.220
		4	1.314 / 0.243	1.246 / 0.231	1.125 / 0.220
	Perpendicular	1	0.823 / 0.246	0.845 / 0.245	0.648 / 0.224
		2	0.820 / 0.246	0.777 / 0.245	0.619 / 0.224
		3	0.801 / 0.246	0.769 / 0.245	0.631 / 0.224
		4	0.889 / 0.246	0.756 / 0.245	0.665 / 0.224
	Vertical	1	0.384 / 0.144	0.402 / 0.134	0.410 / 0.148
		2	0.379 / 0.144	0.426 / 0.134	0.361 / 0.148
		3	0.392 / 0.144	0.363 / 0.134	0.325 / 0.148
		4	0.334 / 0.144	0.326 / 0.134	0.302 / 0.148
Natural	Parallel	1	0.983 / 0.196	0.934 / 0.196	0.933 / 0.171
		2	0.900 / 0.196	1.280 / 0.196	1.029 / 0.171
		3	1.169 / 0.196	0.900 / 0.196	1.041 / 0.171
		4	1.307 / 0.196	0.924 / 0.196	0.930 / 0.171
	Perpendicular	1	0.781 / 0.218	1.238 / 0.317	0.767 / 0.258
		2	0.801 / 0.218	1.142 / 0.317	0.790 / 0.258
		3	0.821 / 0.218	1.069 / 0.317	0.821 / 0.258
		4	0.871 / 0.218	1.141 / 0.317	0.844 / 0.258
	Vertical	1	0.654 / 0.147	0.583 / 0.117	0.370 / 0.107
		2	0.625 / 0.147	0.591 / 0.117	0.430 / 0.107
		3	0.631 / 0.147	0.590 / 0.117	0.485 / 0.107
		4	0.633 / 0.147	0.535 / 0.117	0.467 / 0.107

normal water level, and corresponding amplification between the top and bottom of the tower was about 3.67 times, well the largest amplification was about 4.41 times under the standard earthquake at normal water level. That in the direction perpendicular to flow was 2.33g due to site-specific earthquake without water, and corresponding amplification more than 5.5 times, the largest amplification in the direction was about 6.4 times under the same excitation but at low water level. Except for few cases, the responses decreased as the water level increased.

For the group of towers, the maximum accelerations at the top depend on the ground motions as well, and they are different from tower to tower. However, the maximum accelerations of the four towers in the direction perpendicular to flow tend to approach each other as the water level increases, which again confirm the effect of normal force transfer due to the water between the contraction joints of the towers. However, the phenomenon that the responses decrease as the water level increases in the single tower model was not obvious for the group of towers model.

Fig. 10 and Fig. 11 show the distribution of the maximum accelerations in elevation on the tower No. 2 for the single tower model and group of towers model, respectively. First, the shapes of the distribution are similar to their fundamental mode shapes of vibration in each direction, implying that the fundamental modes are dominant for the seismic responses. Furthermore, it can be identified easily from the figures that the amplification from the table surface to the tower base is close to 2 for both models in the direction parallel to flow in an average sense, but less than 1.2 in the direction perpendicular to flow. This is attributed to the different contour shapes in the cross section of the foundation rock in the two directions. The steep slope below the tower base in the direction parallel to flow causes the amplification. Therefore, it is better to leave more flat space at

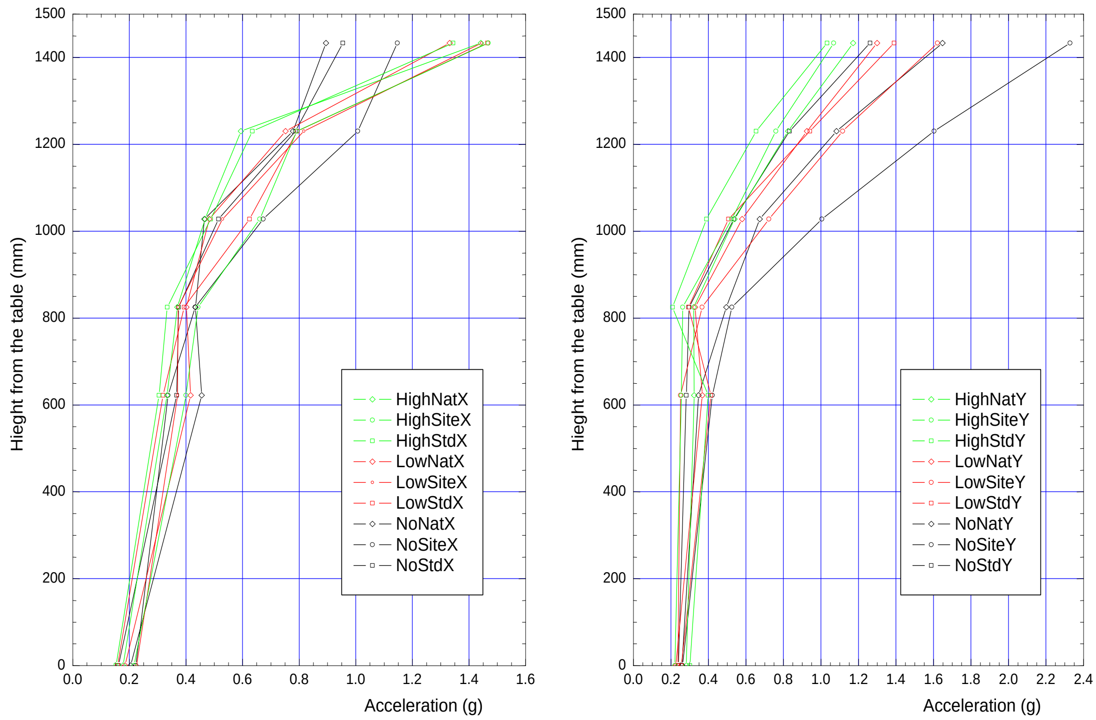

Fig. 10 Distribution of maximum response accelerations in elevation for the single tower model left: parallel to flow, right: perpendicular to flow

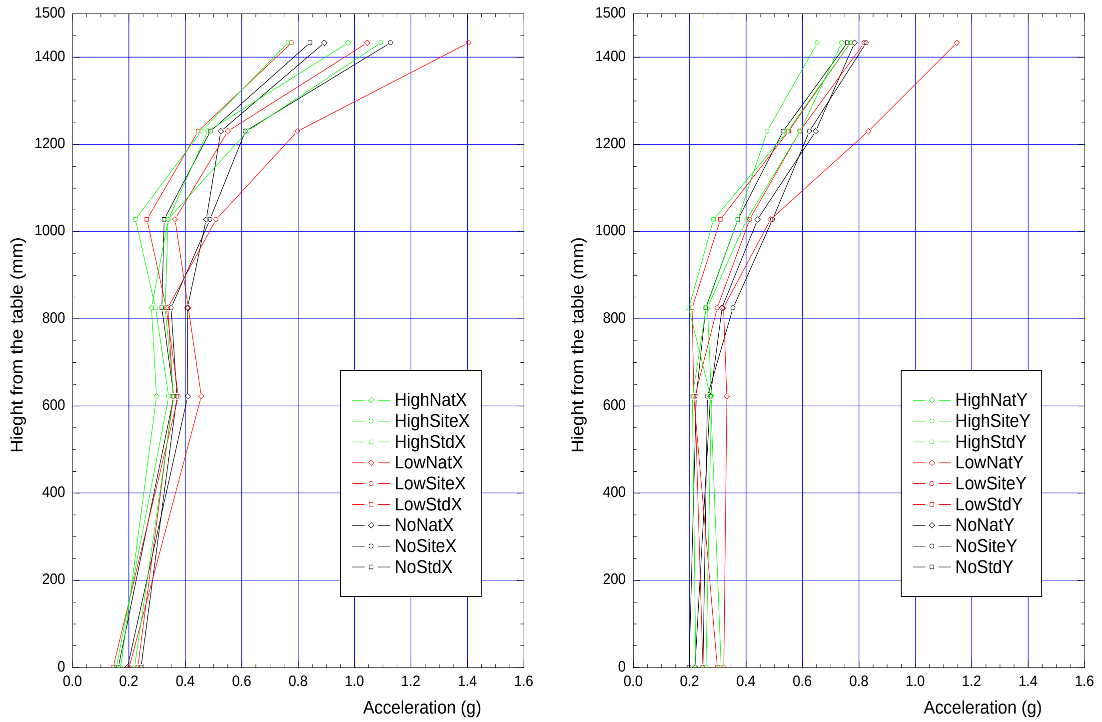

Fig. 11 Distribution of maximum response accelerations in elevation for the group of towers model left: parallel to flow, right: perpendicular to flow

the upstream side of the tower base in view of the seismic responses. The farther the towers from the steep slope, the smaller their seismic responses. Moreover, in general, the acceleration responses of the single tower model are larger than those of the group of towers model in both directions except for very few cases. It is hard to explain based on the difference in fundamental resonant frequencies of the tower since it is small in the direction perpendicular to flow.

4.2.2 Hydrodynamic pressure responses

The hydrodynamic pressure is a scalar. Therefore the measured values are the total contribution of structure-water interaction in all directions. All the data summarized here are under seismic excitations in three directions simultaneously.

In Table 5 are listed the maximum hydrodynamic pressures on every measured points for the single tower model. In Table 6 are listed those for the group of towers model, in which, P6*, P7* and P8* were hydrophones installed on the central slab of the tower No. 1, No. 3 and No. 4, respectively, similar to P3* on the tower No. 2.

First of all, hydrodynamic pressures show big difference under three sets of input seismic motions. This may be partially due to the different structural responses. The variations, however, become smaller at normal water level than low water level on outside points for both the single tower model tests and group of towers model tests, although no evidence shows that the acceleration responses get closer under different seismic inputs at normal water level. The pressures inside, that is on points P4* and P5*, display more differences than the pressures outside for the single tower model, which may be attributed to the structural motion perpendicular to the

Table 5 Maximum hydrodynamic pressures with the single tower model (Unit: kPa)

Location of sensor	Low level			Normal level		
	Standard	Site-Specific	Natural	Standard	Site-Specific	Natural
P11	--	--	--	33.0	37.7	28.4
P12	1.4	0.8	1.1	100.8	127.2	103.5
P13	77.3	65.2	88.2	105.0	130.1	106.7
P14	121.1	82.8	145.9	150.3	150.3	156.3
P15	158.0	124.1	185.8	172.3	162.6	173.9
P21	--	--	--	20.2	26.9	20.7
P22	0.8	1.9	1.0	81.3	111.7	75.9
P23	67.5	52.1	78.2	111.7	141.2	132.8
P24	109.7	77.9	132.1	137.1	150.1	140.5
P25	146.4	118.3	169.6	160.1	159.6	158.6
P31	--	--	--	44.1	55.9	46.3
P32	4.0	2.4	2.9	138.0	151.9	139.2
P33	109.0	87.5	109.6	140.7	170.4	151.7
P41	--	--	--	20.1	18.8	20.0
P42	2.8	2.7	2.9	183.3	114.8	220.9
P43	81.8	56.1	110.3	221.8	134.3	271.0
P51	--	--	--	11.7	9.4	17.3
P52	4.7	3.4	3.2	105.3	71.2	145.9
P53	84.3	67.7	120.2	173.6	144.0	202.9
P54	137.9	107.7	180.4	195.4	148.8	248.0

flow.

Regarding the tower No. 2, at points within 30m from the surface of water, hydrodynamic pressures were quite close for the single tower model and group of towers model at two water levels. For points deeper than 30 m, the pressures were about 40 to 55% larger for group of towers model than for the single tower model for both water levels, except for the pressures inside, points P4* and P5*. The pressures inside at normal water level were larger for the single tower model than for the group of towers under natural earthquake, it is reverse for all other cases.

At most load cases, the pressures of P54 were higher than those of P14 and P24 at the same elevation, which could be attributed to the stronger vibration of the slab there. Pressures P43 on the slab were higher than the P53 on the tower body for all but one case at normal water level, confirming the vibration of the slab again. Comparing the pressures inside and outside on the slab at normal water level, that is P32 and P33 with P42 and P43, the inside pressures were usually higher for both the single tower model and the group of towers model.

With respect to the pressures at different towers on the central slab for the group of towers model, the pressures on two mid towers were always higher than those on the two side towers, with the largest ratio of 1.54 and 2.67 at elevation of P*2 and P*3, respectively. The influence of the topography around the towers resulted in larger pressures on the left two towers than two right towers.

Table 6 Maximum hydrodynamic pressures with group of towers model (Unit: kPa)

Location of sensor	Low level			Normal level		
	Standard	Site-Specific	Natural	Standard	Site-Specific	Natural
P11	--	--	--	17.9	25.7	23.8
P12	0.9	2.3	3.1	78.7	106.2	96.5
P13	92.2	76.9	106.7	136.4	133.1	134.1
P14	163.1	139.9	207.6	203.4	195.7	167.4
P15	214.3	195.1	284.2	238.1	238.3	196.8
P21	--	--	--	17.2	21.1	14.2
P22	1.5	1.5	2.3	71.7	91.0	87.9
P23	86.6	73.9	105.1	130.6	131.9	126.9
P24	155.0	138.5	199.6	192.9	190.7	157.7
P25	200.0	187.2	264.2	226.8	229.5	183.7
P31	--	--	--	32.2	47.3	34.8
P32	2.3	2.8	2.4	100.4	122.2	113.1
P33	132.7	99.7	160.2	196.9	165.2	162.7
P41	--	--	--	9.2	10.2	10.7
P42	1.3	3.0	2.4	187.4	147.8	204.2
P43	112.4	97.2	168.9	224.3	191.7	251.1
P51	--	--	--	9.8	13.5	10.9
P52	3.7	4.2	1.5	107.2	78.6	108.5
P53	129.3	98.8	170.7	209.8	173.8	198.9
P54	194.4	157.7	241.8	233.3	211.6	214.2
P62	1.1	0.7	2.5	82.1	118.5	79.5
P63	77.1	77.4	87.8	121.4	129.0	112.0
P72	1.0	0.7	0.7	78.9	91.3	95.3
P73	80.8	88.1	103.7	121.8	151.8	131.9
P82	0.7	1.0	0.7	68.5	97.6	73.6
P83	45.8	56.0	67.5	73.9	99.1	87.0

4.2.3 Strain responses

Dynamic strains were measured at the base and beams of the tower No. 2. For the tests with the single tower model, the largest dynamic strains at the base and on the beams were recorded without water for same seismic inputs at nearly all measured points. The strains at normal water level may be larger or smaller than low water level depending on the seismic inputs.

For the tests with group of towers model, the strain responses were smaller than the single tower test for all water level and all seismic inputs except at very few measured points. The strains for the cases without water decreased more than 50%. The main reason is the smaller seismic responses for group of towers in the direction perpendicular to flow, refer to Fig. 10 and Fig. 11. The largest dynamic strains at the base and on the beams were recorded at normal or low water level for the same seismic inputs in general.

5. Conclusions

The dynamic interaction between a group of intake towers, the water and the foundation system was studied by means of a scaled model on the shaking table. And tests with only a single intake tower were carried out for comparison.

The interaction between the intake towers themselves as well as that with the foundation reduces the seismic responses of the tower considerably. Water-tower interaction decreases the resonant frequencies of the tower in all directions for the single tower, but seems to have no influence on the resonant frequencies in the direction perpendicular to flow for the group of towers due to the normal force transfer role of the water in the contraction joints between towers. The role of normal force transfer also made the seismic responses of all towers closer to each other.

Hydrodynamic pressures measured in the group of towers tests are larger than in the single tower tests 30m below the water surface for the outside of towers. In the group of towers tests, two mid towers experienced higher pressure than two side towers, and the deeper, the larger the differences. Besides, the inside hydrodynamic pressures are more dependent on ground motions than the outside.

In the tests with the group of towers model, the strain responses were smaller than in the single tower tests for all water level and all seismic inputs except at very few measured points. The strains for the cases without water decreased more than 50%.

The seismic responses of group of intake towers should be analyzed with all the towers to take into account of the dynamic interaction between towers themselves and between towers and foundation. Not only the water surrounding the towers, but also the water in the contraction joints between towers affects both the dynamic characteristics as well as seismic responses of the towers. How to simulate the normal force transfer role of the joint water in numerical analysis requires further investigation.

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