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Soil–structure–water interaction of intake–outlet towers allowed to uplift

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A procedure that can be used for preliminary seismic analysis of intake–outlet towers including soil–structure–water interaction is developed. The formulation also considers the effect of partial soil–foundation separation, a phenomenon of considerable interest for the design of such systems that has not been addressed in the literature. The hydrodynamic pressure of the water is accounted for through added masses given in concise closed-form expressions for easy use in analysis and design. The nonlinear equations of motion, based on foundation–soil bond conditions, are solved numerically.

The effects of soil–structure–water interaction are evaluated for a representative intake–outlet tower. Parametric studies are also conducted in the presence and absence of surrounding and contained water for typical cases of soil conditions and tower height–foundation width ratios. The results indicate that hydrodynamic effects are significant and cause an increase in deflections, moments and shears and a decrease in foundation rotation. The study shows that for short towers, foundation uplift is unlikely to occur. On the contrary for slender towers, uplift is more likely to appear, especially for foundations supported by stiff soil, causing significant decrease in moments and deflections.

INTRODUCTION

The seismic analysis of structures such as intake–outlet towers is rather involved. Interaction of the tower with soil and water result in modification of the system's dynamic properties, which in turn alter its seismic response. Therefore, seismic studies of intake–outlet towers should incorporate the effects of fluid–structure as well as soil–structure interaction.

Considerable work has been carried out on the subject of soil–structure interaction of towers. Earlier work concentrated on the overturning of objects such as furniture, equipment and inverted pendulum type systems in which both the structure and soil were considered as rigid and the structure was bonded to the soil through gravity.[1–4] In later studies related to the seismic analysis of tall slender structures, flexibility of both the structure and the soil were incorporated.[5–8] Clear evidence of foundation uplift has been observed during strong earthquakes.[2] Foundation uplift could dramatically alter the seismic response of a structure, usually leading to reduction of structural deformation and forces for long period structures and with an opposite result for short period systems. Several researchers have included the effects of foundation uplift in their work.[9–12] Results have indicated that no definite conclusion could be drawn regarding whether or not soil–structure interaction and foundation uplift are beneficial.[9,13]

Rigorous analysis of seismic soil–structure–water interaction of intake–outlet towers leads to rather complex boundary value problems that require involved treatments when the physical parameters such as the unbounded extent, compressibility and surface waves of the water, the geometry and flexibility of the structure and the stochastic property of the earthquake ground excitation are considered. Reviews of studies on structure–water interaction could be found in several papers and reports.[14–16]

For preliminary analysis and design of hydraulic structures, simplified procedures have been proposed to simulate the hydrodynamic effects of structure–water interaction. Such a procedure is the so-called 'added mass' approach, first proposed by Westergaard[17] in 1933. It was later extended by Liaw and Chopra[15] to calculate the added hydrodynamic mass of the water for a uniform cylindrical intake tower, assuming incompressive water and no surface waves. Goyal and Chopra[18,19] extended their previous work to evaluate the added mass of surrounding and contained water for towers with an arbitrary cross-section that varies along the height and has two axes of symmetry.

In this work the added mass procedure is further enhanced. The hydrodynamic effect of the surrounding and contained water are accounted for through simple closed form expressions that provide the 'added-mass'. Soil–structure interaction including foundation uplift is also considered, a phenomenon that to the authors' knowledge has not been addressed but could be of considerable concern for the design of intake–outlet towers subjected to strong ground motions.[20] Specifically, the tower is modeled as a cantilever beam with flexural deflection expressed through an assumed modes method. A spring–dashpot system attached to a foundation allowed to uplift is supporting the tower. The nonlinear equations of motion obtained via Langrange's equation applied at the contact and uplift phases are solved numerically. The significance of the salient physical parameters of the tower–soil–water system on its seismic response is examined through parametric studies.

TOWER–SOIL–WATER SYSTEM

As shown in Fig. 1(a), the structure is a slender intake–outlet tower submerged in water with a height H. The depths of surrounding water h_0 and contained water h_i can be different. The tower is idealized as a homogeneous isotropic tapered beam with a distributed moment of inertia $I(z)$ and mass per unit length $m(z)$. The interaction of water and structure is treated by the added mass method which is described in the next section. Applying the added mass method, the total mass per unit length, $m_i(z)$, can be expressed as the sum of three parts

$$m_t(z) = m(z) + m_0(z) + m_i(z) \tag{1}$$

where $m_0(z)$ and $m_i(z)$ are the added masses of the surrounding and contained water, respectively. The total mass of the tower is denoted as M_b. The operation unit at the top of the tower has a lumped mass M_0. The tower is connected to a rigid circular foundation with height h_f and radius R. The total mass and mass moment of inertia of the foundation are M_f and I_f, respectively. The soil supporting the foundation is modeled as a two spring–dashpot system connected to the foundation as shown in Fig. 1(b). Even though the stiffness and damping coefficients of the foundation–soil system are frequency dependent, in this study they are approximated by the expressions given in Table 1, an assumption which would be sufficient for preliminary analysis.[6,21] In the expressions, G_s, ν_s, and ρ_s are the shear modulus, Poisson's ratio and mass density of the soil, respectively.

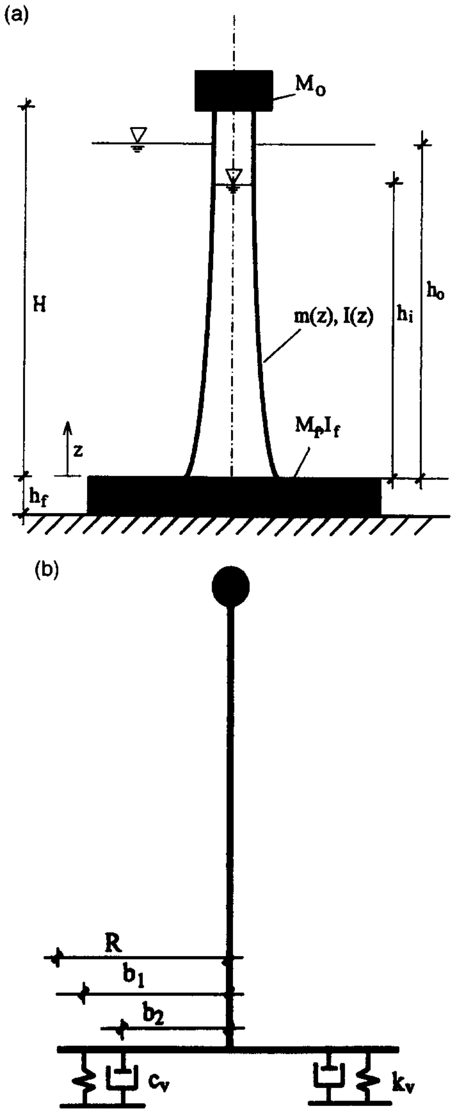

Fig. 1. (a) Intake–outlet tower system; (b) simplified model.

Table 1. Coefficients of stiffness and damping

Stiffness		Damping	
Vertical k_v	Rocking k_ϕ	Vertical c_v	Rocking c_ϕ
$\dfrac{4G_sR}{1-\nu_s}$	$\dfrac{8G_sR^3}{3(1-\nu_s)}$	$\dfrac{3\cdot 2}{1-\nu_s}\rho_s c_s R^2$	$\dfrac{0\cdot 4}{2-\nu_s}\rho_s c_s R^4$

To consider the effect of foundation uplift, the vertical and rocking springs have been replaced by a pair of vertical springs placed at a distance b_1 to provide a system that develops equal resistance of the vertical and rocking springs as shown in Fig. 1(b). The distance b_1 is equal to[22]

$$b_1 = \sqrt{\frac{2}{3}}R. \tag{2}$$

In the same manner, a pair of vertical dampers has also been placed at a distance b_2 to simulate the effects of the vertical and rocking dampers, the distance b_2 is determined as

$$b_2 = \sqrt{\frac{1}{8}}R. \tag{3}$$

In Fig. 1(b), the values of the equivalent vertical springs, k_v and dampers, c_v are half of the k_z and c_z, respectively.

The vertical component of the ground motion is neglected and only the horizontal ground motion, $\ddot{u}_g(t)$, is applied to the system. Further, it is assumed that the frictional force between the foundation and the soil is large enough to prevent horizontal slippage. The inertia effects are considered by adding D'Alembert's forces to the superstructure. The displacement and force configuration is shown in Fig. 2, where $u(z,t)$ is the deflection of the tower and $\theta(t)$ is the foundation rotation. Prior to the earthquake excitation, the foundation rests on the spring–dashpot system through gravity causing a vertical displacement of the soil v_{st}. For small displacements, the relative displacement $U(z,t)$ with respect to the ground is the summation of the tower deflection $u(z,t)$ and the displacement due to the rotation of the foundation $z * \theta(t)$, see Fig. 2. The assumed-modes method is employed to approximate the deflection of the tower,[23,24] that is,

$$u(z,t) = \sum_{i=1}^{N} \phi_i(z) q_i(t) \tag{4}$$

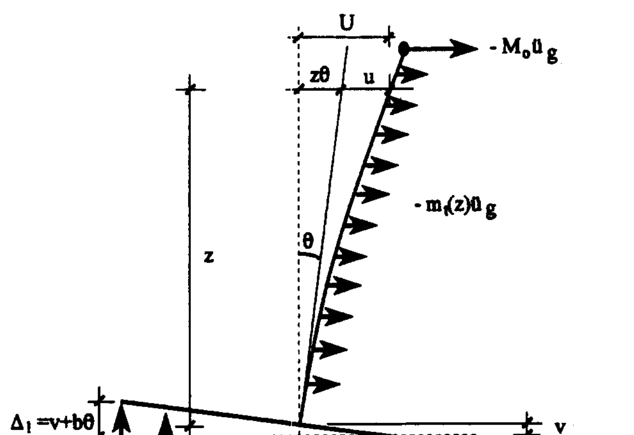

Fig. 2. Displacements and forces.

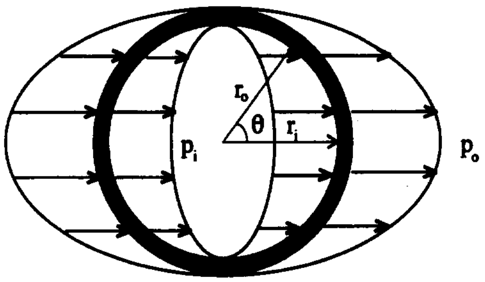

Fig. 3. Hydrodynamic pressure.

where the lowest N-modes of a uniform cantilever beam $\phi_i(z)$ are selected as assumed-modes for the deflection and $q_i(t)$ are generalized coordinates. Consequently, the system shown in Fig. 2 is characterized by a set of $N+2$ generalized coordinates $q_i(t)(I = 1,\ldots,N)$, $\theta(t)$ and $v(t)$, where $v(t)$ is the vertical response from the position of equilibrium.

TREATMENT OF HYDRODYNAMIC EFFECTS

The seismic response of intake–outlet towers is significantly influenced by the water through hydrodynamic forces acting during the excitation. For preliminary analysis, the added mass method has been successfully employed to approximate the interaction of a structure with water.[22,25] Assuming incompressive water, the governing equation for the hydrodynamic pressure, $p(r,\theta,z,t)$, in cylindrical coordinates is expressed as

$$\frac{\partial^2 p}{\partial r^2} + \frac{1}{r}\frac{\partial p}{\partial r} + \frac{\partial^2 p}{\partial z^2} = 0 \tag{5}$$

Figure 3 depicts variation of contained water pressure p_i and surrounding water pressure p_o for a cross-section of the tower.[15] For a cylindrical tower, the boundary conditions for the water specify: (1) no vertical motion at the boundary $z = 0$, when subjected only to horizontal motion; (2) no surface waves at $z = h_i$ and $z = h_o$, since sloshing is not important for slender towers and can be neglected; (3) compatibility of the water and tower displacements at $r = r_o$ and $r = r_p$ where $r = r_o$ and $r = r_i$ are the outside and inside radii, respectively; (4) symmetry at the plane $\theta = 0$. The corresponding boundary conditions are given by

$$\frac{\partial p}{\partial z} = 0$$

$$p(r,\theta,0,t) = 0$$

$$\frac{\partial p}{\partial r}\Big|_{r=r^*} = m\ddot{u}n_i$$

$$\frac{\partial p}{\partial \theta}\Big|_{\theta=0} = \frac{\partial p}{\partial \theta}\Big|_{\theta=\pi} \tag{6}$$

where $r^* = r_o$ for surrounding water and $r^* = r_i$ for contained water and n_i is the outward normal from the surface of the tower towards the water.

Further it is assumed that the tower distorts only in its fundamental flexural mode of vibration. Thus, the hydrodynamic effects can be simulated through a distributed added mass along the height of the tower. The expressions of the added mass of water are given in series form[15] for the surrounding water

$$m_a^{\mathrm{o}}(z) = m_{\infty}^{\mathrm{o}}\left[\frac{16}{\pi^2}\frac{h_{\mathrm{o}}}{r_{\mathrm{o}}}\sum_{m=1}^{\infty}\frac{(-1)^{m-1}}{(2m-1)^2}E_m \times\left(\alpha_m\frac{r_{\mathrm{o}}}{h_{\mathrm{o}}}\right)\cos\left(\alpha_m\frac{z}{h_{\mathrm{o}}}\right)\right] \tag{7}$$

and

$$m_a^{i}(z) = m_{\infty}^{i}\left[\frac{16}{\pi^2}\frac{h_i}{r_i}\sum_{m=1}^{\infty}\frac{(-1)^{m-1}}{(2m-1)^2}D_m \times\left(\alpha_m\frac{r_i}{h_i}\right)\cos\left(\alpha_m\frac{z}{h_i}\right)\right] \tag{8}$$

for the contained water, where $\alpha_m = (2m-1)\pi/2$. The $m_{\infty}^{\mathrm{o}} = \rho_w\pi r_{\mathrm{o}}^2$ and $m_{\infty}^{\mathrm{I}} = \rho_w\pi r_i^2$ are the added mass of surrounding and contained water per unit length for an infinitely long tower with a constant cross-section with ρ_w denoting the mass density of water and

$$E_m\left(\alpha_m\frac{r_{\mathrm{o}}}{h_{\mathrm{o}}}\right) = \frac{K_1\left(\alpha_m\frac{r_{\mathrm{o}}}{h_{\mathrm{o}}}\right)}{K_0\left(\alpha_m\frac{r_{\mathrm{o}}}{h_{\mathrm{o}}}\right)+K_2\left(\alpha_m\frac{r_{\mathrm{o}}}{h_{\mathrm{o}}}\right)} \tag{9}$$

$$D_m\left(\alpha_m\frac{r_i}{h_i}\right) = \frac{I_1\left(\alpha_m\frac{r_i}{h_i}\right)}{I_0\left(\alpha_m\frac{r_i}{h_i}\right)+I_2\left(\alpha_m\frac{r_i}{h_i}\right)} \tag{10}$$

in which I_n and K_n are the first and second kind modified Bessel functions of order n.

Figure 4 shows the normalized added mass of contained water, $m_a^i(z)/m_{\infty}^i$, computed from eqn (8) for a tower with a constant radius r, when $h_i/r_i = H_r = 10$ and $m = 10$, 20 and 50, respectively. The convergence is slow and the computation of the Bessel functions is rather cumbersome. Equations (7) and (8) have been simplified through curve fitting techniques to arrive at concise and easy to use expressions for the added masses[22] to obtain

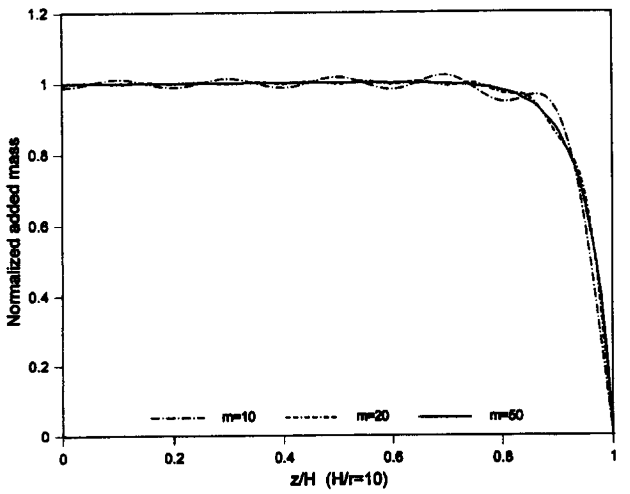

Fig. 4. Normalized added mass for contained water with varied summation terms.

Table 2. Coefficients of a_0, b_0, c_0

When $h_{\mathrm{o}}/r_{\mathrm{o}} \le 3$	Otherwise
$a_{\mathrm{o}} = 27\frac{h_{\mathrm{o}}}{r_{\mathrm{o}}} - 11$	$a_{\mathrm{o}} = 65\frac{h_{\mathrm{o}}}{r_{\mathrm{o}}} - 125$
$b_{\mathrm{o}} = 26{\cdot}96\frac{h_{\mathrm{o}}}{r_{\mathrm{o}}} - 10{\cdot}98$	$b_{\mathrm{o}} = 65{\cdot}198\frac{h_{\mathrm{o}}}{r_{\mathrm{o}}}0{\cdot}033\left(\frac{h_{\mathrm{o}}}{r_{\mathrm{o}}}\right)^2 - 125{\cdot}397$
$c_{\mathrm{o}} = 0{\cdot}031 + 0{\cdot}058\frac{h_{\mathrm{o}}}{r_{\mathrm{o}}}0{\cdot}009\left(\frac{h_{\mathrm{o}}}{r_{\mathrm{o}}}\right)^2$	$c_{\mathrm{o}} = 0{\cdot}1074 + 0{\cdot}0047\frac{h_{\mathrm{o}}}{r_{\mathrm{o}}}$

(a) surrounding water

$$m_a^{\mathrm{o}}(z) = C_{\mathrm{o}}m_{\infty}^{\mathrm{o}}\ln\left(\frac{\sqrt{1+\left(a_{\mathrm{o}}\frac{z}{h_{\mathrm{o}}}\right)^2}+b_{\mathrm{o}}\frac{z}{h_{\mathrm{o}}}}{\sqrt{1+\left(a_{\mathrm{o}}\frac{z}{h_{\mathrm{o}}}\right)^2}-b_{\mathrm{o}}\frac{z}{h_{\mathrm{o}}}}\right) \tag{11}$$

where, a_{o}, b_{o}, c_{o} are given in Table 2.

(b) contained water

$$m_a^{\mathrm{i}}(z) = C_i m_{\infty}^{\mathrm{i}}\ln\left(\frac{\sqrt{1+\left(a_i\frac{z}{h_i}\right)^2}+b_i\frac{z}{h_i}}{\sqrt{1+\left(a_i\frac{z}{h_i}\right)^2}-b_i\frac{z}{h_i}}\right) \tag{12}$$

where, a_i, b_i, c_i are given in Table 3.

A comparison between the simplified eqns (11) and (12) and the exact formulae of eqns (7) and (8) for m = 50 and a tower with a constant radius r is shown in Fig. 5 (a) and (b) for $h_i/r_i = H/_r$. The comparison shows that both sets of equations give close results.

GOVERNING EQUATIONS AND NUMERICAL TREATMENT

Lagrange's equations are employed to derive the equations

Table 3. Coefficients of a_i, b_i, c_i

When $h_i/r_i \le 3$	Otherwise
$a_i = 10\frac{h_i}{r_i}$	$a_i = 41{\cdot}54 - 11{\cdot}033\frac{h_i}{r_i} + 2{\cdot}6\left(\frac{h_i}{r_i}\right)^2$
$b_i = 10{\cdot}92\frac{h_i}{r_i} - 0{\cdot}46\left(\frac{h_i}{r_i}\right)^2 - 0{\cdot}508$	$b_i = 30$
$c_i = 0{\cdot}113\frac{h_i}{r_i} - 0{\cdot}008\left(\frac{h_i}{r_i}\right)^2 + 0{\cdot}034$	$c_i = 0{\cdot}135 - 0{\cdot}05\frac{h_i}{r_i} + 0{\cdot}035\left(\frac{h_i}{r_i}\right)^2$

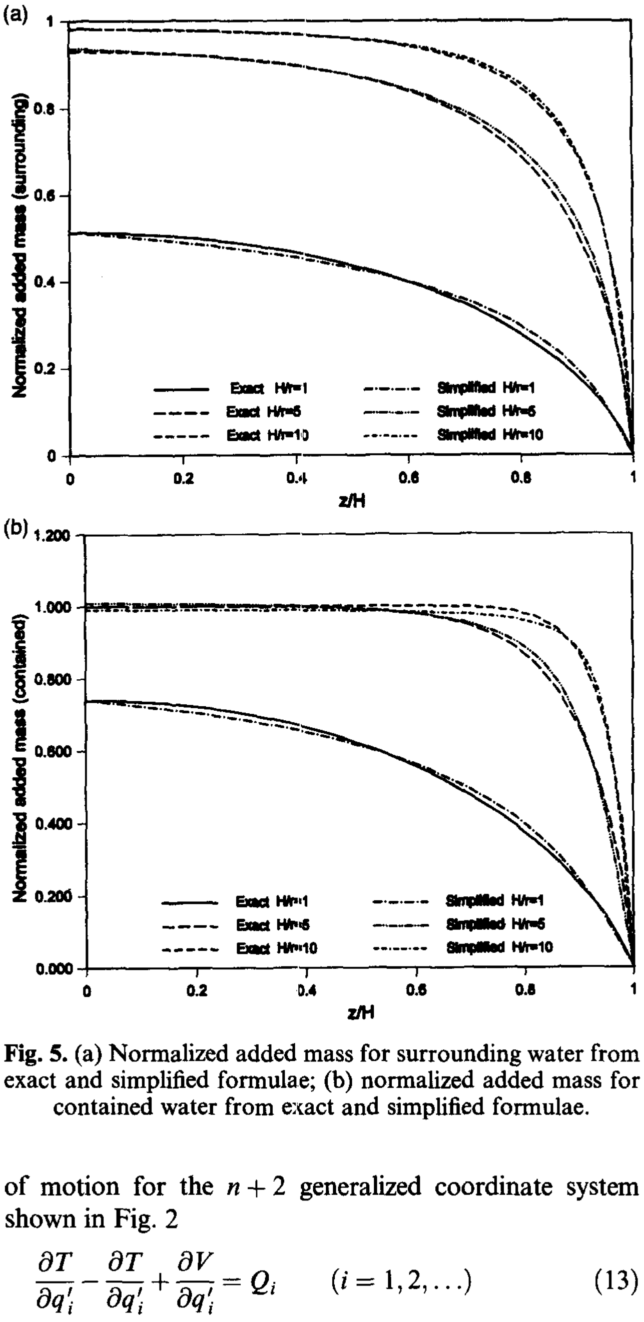

Fig. 5. (a) Normalized added mass for surrounding water from exact and simplified formulae; (b) normalized added mass for contained water from exact and simplified formulae.

of motion for the $n+2$ generalized coordinate system shown in Fig. 2

$$\frac{\partial T}{\partial q_i'} - \frac{\partial T}{\partial q_i'} + \frac{\partial V}{\partial q_i'} = Q_i \qquad (i = 1, 2, \ldots) \tag{13}$$

where T and V denote the kinetic and potential energy, respectively, Q_i are the generalized forces and q_i' corresponds to $q_i(t)(I = 1, \ldots, N)$, $\theta(t)$ and $v(t)$. A detailed presentation of the derivation presented by Xu,[22] where the formulation is developed under the assumption of small displacements and rotation and neglecting $P - \Delta$ effects. Depending on contact conditions, the governing equations take different forms to express three phases: no foundation uplift, left edge uplifted and right edge uplifted. For simplicity, they are expressed in matrix form

$$|\mathbf{M}|\{\ddot{\mathbf{x}}\} + |\mathbf{C}|\{\dot{\mathbf{x}}\} + |\mathbf{K}|\{\mathbf{x}\} = \{\mathbf{Q}\} \tag{14}$$

The matrices [**M**], [**K**] and [**C**] and vectors are given in Appendix I. Due to the complexity of the integrands, the integrations indicated in Appendix I have been performed numerically.

The equations of motion, a set of piecewise linear differential equations of second-order, have been solved with Newmark's direct integration method for $\delta = 0{\cdot}5$ and $\alpha = 0{\cdot}25$, e.g. Bathe.[26] In the direct integration method the selection of time step Δt is of critical importance to solution accuracy. Detail discussions and rules to select a time step Δt can be found in several references, e.g. Spyrakos.[27] For the representative examples, it is found that a time step of 0·0005 s provides sufficient accuracy.[22] The accuracy also depends on the number of modes included in the formulation. A comparative study indicates that a five-mode formulation maintains good balance between solution accuracy and computer processing time.[22]

NUMERICAL EXAMPLES

As an illustrative example, the Briones Dam intake–outlet tower located east of the San Francisco Bay is

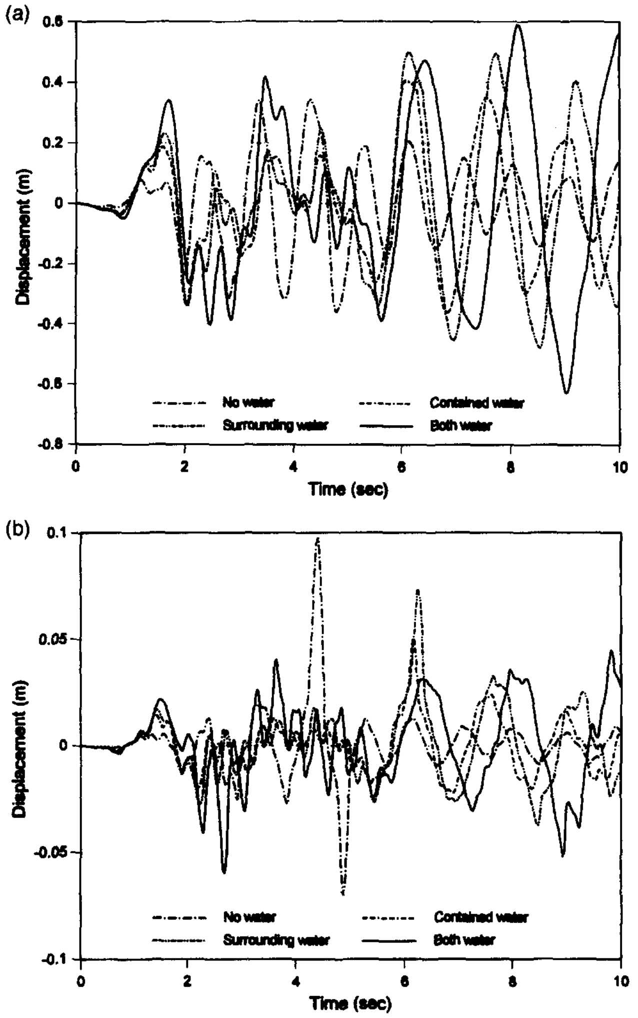

Fig. 6. (a) Deflection at the top; (b) Displacement due to foundation rotation.

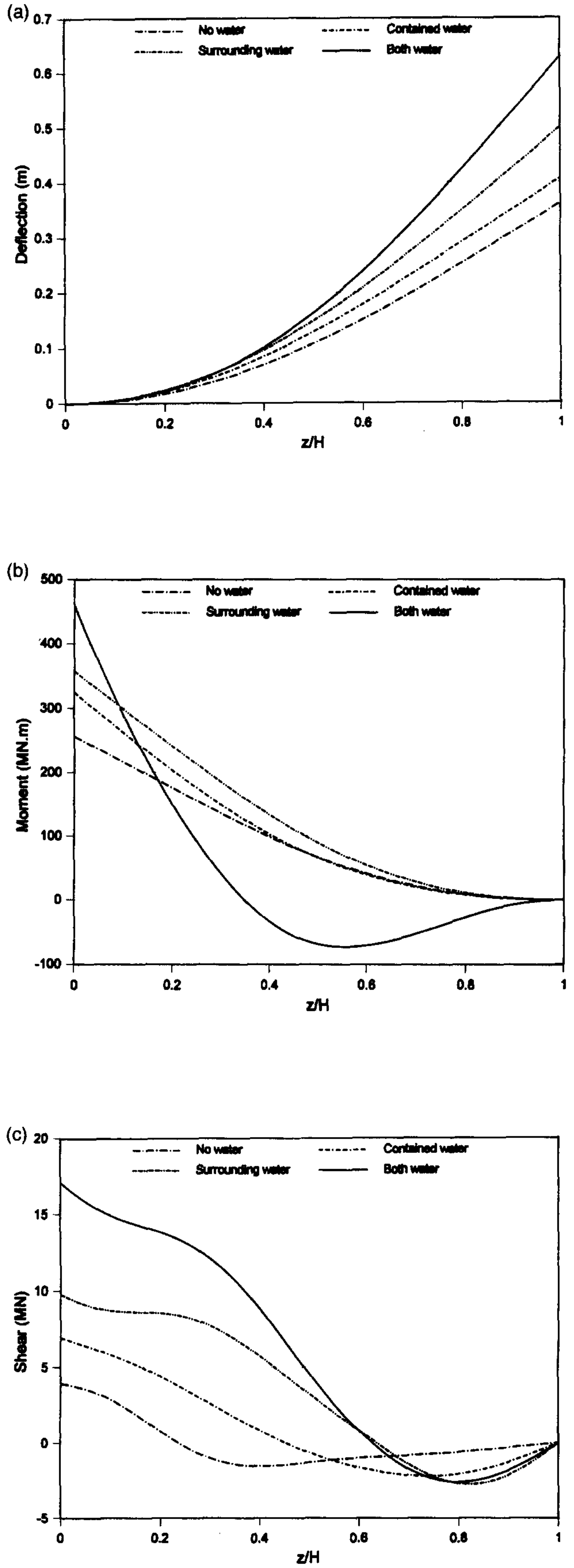

Fig. 7. (a) Deflection along the tower height; (b) moment along the tower height; (c) shear along the tower height.

studied.[18] The radius for the tower varies linearly along the height. Its cross-sectional parameters are: top inner radius = 1·52 m, top outer radius = 1·86 m, bottom inner radius = 3·05 m and bottom outer radius = 3·45 m. The height of the tower is 70·10 m and the radius of the foundation is 9·14 m.

The reinforced concrete tower is considered to be homogeneous, isotropic, linear elastic with Young's modulus $E = 31{,}000$ MPa and unit weight = 2483 kg/m^3. The damping ratio for each mode, i.e. $\xi_i (i = 1, \ldots, 5)$ is set equal to 0·05. The material properties of the supporting soil are: mass density = 2644 kg/m^3, shear modulus $G = 245{\cdot}6$ MPa, Poisson's ratio = 1/3 and shear wave velocity = 304·8 m/s.

The N–S component of the 1940 El-Centro earthquake ground motion is selected as the excitation. The maximum amplitude of this ground motion is 0·33 g. In order to amplify the effects of uplift, the amplitude of the ground motion has been increased by a factor of two, an order of magnitude that has been recorded in several seismic motions.

EFFECTS OF WATER–STRUCTURE INTERACTION

In order to examine the effects of hydrodynamic forces with uplift permitted, the seismic response of the tower is determined for the following four cases: (1) no water; (2) surrounding water only; (3) contained water only; and (4) both surrounding and contained water. In cases (2), (3), and (4) $h_0/H = 1$ and $h_i/H = 1$. The deflection and displacement due to foundation rotation at the top of the tower are shown in Figs 6(a) and (b). The deflection, moment and shear for the corresponding maximum responses are presented in Figs 7(a)–(c). Table 4 lists the maximum top displacement, deflection, displacement due to foundation rotation and maximum base moment and shear. Figures 6 and 7 and Table 4 demonstrate that the presence of water greatly increases the seismic response of the tower. This is attributed to two factors, that is, the decrease of the natural frequencies of the tower–water system due to the presence of water and the Fourier amplitude spectrum of the N–S component of the El-Centro earthquake which is characterized by large amplitudes for frequencies less than 5 Hz.[28] It is worth mentioning that when both surrounding and contained water are present, the base shear is four times of that with no water included, while the top deflection and base moment are almost doubled. Notice however, that the displacement due to foundation rotation is greatly reduced.

To evaluate the effects of water–structure interaction for soil conditions and foundation widths other than the Briones Dam tower, parametric studies have been conducted. Two typical soil conditions are considered. They correspond to soft and hard soil with shear moduli $G = 34{\cdot}5$ and 245·6 MPa, respectively. Two tower

Table 4. Seismic response of Briones Dam tower

Experimental cases	Displacement (cm)	Deflection (cm)	Displ. due to rotation (cm)	Moment (MN.m)	Shear (MN)
No water	41·27	36·24	9·82	256·60	3·93
Contained water	44·87	40·77	5·16	325·30	6·93
Surrounding water	53·90	50·04	7·34	358·30	9·76
Both water	65·64	62·95	5·98	459·00	17·05

Table 5. Response for various soil conditions and H/R ratios

G MPa	H/R	Hydrodynamic forces	Displacement (cm)	Deflection (cm)	Displ. due to rotation (cm)	Moment (MN.m)	Shear (MN)
245·6	7·7	Water	65·64	62·95	5·98	459·00	17·05
		No water	41·27	36·24	9·82	256·60	3·93
	20	Water	185·30	55·84	150·60	209·60	14·44
		No water	66·40	17·60	56·02	94·44	2·52
34·5	7·7	Water	100·50	72·41	36·26	443·30	15·75
		No water	40·97	29·06	12·25	186·40	2·84
	20	Water	137·60	41·87	127·70	107·10	11·61
		No water	178·70	15·24	167·80	87·07	2·32

height to foundation width ratios (H/R) for a short and a slender tower with $H/R = 7{\cdot}7$ and 20, respectively, have been examined. The maximum response for seismic analyses are listed in Table 5 for surrounding and contained water included and excluded in the analysis. The interaction of water–structure significantly influences the seismic response of the tower for all cases. The effect of water–structure interaction is greater on shear than that on moment and deflection. This behavior is attributed to the lowering of the higher natural frequencies leading to higher amplitudes since the selected seismic record is characterized by energy concentrated at frequencies lower than 5 Hz. The significance of the role that soil–structure interaction (SSI) plays on the response is also greatly affected by the presence or absence of water. Table 5 demonstrates that water–structure interaction decreases the effect of SSI on squat towers but increases the effect of SSI on slender towers.

EFFECTS OF FOUNDATION UPLIFT

In this section the effects of foundation uplift of the intake–outlet tower are studied by comparing the responses obtained by either allowing or constraining uplift. In both cases, the surrounding and contained water is included. In order to examine the effect of foundation uplift, the same tower height to foundation width ratios and soil stiffnesses that had been used in the parametric study listed in Table 5 were considered again.

The results of the parametric study are presented in Table 6. Notice that no uplift is observed when the soil stiffness is small, regardless the tower height–width ratio. Whereas, an increase of soil stiffness increases the likelihood of uplift. As the relative stiffness between the supporting soil and structure increases, i.e. the case of a slender tower on stiff soil, the beneficial effects of uplift in decreasing the moment and deflection become more pronounced.

For the Briones Dam tower which has $G = 245{\cdot}6$ MPa and $H/R = 7{\cdot}7$, the effect of the foundation uplift is not significant. The maximum values of the top displacement, deflection, displacement due to foundation rotation,base moment, and base shear are listed in the first two lines of Table 6. The responses are practically identical, which indicates that the magnitude of uplift is very small and can be ignored in the analysis of this

Table 6. Response under various bonded conditions

G (MPa)	H/R	Contact conditions	Displacement (cm)	Deflection (cm)	Displ. due to rotation (cm)	Moment (MN.m)	Shear (MN)
245·6	7·7	Uplifted	65·64	62·95	5·98	459·00	17·05
		No uplift	66·38	63·74	4·44	478·60	18·01
	20	Uplifted	185·30	55·84	150·64	209·60	14·44
		No uplift	190·30	102·40	101·80	595·90	16·12
34·5	7·7	No uplift occurs					
		No uplift	100·50	72·41	36·26	443·30	15·75
	20	No uplift occurs					
		No uplift	137·60	41·87	127·70	107·10	11·61

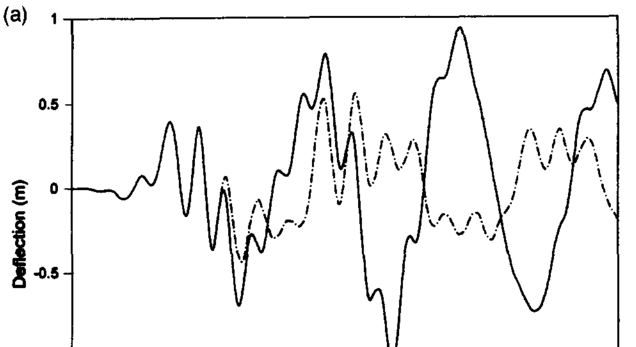

Fig. 8. (a) Deflection at the top; (b) displacement at the top due to foundation rotation.

tower, but if the H/R ratio is increased from 7·7 to 20, the effect of the uplift becomes more significant. The top deflection and displacement due to foundation rotation are shown in Figs 8(a) and (b). Clearly, the foundation uplift greatly increases the foundation rotation, while decreasing the deflection of the tower. The maximum values of the top displacement, deflection, displacement due to foundation rotation, base moment and base shear are also listed in Table 6. Notice the 2/3 and 1/2 decrease of moment and deflection, respectively, and the 1/3 increase of foundation rotation, due to foundation uplift. The base shear and top displacement remain almost the same.

CONCLUSIONS

The seismic response of an intake–outlet tower subjected to earthquake excitation including the effects of water–structure interaction, soil–structure interaction and foundation uplift is determined by a simplified procedure which can be used for preliminary analysis and design. Parametric studies are carried out to investigate the significance of soil–structure–water interaction and foundation uplift. The conclusions can be summarized as follows:

In this application it is shown that the interaction of water–structure greatly increases the seismic response. Increase of as much as two to four times the base shear, moment and top displacement can be observed. The presence of water affects the significance of SSI on the response. Specifically, it decreases the effect of SSI on short towers while it has the reverse effect on slender towers.

The influence of foundation uplift is greatly affected by the soil stiffness and the slenderness of the tower. It is most significant for slender towers on relatively stiff soil conditions. Decrease of moment and deformation of more than 50% by uplift were observed.

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APPENDIX I

$$\{\mathbf{x}\} = \{q_1(t)q_2(t), \dots q_N(t)\theta(t)v(t)\}^{\mathrm{T}}$$

$$\{\mathbf{Q}\} = -\ddot{u}_g\{M_1^\phi + M_o\ M_2^\phi + M_o \dots M_N^\phi + M_o\ M_z + M_o h\ (M_o + M_b + M_f)g/\ddot{u}_g\}^{\mathrm{T}}$$

$$[\mathbf{M}] = \begin{bmatrix} M_{11}^\phi + M_o & M_{12}^\phi + M_o & ,\dots, & M_{1N}^\phi + M_o & M_{1\phi}^z + M_o h & 0 \\ M_{21}^\phi + M_o & M_{22}^\phi + M_o & ,\dots, & M_{2N}^\phi + M_o & M_{2\phi}^z + M_o h & 0 \\ \vdots & \vdots & & \vdots & \vdots & \vdots \\ M_{N1}^\phi + M_o & M_{N2}^\phi + M_o & ,\dots, & M_{NN}^\phi + M_o & M_{N\phi}^z + M_o h & 0 \\ M_{\phi 1}^z + M_o h & M_{\phi 2}^z + M_o h & ,\dots, & M_{\phi N}^z + M_o h & M_z^z + M_o h^2 + I_f & 0 \\ 0 & 0 & ,\dots, & 0 & 0 & M_o + M_b + M_f \end{bmatrix}$$

$$[\mathbf{K}] = \begin{bmatrix} K_{11}^\phi & K_{12}^\phi & ,\dots, & K_{1N}^\phi & 0 & 0 \\ K_{21}^\phi & K_{22}^\phi & ,\dots, & K_{2N}^\phi & 0 & 0 \\ \vdots & \vdots & & \vdots & \vdots & \vdots \\ K_{N1}^\phi & K_{N2}^\phi & ,\dots, & K_{NN}^\phi & 0 & 0 \\ 0 & 0 & ,\dots, & 0 & \epsilon_1 k_v b^2 & \epsilon_2 k_v b \\ 0 & 0 & ,\dots, & 0 & \epsilon_2 k_v b & \epsilon_1 k_v \end{bmatrix}$$

and

$$[\mathbf{C}] = \begin{bmatrix} C_{11}^\phi & C_{12}^\phi & ,\dots, & C_{1N}^\phi & 0 & 0 \\ C_{21}^\phi & C_{22}^\phi & ,\dots, & C_{2N}^\phi & 0 & 0 \\ \vdots & \vdots & & \vdots & \vdots & \vdots \\ C_{N1}^\phi & C_{N2}^\phi & ,\dots, & C_{NN}^\phi & 0 & 0 \\ 0 & 0 & ,\dots, & 0 & \epsilon_1 c_v b^2 & \epsilon_2 c_v b \\ 0 & 0 & ,\dots, & 0 & \epsilon_2 c_v b & \epsilon_1 c_v \end{bmatrix}$$

where ϵ_1, ϵ_2 are given by

$$\epsilon_1 = \begin{cases} 2 & \text{no uplift} \\ 1 & \text{uplifted} \end{cases}$$

$$\epsilon_2 = \begin{cases} 1 & \text{right side uplifted} \\ 0 & \text{no uplift} \\ 1 & \text{left side uplifted.} \end{cases}$$

The entries of matrices $[\mathbf{M}]$, $[\mathbf{K}]$ and $[\mathbf{C}]$ are calculated from

$$M_{ij}^\phi = \int_0^n m_t(z)\phi_i(z)\phi_j(z)\mathrm{d}z$$

$$K_{ij}^\phi = \int_0^h EI(z)\frac{\mathrm{d}^2\phi_i(z)}{\mathrm{d}z^2}\frac{\mathrm{d}^2\phi_j(z)}{\mathrm{d}z^2}\mathrm{d}z$$

$$M_{i\phi}^z = M_{\phi i}^z = \int_0^h z m_t(z)\phi_i(z)\mathrm{d}z$$

$$M_z^z = \int_0^h z^2 m_t(z)\mathrm{d}z$$

$$M_i^\phi = \int_0^h m_t(z)\phi_i(z)\mathrm{d}z$$

$$M_z = \int_0^h z m_t(z)\mathrm{d}z.$$

The coefficients of the upper diagonal of matrix $[\mathbf{C}]$ can be expressed in terms of damping ratios ξ_i as given in Refs 24 and 27.