# Dynamic analysis of ground-supported circular concrete tanks

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**ABSTRACT**

Conventionally reinforced and prestressed concrete circular tanks for liquid containing structures are extensively used in municipal and industrial facilities. The design of such structures requires that attention be given not only to strength requirements, but also to serviceability requirements under both hydrostatic and hydrodynamic loads. This study is focused on the nonlinear behavior of ground-supported open top circular concrete tanks under the effect of seismic loads. The finite element (FE) method is used to study the nonlinear response of the tank under time-history analysis. Furthermore, the response modification factors (R) included in current practice are evaluated based on the results of such studies. Several tank configurations with different aspect ratios, and base conditions including fixed, hinged and flexible are used in this study to attain reliable results and to validate the R-values used in current practice. A parametric study is carried out to determine the effect of tank relative dimensions and support conditions on response and specifically the response modification factor, R. It is found that the values of R in current practice are appropriate for fixed and hinged base tanks. However, the R-values for flexible base tanks (PC) are not appropriate and smaller values are suggested.

**KEYWORDS**: dynamic analysis, concrete tank, time- history analysis, finite element method, response modification factor
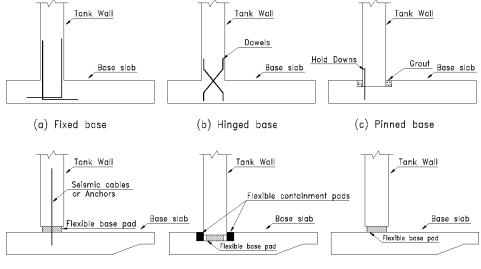
## INTRODUCTION

Liquid-containing structures (LCS) are important components in the commercial and industrial applications as they are used for storage water and other products such as oil and gas. Therefore, storage tanks can be considered as the lifeline of the industrial facilities. As numbers and sizes of the liquid-containing structures have increased over the years, so have their importance and the need for a better understanding of their behavior in order to formulate rational and efficient methods for their analysis and design. This need has been particularly pressing for developing systems that can withstand the applied loads including earthquakes and other dynamic excitations. Many research studies have been performed on tanks subjected to ground shaking. Extensive study on the dynamic behavior of LCS started in the late 1940's. Elkholy et al. [1] investigated on the optimal finite element modeling for modal analysis of liquid storage circular tanks. The FEM predictions compared well with available experimental and numerical results. A set of FEM options of parameters is recommended for elastic and inelastic analysis of such tanks. An analytical method was proposed to determine the dynamic response of 3-D flexible rectangular fluid containers by Hashemi et al. [2]. A simplified but sufficiently accurate design procedure was developed to improve code formulas for the seismic design of liquid storage tanks. They found that Rayleigh–Ritz method using the vibration modes of flexible plates, fluid–structure interaction effects on the dynamic responses of fluid containers, is the suitable method for dynamic analysis. Vathi et al. [3] investigated the seismic response of unanchored liquid storage tanks. In their research, base uplifting mechanics was examined numerically through a two-step methodology: (a) a detailed finite element shell model of the tank for incremental static analysis, capable of describing the state of stress and deformation at different levels of loading and (b) a simplified modeling of the tank as a spring-mass system for dynamic analysis, enhanced by a nonlinear spring at its base to account for the effects of uplifting. It was found that the results were aimed at possible revisions in the relevant seismic design provisions of design codes. Moslemi et al. [4] evaluated, using finite element method, the seismic response of concrete ground-supported cylindrical tanks. It was found that, the effects of the tank wall flexibility, vertical ground acceleration, base fixity, and earthquake frequency content have a significant effect on the dynamic behavior of LCS. It was also concluded that the current design procedure in estimating the hydrodynamic pressure is too conservative. Sadjadi [5] aimed to evaluate the leakage behavior of ground supported open top rectangular RC tanks under the effect of cyclic

loading. It was found that it may be appropriate to assume that leakage occurs soon after the yielding of the reinforcement. Ghaemmaghami et al. [6] investigated the seismic response of concrete rectangular and cylindrical liquid containing tanks in three-dimensional space. The study indicated that the effect of vertical excitation on the seismic response of the liquid tanks could be significant when considered separately; however, it was not as significant when the horizontal earthquake component was included, simultaneously, with the vertical component. A review of ten seismic codes on liquid- containing tanks, was performed by Jaiswal et al. [7] .It was concluded that there are significant differences among these codes on design seismic forces for various types of tanks. Reasons for these differences were critically examined and the need for a unified approach for seismic design of tanks was highlighted.

Even though, some studies were performed on the response of the tanks subjected to ground shaking, very little attention has been given to the nonlinear response and the level of available ductility for the ground-supported concrete circular tanks. As LCS are unique in their behavior under seismic loads, there are some debates about the basis on recommended values of the response modification factors (R) included in the current standards. The R-values have a significant effect on the seismic forces considered in the design and accordingly the required concrete dimensions and the amount of reinforcement. This study is focused on the nonlinear behavior of ground-supported circular reinforced concrete tanks under the effect of seismic loads. The nonlinear response of ground-supported circular tanks using time-history analysis is investigated. The values of response modification factors included in the current design codes and standards are evaluated.

## GROUND- SUPPORTED TANK BASE CONNECTION

Figures 1 shows the types of connection between tank wall and base slab. For fixed base support, no movement or rotation are allowed at the wall base. The bending moment at tank base is resisted by vertical reinforcement connecting tanks base to the tank wall where the vertical reinforcement extends across the joint. For hinged base support, no bending moment is transmitted between the tank wall and base in which rotation is allowed. In addition, for fixed and hinged base tanks, the earthquake base shear is transmitted partially by membrane (tangential shear) and the rest by radial shear that cause vertical bending. The flexible base supports are used for prestressed circular tanks (ACI 350.3-06 2006 [8]). For unanchored, flexible-base tanks, it is assumed that the base shear is transmitted by friction only. If friction between the wall base and footing, or between the wall base and bearing pads, is insufficient to resist earthquake shear, some form of mechanical restraint may be required. For anchored, flexible-base tanks, it is assumed that the entire base shear is transmitted by membrane (tangential) shear. The anchored, flexible-base support consists of seismic cables connecting the wall and the footing, as well as elastomeric bearing pads. The main mechanism for transferring the base shear from the wall to the foundation is the tangential resistance offered by a system of seismic cables connection the wall to the perimeter footing.



Figure 1. Ground-supported tank base connections

## TANK BEHAVIOR UNDER SEISMIC LOADS

Most of the design codes such as ACI 350.3 (2006) [8] assume an equivalent static model (Housner [9]) shown in figure 2 for calculating the resultant seismic forces acting on the ground-based fluid container with rigid walls. The equivalent mass of the impulsive component of the stored liquid $W_i$ represents the resultant effect of the impulsive seismic pressure

on the tank walls. In the model, it is assumed that $W_i$ acts rigidly with the tank walls at an equivalent height $h_i$ above the tank base that corresponds to the location of the resultant impulsive force $P_i$. The impulsive pressure is generated by seismic accelerations of the tank walls so that the force $P_i$ is evenly divided into a pressure on the wall accelerating into the fluid, and a suction on the wall accelerating away from the fluid. During an earthquake, the force $P_i$ changes direction several times, corresponding to the change in the direction of the base acceleration. $W_c$ is the equivalent mass of the oscillating fluid that produces the convective pressures on the tank walls with resultant force $P_c$, which acts at an equivalent height of $h_c$ above the tank base. In the model, it is assumed that $W_c$ is flexibly connected to the tank walls that produce a period of vibration corresponding to the period of fluid sloshing. The sloshing pressures on the tank walls result from the fluid motion associated with the wave oscillation. The period of oscillation of sloshing depends upon the ratio of fluid depth to tank diameter and is usually several seconds. The forces $P_i$ and $P_c$ exert overturning moments at the base of the tank wall.
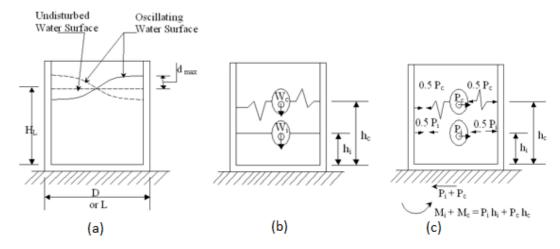


Figure 2. Liquid-containing tank rigidly supported on the ground; (a) Fluid motion in tank, (b) Dynamic model, (c) Dynamic equilibrium of horizontal forces

**TANK CONFIGURATION AND DESIGN PARAMETERS**

In this study, nonlinear FE time-history analysis is conducted on RC and PC circular tanks in order to investigate the nonlinear behavior of such structures under dynamic seismic loads. Circular RC and PC tanks with a wide range of $D/H_L$ ratios and different support conditions (flexible and nonflexible) are considered in order to verify the effect of these parameters on the tank response, and therefore verify $R_i$-values (implosive response modification factor) specified in the current practice [8].

Three models referred to as tanks 1, 2 and 3 with different base conditions are used in this study. These models are corresponding to $D/H_L$ ratios of 13.33, 6.67 and 4.44, respectively. The detail of tanks is shown in Table 1.

Nonflexible base is referred to as fixed and hinged base conditions. For flexible base tanks, only PC tank are investigated. The hinged and fixed bases are referred to as H, F respectively.

The flexible base tanks with horizontal prestressing and vertical conventional reinforcement are referred to as Flexible H-PC, and The flexible base tanks with horizontal and vertical prestressing are referred to as Flexible HV-PC.

The tank diameter (D) is equal to 40m. $H_L$, $H_W$ and $t_W$ are water depths, wall heights and wall thickness respectively.

Table 1. Tank details

| Tank type | $H_L$ (m) | $H_W$ (m) | $t_W$ mm | $D/H_L$ |
|---|---|---|---|---|
| 1H, 1F, 1H-PC, 1HV-PC | 3 | 3.25 | 250 | 13.33 |
| 2H, 2F, 2H-PC, 2HV-PC | 6 | 6.50 | 300 | 6.67 |
| 3H, 3F, 3H-PC, 3HV-PC | 9 | 9.60 | 400 | 4.44 |

The hydrodynamic forces are calculated based on ACI 350.3-06 [8], where tank walls are designed based on ACI 350-06 [10] for RC circular tanks, and for PC tanks, the design is according to ACI 373R [11], AWWA D110 [12] and Chapter 18 of ACI 350-06 [10]. The internal design forces and bending moments of tanks under consideration are calculated based on the results of linear static FE analysis. It is noted, the current practice [8] assigns the value of R for fixed and hinged base tanks ($R_i = 2$) and for flexible base tanks ($R_i = 3.25$).

The tanks are designed representing high seismic zone having $S_s$= 150% and $S_1$= 60%, corresponding to 1940 El-Centero earthquake record. $S_s$ is the mapped maximum considered earthquake 5% damped spectral response acceleration parameter at short periods, expressed as a fraction of acceleration due to gravity g. $S_1$ is the mapped maximum considered earthquake 5% damped spectral response acceleration; parameter at a period of 1 second, expressed as a fraction of acceleration due to gravity g. Furthermore, for time-history FE analysis, the El-Centro record, is scaled in such way that its peak ground acceleration (PGA) in the horizontal direction reaches 0.4g from its original value of 0.32g, where (g) is the acceleration due to gravity.

## CONSTITUENT MATERIALS

For linear elastic analysis, the material properties are specified as follows; the specified compression strength of concrete ($f'_c$) and yield strength of reinforcement ($f_y$) are 30 MPa and 400 MPa, respectively, where the modulus of elasticity of concrete ($E_c$) and reinforcement ($E_s$) are taken as 26000 MPa and 200000 MPa respectively. The concrete section is considered as uncracked section. For nonlinear analysis, concrete damaged plasticity (CDP) model is used since it is the most suitable concrete model for dynamic analysis. The stresses in concrete shell of the circular tanks are bending and bending plus axial stresses. Therefore, the Modified Hognestad method [13] is used as analytical approximation for the compressive stress-strain curve for concrete. The steel reinforcement is considered elastic perfectly plastic material.

The design requirements for prestressing tendons are specified in CSA Standards A23.3 [14] and A23.1 [15] including the minimum specified yield strength, and the minimum ultimate tensile strength. In this study, seven-wire strands grade CSA G279 are used for prestressing steel. It should be noted that, the steel grades for tendons depend on the minimum tensile strength ($f_{pu}$) which is 1860 MPa, and the yield strength ($f_y$) is 1581 MPa. The elastic modulus of non-prestresses tendons (Es) for prestressing steel is taken as 200,000 MPa in this study.

For both linear and nonlinear FE analysis, the following material proprieties are considered:

Thermal expansion coefficient of concrete ($\alpha_c$) and reinforcement ($\alpha_s$) = 0.0

Thermal expansion coefficient of prestressing tendons ($\alpha_{spt}$) = $1 \times 10^{-5}$

Poisson's ratio of concrete ($v_c$) = 0.18

Poisson's ratio of reinforcement ($v_s$) and prestressing tendons ($v_{pt}$) = 0.0

The most common type of elastomeric pads is 40H which is used for the flexible based tank bearing pads. The shear module of elastomeric bearing pad ($G_p$) is taken 0.345 MPa (50 psi) for type 40H.

## COMBINATION OF IMPLOSIVE AND CONVECTIVE COMPONENT

In order to verify, whether or not, the effect of convective component can be neglected for tanks under consideration, seismic forces for Tanks 1, 2 and 3 are calculated based on ACI 350.3-06 [8] including the effect of ($P_i$), ($P_c$), and ($P_w$). The base shear and bending moment are calculated by the square root of the sum of the squares (SRSS) method (procedure specified in AWWA [12] and ACI 350.3-06 [8] documents).

In addition, for the same tanks, the seismic forces are calculated based on ACI 350.3-06 [8] excluding the effect of ($P_c$), where only ($P_i$) and ($P_w$) are included. The results are referred to as ($P_i+P_w$) and ($M_i+M_w$), for base shear and bending moment, respectively. Accordingly, a comparison between the results of these two cases (including and excluding $P_c$) is carried out.

Where

V = Base shear; $P_c$ = Convective force; $P_i$ = Implosive force; $P_w$ = Lateral inertial forces of the accelerating wall; $M_b$ = Bending moment on the entire tank cross section just above the base of the tank wall; $M_c$ = Bending moment on the entire tank cross section just above the base of the tank wall due to the convective force $P_c$; $M_i$ = Bending moment on the entire tank cross section just above the base of the tank wall due to the impulsive force $P_i$; $M_w$ = Bending moment on the entire tank cross section just above the base of the tank wall due to the wall inertia force $P_w$.

The comparison between the results as shown in table 2 and table 3 show that, the ratios of ($P_i+P_w$) to total base shear V are more than 98% and 95% for nonflexible and flexible base conditions, respectively. In addition, the ratios of ($M_i+M_w$) to $M_b$ are around 97% and 94% for nonflexible and flexible base conditions, respectively. Therefore, the effects of ($P_i$) and ($P_w$) are much higher than the ($P_c$). Since the convective component has a negligible effect on the overall seismic response, it is ignored in this study.

The effects of impulsive and convective forces on the overall dynamic response are in agreement with those of previous research studies (Wood et al. [16] and Moslemi et al. [4, 5]).

Moreover, according to ACI350.3-06 [8], response modification factor for convective component of the accelerating liquid ($R_c$) is equal to one. Since $R_c$ is equal to 1.0, for all types of tanks, this parameter is not investigated in this study.

## COMPUTER MODEL AND FE ANALYSIS

The nonlinear time-history dynamic FE analysis is conducted using ABAQUS/CAE Version 6.8.3 (Dassault Systèmes Simulia Corp.) [17]. The entire tank is modeled using four-node quadrilateral shell elements. The wall thickness is significantly smaller

Table 2.  Earthquake hydrodynamic forces

| Tank No. | $P_i$ (kN) | $P_W$ (kN) | $P_c$ (kN) | V (kN) | $P_i+P_W$ (kN) | $(P_i+P_W)/V$ % |
|---|---|---|---|---|---|---|
| 1H, 1F | 1281 | 957 | 360 | 2276 | 2238 | 98.3 |
| 2H, 2F | 5125 | 974 | 1253 | 6227 | 6099 | 98 |
| 3H, 3F | 11521 | 2155 | 2295 | 13867 | 13676 | 98.6 |
| 1H-PC, 1HV-PC | 789 | 594 | 360 | 1429 | 1383 | 96.8 |
| 2H-PC, 2HV-PC | 3154 | 599.5 | 1253 | 3957 | 3753 | 94.9 |
| 3H-PC, 3HV-PC | 7090 | 1326 | 2295 | 8723 | 8416 | 96.4 |

Table 3.  Earthquake hydrodynamic bending moments on the entire tank

| Tank No. | $M_i$ (kN-m) | $M_W$ (kN-m) | $M_c$ (kN-m) | $M_b$ (kN-m) | $M_i+M_W$ (kN-m) | $(M_i+M_W)/M_b$ % |
|---|---|---|---|---|---|---|
| 1H, 1F | 1441 | 1569 | 544 | 3059 | 3010 | 98.4 |
| 2H, 2F | 11531 | 3166 | 3852 | 15194 | 14697 | 96.7 |
| 3H, 3F | 38884 | 10343 | 10880 | 50414 | 49227 | 97.6 |
| 1H-PC, 1HV-PC | 887 | 966 | 544 | 1931 | 1853 | 96 |
| 2H-PC, 2HV-PC | 7096 | 1948 | 3852 | 9831 | 9044 | 92 |
| 3H-PC, 3HV-PC | 23928 | 6365 | 10880 | 32188 | 30293 | 94.1 |

than the wall height and tank diameter; therefore, shell elements were considered appropriate to be used in modeling the tank walls. In this case, the thickness is defined through the section property definition. The conventional shell elements used in the analysis have three displacement and three rotational degrees of freedom at each joint. The numbers of elements along the wall height are considered four, seven, and ten elements for Tanks 1, 2 and 3 respectively. Along the water height (Hw), the tank wall is divided equally into one meter long elements, where the number of elements along the wall up to the water height is considered as three, six, and nine elements for Tanks 1, 2 and 3 respectively. The length of the very top element is considered to be equal to the tank freeboard which is equal to 0.25m, 0.5m and 0.6m for Tanks 1, 2 and 3, respectively. In order to maintain the aspect ratio to one, each tank circumference was divided into 128 equal elements where the element size along the circumference is approximately equal to 1 m.  Accordingly, the total number of elements for Tanks 1, 2 and 3 are 512, 896 and 1280 respectively.

In this study, five percent damping was considered in the time-history dynamic analysis. General damping was introduced in ABAQUS [17] in the form of "Rayleigh" damping.

For fixed and hinged base tanks, the tank supports were defined by means of applying boundary conditions that would restrain the movements or rotations in the desired direction. For anchored flexible base tanks, the seismic cables and bearing pads are modeled as spring elements.The stiffness of the anchored flexible support in the tangential direction ($K_t$) is the summation of K of Seismic cables and K of Bearing pads [8].In the radial direction, the stiffness of the bearing pads is also considered as the stiffness of the anchored flexible support ($K_r$) [8].As shown in Figure 3, the flexible base is modeled using two spring elements at each node of the tank base in the tangential and radial directions.  Each spring was defined by connecting two points, where one end of each spring was selected to be one of the tank joints at the base, where the other end was modeled as fixed support.

The prestressing force is applied in the form of thermal contraction that is applied only to the prestressing tendons assumed to be fully bonded to concrete. Therefore, the thermal expansion coefficient of concrete and reinforcement is assumed to be zero.

Reinforcement is modeled in concrete walls by means of rebars. Rebars are one-dimensional strain theory elements (rods); which are defined as embedded elements in oriented surfaces. Since the tendons are fully bonded for PC tanks, the prestressing tendons are modeled using the same technique as the reinforcement.

**MODELING OF MASSES**

In FE analysis, only the masses associated with impulsive component, which was modeled as the nonstructural mass and the
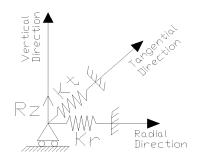
Figure 3. Flexible support model

tank wall are included since the effect of the convective component is negligible.

Virella et al. [18] researched on anchored ground supported circular tanks in order to investigate the validity of modeling the impulsive mass using added mass approach versus modeling with the acoustic elements. The response of the tanks that were examined using both approaches (added mass and acoustic elements) was very similar; in fact, the differences between the fundamental periods obtained using both approaches were smaller than 3 %. .

In this study the mass of the tank wall was defined as a "structural" mass which is the sum of all the mass contributions to the shell elements of the tank model. The structural mass was calculated from the material and section properties. Accordingly, the structural mass includes the mass due to any material definitions associated with the shell elements such as the mass from any rebars included in shell elements. In ABAQUS, the nonstructural mass contribution to an element is not allowed if that element has no structural mass. Since the impulsive mass of the contained liquid acts rigidly with the tank walls; therefore it was defined as added mass in the form of a "non-structural" mass which is the contribution to the model mass from features that are not part of the structural properties. Furthermore, in order to verify using the nonstructural mass approach in modeling the impulsive mass, the mass for Tank 1H is defined using added mass and nonstructural mass approaches. The material nonlinearity was not considered in this verification. The scaled El-Centro is used for time-history FE analysis. Results shows that the time-history responses of base shear for both methods are identical with only 1.5% difference for peak value.

## R-VALUES BASED ON THE RESULTS OF TIME-HISTORY FE ANALYSIS

The design of the tank walls is usually dictated by controlling the crack width. According to Sadjadi and Kianoush [5], in liquid containing structures (LCS), leakage occurs soon after the yielding of the reinforcement. Also, considering that the earthquake load is a transient load for a very short period of time, the stress in reinforcement can reach the yield stress at a certain location without compromising the structural integrity of the tank. Therefore, for transient loads, such as an earthquake, it is considered appropriate to estimate R-value at the first yield in the reinforcement.

For the nonflexible base tanks (H and F), to find R-value at which yield of wall barst is initiated, the following steps are used:

-The hydrostatic and equivalent static seismic loads for $R_i$ equal to 1 are calculated based on ACI 350.3 [8].
-The tank reinforcement is designed based for the combined effect of hydrostatic and seismic loads according to ACI 350 [10].
-Nonlinear dynamic time-history FE analysis is carried out, and the stress in the bars is obtained and compared with yield stress.
-If the reinforcement does not yield, the above steps are repeated for different Ri-values until reinforcement starts to yield.
-The R-value is considered based on the value at which yield of bars is initiated.
The summary of R-values at which first bar yield is initiated is presented in Table 4 for nonflexible and RC flexible base tanks.

Table 4. $R_i$-values based on results of FE analysis

| Tank No. | 1H | 2H | 3H | 1F | 2F | 3F |
|---|---|---|---|---|---|---|
| $R_i$ | 2.90 | 2.10 | 2.50 | 3.20 | 2.40 | 2.90 |

The results of this study show that, the tank reinforcement for fixed base tanks yields at higher assigned R-values than those for hinged base tanks. R-values based on FE analysis for fixed and hinged and base tanks are higher than the values specified in the current practice. For all cases, the results of FE analysis also indicate that, there is no single value for R, where R-values depend on tank relative dimensions and support conditions. Due to results of this study, the authors recommend the Ri-values for RC tanks with fixed and hinged base, 2.5 and 2.0 respectively.

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Figure 4 shows the percentage of $V_{Linear}/V_{Nonlinear}$ for the different supports and $D/H_L$ ratios. The maximum base shear based on linear and nonlinear FE analysis are referred to as $V_{Linear}$ and $V_{Nonlinear}$, respectively. As shown in this figure, for tanks with horizontal and vertical prestressing, the percentages of $V_{Linear}/V_{Nonlinear}$ are close to 100% for all $D/H_L$ ratios. In this case, the tank wall is initially under compression due to the prestressing force. Therefore, the effective (cracked) section properties are similar to the gross (uncracked) section properties since no cracks are developed under hydrostatic and seismic loads. According to Newmark and Hall [19], $V_{Linear}/V_{Nonlinear}$ ratios define the system ductility factor as per FEMA 450 [20]. Thus, tanks with horizontal and vertical prestressing may not sustain the required level of ductility, and therefore, may not dissipate energy.
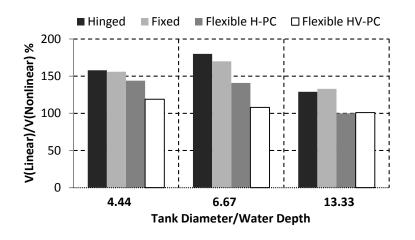


*Figure 4. Effect of tank dimensions and support conditions on the ratio between  nonlinear and linear dynamic base shear*

For PC flexible base tanks, the increase in stresses in prestressing tendons under the effect of hydrostatic and dynamic earthquake loads is relatively small for all tanks. Therefore, to determine the R-values at which yield of prestressing tendons is initiated may not be practical for PC tanks. In this case, the ratio between linear and nonlinear base shear based on time-history FE analysis is used to determine the value of ductility reduction factor  according to (ATC-19 1995a [21]). For this case, the overstrength factor is considered to be equal to 1.4 as per FEMA 450 [20]. The R-values for PC flexible base tank are calculated as the product of the ductility factor and the overstrength factor (Newmark and Hall [19]). Accordingly, the R-values are calculated based on the results of the time-history FE analysis that are performed on tanks with different support conditions and various $D/H_L$ ratios as presented in table 5. As shown in this table, the $R_i$-values for PC anchored flexible base tanks that are considered in this study are less than the $R_i$-value specified in current practice. Therefore, the flexible base tanks with seismic cables may not dissipate the seismic forces as expected. The main reason is due to the linear behavior of the seismic cables as these cables are made of high yield strength material. Accordingly, it can be concluded that the anchored flexible supports should be designed with smaller $R_i$-values or other mechanism that can dissipate the seismic energy should be used. Also, $R_i$-values for PC flexible base tanks are less than those for fixed and hinged base tanks.

The results of the nonlinear time-history FE analysis show that cracks develop in concrete near tank supports for fixed and hinged tanks. Vertical reinforcement may yield at the base for fixed base tanks. Therefore, the nonflexible tank supports may have more ductility and dissipate more energy than the flexible tank supports. While using prestressing tendons improves the tank serviceability by controlling crack width and reducing tensile stress in concrete, yet, this method may reduce the level of ductility and reduces R-values assigned to PC tanks. Thus, the recommended Ri-value by the authors for PC tanks with anchored flexible support is 1.5.

Table 5.    $R_i$-values for Flexible base PC tanks based on results of FE analysis

| Tank No. | 1H-PC | 2H-PC | 3H-PC | 1HV-PC | 2HV-PC | 3HV-PC |
|---|---|---|---|---|---|---|
| $R_i$ | 1.40 | 1.97 | 2.02 | 1.41 | 1.51 | 1.66 |

## CONCLUSIONS

Based on the results of the nonlinear time-history FE analysis, it is found that the current practice provides reasonably accurate results in terms of Ri-values compared to extensively detailed and timely consumed nonlinear time-history FE analysis for fixed and hinged base tanks. However, Ri-values specified in current practice for anchored flexible base tanks are more than

those based on the results of FE analysis. The results of this study can be summarized as follows:

1- The impulsive force and the lateral inertial force of the accelerating wall have a much bigger contribution to the total response than the convective term for the tanks considered in this study.

2- Modeling the impulsive mass using the nonstructural mass or added mass is a reliable approach since the results based on this approach are very similar to those of models with the fluid inside the tank using acoustic elements.

3- The use of prestressing affects the tanks ductility. The case of combined horizontal and vertical prestressing result in linear response of the tank wall as cracks may not develop in concrete.

4- The flexible base tanks with seismic cables do not dissipate the seismic energy as expected due to the linear behavior of the seismic cables.

5- The nonflexible base tanks show more ductility and dissipate more energy than the flexible base tanks. This is due to development of cracks in concrete walls in fixed and hinged base tanks.

6- The recommended Ri-values by authors for RC tanks with fixed and hinged base and PC tanks with anchored flexible support are 2.5, 2.0, and 1.5, respectively.

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