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Seismic Design Provisions for Concrete Water Storage Tanks in Iranian Publication 123: Changes and Updates from Edition 1993 to 2015

#	Name	Email Address	Degree	Position	Country	Affiliation
1	Razavi, Mojtaba	omran.saze@gmail.com	Ph.D.	Other	Iran	Department of Civil Engineering, University of Tabriz, Tabriz, Iran
2	Ashrafzadeh, Fedra	f.ashrafzadeh@iau.ac.ir	Ph.D.	Assistant Professor	Iran	Department of Civil Engineering, Ur.C., Islamic Azad University, Urmia, Iran

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ABSTRACT

Water storage tanks must remain operational post-earthquake to sustain emergency response and public health. Seismic-induced liquid–structure interaction generates hydrodynamic pressures that produce critical stresses on structural components, often leading to failures such as cracking or leakage in concrete tanks. Consequently, their functional importance and damage susceptibility demand rigorous design standards to ensure seismic resilience. Iran's principal design standard for concrete water storage tanks, Publication 123, was last substantially revised in 1993. However, post-earthquake observations and advances in global seismic design practices later revealed significant shortcomings in its provisions. To address these gaps, the standard underwent a comprehensive revision, integrating modern hydrodynamic models, load combinations, and performance-based criteria from ACI 350.3, thus markedly enhancing the seismic performance and reliability of such infrastructure. This paper presents a comprehensive comparative analysis of the 1993 and 2015 editions, examining the evolution of seismic design provisions for buried, non-buried, and semi-buried concrete tanks. The analysis reveals significant improvements in seismic zoning, importance factor differentiation, response modification factors, pressure distribution modeling, and load combinations. The practical implications of these changes on reinforcement detailing, wall thickness, and overall structural performance are discussed, and recommendations for future code development are provided.

KEYWORDS: Seismic Design, Concrete Water Storage Tanks, Publication 123, Rectangular Tanks, Cylindrical Tanks

1. Introduction

The seismic response of liquid storage tanks remains a persistent structural engineering challenge, arising from the complex coupling of hydrodynamic forces, wall flexibility, and soil-structure interaction. As critical infrastructure components, these tanks must preserve post-earthquake functionality to ensure uninterrupted water supply and firefighting capacity. However, historical events, including the 1960 Chile and 1964 Alaska earthquakes, have consistently revealed deficiencies in conventional design approaches. This concern is further underscored by a legacy of tank failures, such as sloshing damage in the 1933 Long Beach earthquake, roof collapses and upper-wall distress during the 1952 Kern County earthquake in California, and similar oil-sloshing failures in the 1960 Chile event (Rawat, Mittal, Chakraborty, & Matsagar, 2019), collectively emphasizing the urgent need for ongoing methodological refinement. Central to these vulnerabilities is fluid-structure interaction, which induces dynamic pressure distributions markedly different from those in conventional structures.

Recent research has underscored the critical role of seismic excitation characteristics and the dynamic properties of liquid storage tanks in governing their seismic response. Ranjbar et al. (Bozorgmehrnia, Ranjbar, & Madandoust, 2013) demonstrated through time-history analyses of an elevated reinforced-concrete water tank that key structural responses, including base shear, overturning moment, and displacement, are strongly influenced by earthquake characteristics, particularly frequency content. Similarly, Yazdanian and Ghasemi (Yazdanian & Ghasemi, 2017) highlighted the importance of convective and impulsive frequencies as fundamental dynamic characteristics of liquid storage tanks and demonstrated that these frequencies can be determined through finite-element modal analysis and evaluated against analytical formulations specified by seismic design codes.

The foundation of seismic analysis for liquid storage tanks was established by Housner (Housner, 1957), who proposed a mechanical analog model with a two-mass representation of impulsive and convective fluid responses. Recognized as a pioneer in this domain, Housner formulated a two-degree-of-freedom model that separates the liquid response into an impulsive component (moving with the tank wall) and a convective component (sloshing), a formulation subsequently adopted by numerous design codes and standards (Housner, 1954). Though this early model assumed rigid walls, later studies indicated that this simplification can produce unconservative results. A major advance showed that wall flexibility significantly amplifies hydrodynamic pressures through interaction between the deforming structure and the contained fluid, a finding that fundamentally shifted the understanding of the problem and underscored the need for more sophisticated analytical approaches (Veletsos, 1984).

Building upon the seminal contributions of Housner and Veletsos, subsequent research has expanded in several key directions. Studies on rectangular tanks (Medhat Ahmed Haroun, 1980; Kianoush & Chen, 2006; Kianoush & Ghaemmaghami, 2011; Kim, Koh, & Kwahk, 1996) have uncovered added complexities related to three-dimensional response and aspect ratio variations. Their findings consistently demonstrate that hydrodynamic pressures in flexible-walled tanks deviate substantially from rigid-wall predictions, carrying significant implications for design load estimation. Modern numerical techniques, including the coupled FDM-FEM approach have advanced this research, confirming that neglecting two-way FSI produces unconservative underestimations of wall pressures, displacements, and bending moments in rectangular RC tanks (Fedra Ashrafzadeh, Razavi, & Tariverdilo, 2025). The growing emphasis on wall flexibility and bidirectional FSI (Medhat A

Haroun, 1983; Kianoush & Ghaemmaghami, 2011; Yang, 1976) has motivated a range of experimental, numerical, and analytical studies (Mandal & Maity, 2015; Matsui, 2009; Yan, Guo, Wu, & Zhang, 2024) aimed at better characterizing these complex phenomena.

Numerous subsequent studies have further illuminated these effects using diverse methodologies. Using a coupled FVM-FEM formulation, Bui (Bui, 2024) found marked differences in hydrodynamic pressure between flexible and rigid tanks, with bidirectional FSI increasing responses by 12.2% and 16.8% over ACI and EC8 predictions, respectively. Rawat et al. (Rawat et al., 2019) validated coupled acoustic-structural and Eulerian-Lagrangian FEM approaches against experiments, confirming their accuracy in reproducing seismic responses. Rassoulpour and Safi (Rassoulpour & Safi, 2020) similarly used SPH/FEM coupling to emphasize that wall flexibility critically affects sloshing and overall tank response, reinforcing the need to account for this effect in seismic design.

Despite the increasing prevalence of numerical methods, analytical approaches remain relatively limited in this field (Wu et al., 2021; Yu & Whittaker, 2020a, 2020b). Kim et al. (Kim et al., 1996) addressed this gap by applying analytical techniques to partially filled 3D rectangular tanks with flexible walls, establishing that hydrodynamic pressures on flexible walls exceed those on rigid counterparts, and that 2D models approach 3D behavior as the width-to-height ratio increases. Hashemi et al. (Hashemi, Saadatpour, & Kianoush, 2013) developed a simplified design procedure to refine code formulas. Rezaiee-Pajand et al. (Rezaiee-Pajand, Aftabi S, & Kazemiyan, 2016) derived closed-form expressions for fundamental frequencies of 2D deformable tanks and proposed approximate pressure distribution formulas. Moradi et al. (Moradi, Behnamfar, & Hashemi, 2018) proposed a Rayleigh-Ritz-based mechanical model for flexible cylindrical tanks, showing that ACI 350.3 (Committee, 2006) either overestimates or underestimates seismic responses relative to their predictions. A common simplification assumes that for tanks with large length-to-width ratios and negligible wall thickness differences, the flexibility of the two longer side walls governs dynamic behavior, allowing 3D configurations to be reduced to 2D problems for FSI analysis (Kim et al., 1996).

Parallel to analytical progress, a range of innovative retrofit and control strategies has emerged. Passive non-structural interventions, such as air cushions against cylindrical walls (F Ashrafzadeh, Tariverdilo, & Sheidaii, 2016), modify boundary conditions to mitigate sloshing forces, base shear, and overturning moments. Active and adaptive internal devices, including submerged pendulum wall systems (Razavi, Alipour, Ashrafzadeh, & Tariverdilo, 2024) and flexible partitions functioning as tuned mass absorbers (Mousavi & Tariverdilo, 2014), offer adjustable mechanisms for suppressing fluid motion and dissipating seismic energy. For elevated tanks, performance-based frameworks, notably a modified Direct Displacement Based Design (DDBD) approach, have been adapted to better represent the dynamic response of RC frame staging, thereby addressing limitations inherent in conventional force-based procedures (Kumbhar, Kumar, P L, & Noroozinejad Farsangi, 2019).

Recognizing that foundation flexibility adds further complexity, researchers have increasingly turned their attention to soil–structure–fluid interaction (SSFI). Studies have shown that soil compliance can significantly alter the fundamental period and damping of the tank-fluid system (Livaoğlu & Doğangün, 2006; Livaoglu, 2008). This is supported by experimental evidence (Hussein, Al Neami, & Rahil, 2021), indicating that earthquake-induced hydrodynamic pressures worsen settlement, lateral drift, and excess pore pressure in saturated sandy soils compared to static conditions, a critical SSFI effect often neglected in practice. Reinforcing this, Livaoğlu et al. (Livaoglu & Dogangun, 2007) used FEM to demonstrate that SSFI substantially affects lateral wall displacements, with wall flexibility and FSI significantly amplifying these deformations.

The multi-directional nature of seismic excitation introduces another critical consideration. While early studies primarily addressed horizontal ground motion, research by Kianoush and Chen (Kianoush & Chen, 2006) has shown that vertical acceleration components can significantly affect wall stresses, particularly when axial and bending responses interact, an effect especially relevant for rectangular tanks. Moreover, the frequency content of ground motions may resonate with the tank-fluid system's dynamic characteristics in ways conventional response spectrum methods fail to capture, highlighting the need for more refined analytical frameworks. Fluid-structure interaction fundamentally alters the dynamic properties of both domains, shaping the overall container response (C.-B. Lee & Lee, 2021; J. H. Lee & Cho, 2024; Shekari, 2018). Yet most current codes require revision, especially regarding two-way FSI, to better reflect actual behavior; neglecting this interaction can lead to improper designs or even collapse (Kalogerakou, Maniatakis, Spyrakos, & Psarropoulos, 2017). Priority improvements include incorporating free-surface sloshing effects and adjacent wall flexibility.

In response, recent studies have introduced various coupled numerical approaches, including SPH/FEM (Constantin, De Courcy, Titurus, Rendall, & Cooper, 2021; Jamalabadi, 2020; Ng, Alexiadis, Chen, & Sheu, 2020), MPS/FEM (Mitsume, Yoshimura, Murotani, & Yamada, 2014; Zhang & Wan, 2018), and BEM/FEM (Dogangun & Livaoglu, 2004; Saghi, Hirdaris, & Saghi, 2021) methods that become particularly complex when two-way FSI is considered (Ghaemmaghami & Kianoush, 2010; Saghi et al., 2021). Coupled techniques generally offer superior accuracy in replicating tank behavior, explaining the growing focus on bidirectional FSI. Bui (Bui, 2024) proposed an FVM/FEM hybrid for flexible tanks under seismic loading, while another study developed FDM/FEM coupling for free-surface flow with elastic plates (Liao & Hu, 2013). Paik and Carrica (Paik & Carrica, 2014) advanced the field with a coupled nonlinear FEM and URANS/DES solver, using a gluing method for force-displacement transfer between fluid and structural domains, along with a linear FEM solver for boundary layer grid deformation.

Substantial progress has been made across multiple fronts of tank seismicity, including refined numerical modeling of flexibility, innovative seismic demand reduction strategies, performance-based design for specific configurations, and improved soil interaction characterization. Yet integrating these disparate effects into a unified framework remains a considerable challenge. The combination of 3D fluid-structure-soil interaction and multi-directional excitation creates considerable analytical complexity. Many current design codes still rely on simplified mechanical models that, though computationally efficient, may not fully capture the coupled system response. This limitation is especially critical for tanks with unconventional geometries, those on soft soils, or those in regions with complex seismic hazards. Consequently, enhanced numerical methods are increasingly employed to more accurately predict hydrodynamic pressures on flexible walls while considering two-way FSI, which constitutes a primary objective of the present investigation.

This study aims to address these challenges through a comprehensive comparative evaluation of the seismic design provisions in the 1993 and 2015 editions of Iranian Publication 123 (Planning & Budget, 1993). By systematically tracing the evolution of design methodologies, this investigation pursues four main objectives: (1) identifying key revisions in the 2015 edition and their technical justifications; (2) quantifying their practical impact on tank design outcomes; (3) assessing alignment with current international standards and research; and (4) pinpointing remaining gaps and proposing recommendations for future code development. The findings are expected to inform ongoing refinement of design methodologies, particularly by facilitating practical implementation of current provisions and identifying research priorities. Ultimately, this integrated approach seeks to bridge the gap between advanced analytical capabilities and practical design needs, contributing to enhanced seismic resilience of these critical infrastructure assets.

2. Seismic Design Provisions for Concrete Water Storage Tanks

2.1. General Design Philosophy

The seismic design of concrete water storage tanks in Iran is primarily governed by Publication 123, issued by the Planning and Budget Organization, which draws substantially from the provisions of the ACI code. Water storage tanks, which are essential for potable water supply and network pressure management, are typically constructed in either cylindrical or rectangular configurations, with cylindrical shapes being generally preferred for their superior structural efficiency. Their geometric characteristics are presented in Figure 1. The construction type of the tank—whether above-ground, buried, or semi-buried—is determined by technical and passive defense requirements and directly influences its detailing.

The roof-to-wall connection represents a critical design consideration. In semi-buried tanks, this connection is typically designed as free to accommodate thermal expansion stresses, whereas in buried tanks, it is generally monolithic to ensure a watertight barrier against groundwater ingress, supplemented by the installation of waterstops. Additionally, the wall system may be configured either as a continuous monolithic structure or as segmented panels with vertical expansion joints to control cracking, ultimately defining the overall structural behavior of the tank.

For dynamic analysis under earthquake loading, the Publication adopts Housner's well-established theory. This method simplifies the complex hydrodynamic fluid-structure interaction by representing the liquid mass as two concentrated components: an impulsive mass that moves rigidly with the tank wall, and a convective mass that captures the sloshing motion. This two-degree-of-freedom model forms the basis for an equivalent static analysis, which is a practical and widely employed approach in seismic design.

The stability, strength, and serviceability of structures and their members must be verified under at least the loadings specified in this section. However, depending on the specific characteristics of each tank, additional special loadings beyond conventional ones should be considered where necessary. In general, loads applied to tanks manifest either as forces or as self-straining effects. For instance, dead, live, and earthquake loads are applied as forces, whereas temperature changes and concrete volume changes produce strains within the structural members. These loads are classified into three distinct types: static loads, dynamic loads, and special loads.

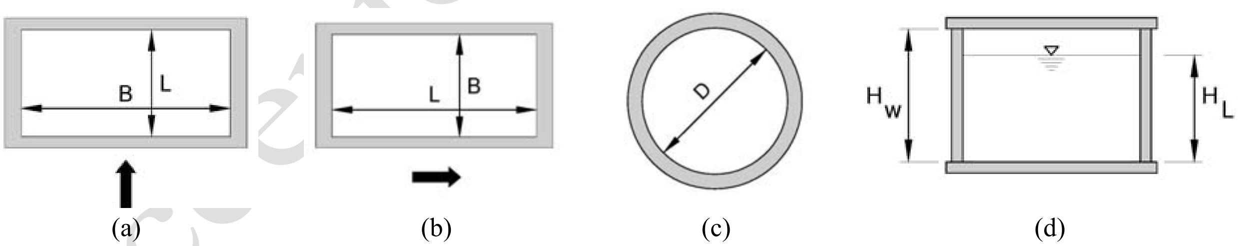


Fig. 1. Definition of the geometric parameters for the tank models: (a) Plan view of a rectangular tank under transverse seismic loading; (b) Plan view of a rectangular tank under longitudinal seismic loading; (c) Plan view of a cylindrical tank; (d) Elevation view of both rectangular and cylindrical tanks.

- **Static Loads:** Static loads include dead loads, live loads, static fluid and soil pressures, and the effects of temperature changes—although the latter is reclassified as a special load in the 2015 Edition.
  - *Dead Loads:* The specific weights of materials for dead load calculations shall be in accordance with Iranian National Publication No. 519 or the Sixth Topic of the National Building Regulations. The weight of permanent equipment and pipes must be included based on their significance and magnitude, as determined by the design engineer.

- *Live Loads:* The live load on the tank roof slab must account for all equipment and vehicles that may be present during construction, operation, and maintenance.
- *Fluid Pressure:* The lateral and vertical static pressure of the stored fluid must be calculated using fluid mechanics principles, based on the fluid's specific weight and the depth from the free surface.
- *Soil Pressure:* The lateral static soil pressure on peripheral walls must be determined by considering the soil height and the tank's geometry. Due to the high stiffness of tank walls and soil creep, the lateral soil pressure at rest (K_0) should be used. A lower pressure coefficient may be applied only if a detailed analysis of wall stiffness and deformation, accounting for soil type, reliably justifies a reduction.
- *Temperature Effects:* Tank structures must be designed to withstand forces from temperature-induced self-straining. For the scope of this Publication, the thermal gradient caused by the temperature difference between the stored fluid and the environment can generally be neglected. If consideration is necessary, this gradient may be assumed linear. Based on engineering judgment, buried and non-buried tanks shall be designed to withstand uniform temperature changes of at least 20°C and 40°C, respectively.

- **Dynamic Loads:** This category primarily comprises inertia forces generated by the vibration of the structure, the stored fluid, and the surrounding soil, where present. The structural analysis for determining internal forces arising from seismic actions must be conducted in accordance with the methods prescribed by Iranian Standard 2800. Dynamic analysis techniques—including those that account for fluid-structure-soil interaction—are permitted, provided their correct implementation can be assured. When such methods are employed, the resulting structural response values (e.g., member forces and displacements) must be scaled by the ratio of the base shear obtained from the equivalent static analysis to that from the dynamic analysis.
- **Special Loads:** This loading category, introduced in the 2015 revision of Publication 123, encompasses temperature variations and passive defense considerations.

2.2. Fundamental Assumptions and Limitations

Before examining the specific provisions of any code edition, it is essential to first establish the fundamental assumptions that underpin the analytical framework. Both the 1993 and 2015 editions of Publication 123 are based on the following key assumptions:

- *Rigid Foundation:* The foundation is assumed to be completely rigid, with no consideration of soil-structure interaction effects.
- *Horizontal Excitation Only:* The analysis considers only horizontal ground motion, neglecting the vertical component of seismic acceleration.
- *Linear Response:* The structural response is assumed to be linear elastic, with no explicit consideration of nonlinear behavior or inelastic deformations.
- *Two-Degree-of-Freedom Model:* The fluid response is modeled using a two-degree-of-freedom representation, with the impulsive and convective components treated independently.
- *Equivalent Static Analysis:* The dynamic response is represented through an equivalent static analysis procedure, with seismic forces calculated based on response spectrum methods.

These assumptions introduce limitations that should be recognized when applying the code provisions. The neglect of soil-structure interaction and vertical excitation, in particular, may lead to unconservative designs for certain site conditions and tank configurations. The following sections provide a detailed examination of the specific provisions in each code edition.

3. Seismic Regulations of Publication 123 (1993 Edition)

3.1. Hydrostatic, Soil, Thermal, and Seismic Hydrodynamic Loads

Hydrostatic pressure exerted by the stored fluid is computed as $P = \gamma h$, with γ denoting the fluid's specific weight and h the depth below the surface. Lateral soil pressures are determined using Rankine's theory, with a design requirement that they shall not be less than the equivalent pressure of a fluid with a unit weight of 6 kN/m³. Where high groundwater levels are present, the associated hydrostatic pressure is added to the soil pressure.

The tank structure must be designed to resist forces arising from temperature fluctuations. For buried installations, a uniform temperature change of $\Delta T = 20°\mathrm{C}$ is assumed for this analysis. Seismic-induced hydrodynamic forces are evaluated using three primary approaches: equivalent static analysis, pseudo-dynamic analysis, and full dynamic analysis. The latter two, more advanced methods rely on finite-element modeling of the coupled tank–fluid system. The subsequent discussion focuses solely on the equivalent static approach.

3.2. Equivalent Static Analysis

Per Housner's theory, the seismic response of water-like fluids in rigid-walled tanks is represented by a two-degree-of-freedom system, as shown in Figure 2. In this model, the fluid mass is split into two components: an impulsive mass (W_i) that moves rigidly with the tank wall, and a convective mass (W_c) that accounts for sloshing action and oscillates with a significantly longer period.

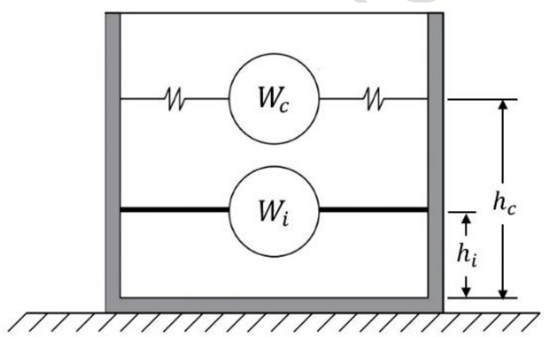


Fig. 2. Dynamic model of liquid-containing tank rigidly supported on the ground (adapted from (Housner, 1957)).

3.3. Seismic Loading on Tank Walls

Seismic excitation induces a range of dynamic pressure components on tank walls. These include pressures generated directly by ground motion, as well as those resulting from structural vibrations. The resulting lateral force system comprises components due to wall motion (P_W), roof motion (P_r), and the predominant impulsive (P_i) and convective (P_c) hydrodynamic forces, as shown schematically in Figure 3.

The impulsive resultant, P_i, acts at a height h_i above the base, while the convective (sloshing) resultant, P_c, acts at a height h_c. Sloshing also produces a maximum wave amplitude, $d_{\max}$ (detailed in Section 3.4), and the associated pressure distributions contribute significantly to the base shear and overturning effects. The dynamic base pressure, P_b, together with the overturning moments generated by both the impulsive and convective masses, is essential for stability evaluation.

The total seismic base moment, M_b, serves as the primary design parameter and is computed as the sum of its impulsive (M_{bi}) and convective (M_{bc}) components.

In the equivalent static analysis method, the dynamic forces arising from horizontal tank vibration are determined using Eqs. (1) – (4). These equations define several distinct forces. The forces P_w and

P_r result from the inertia of the tank's wall and roof masses, which are considered rigid and vibrate with a very low period. The force P_i is due to the vibration of the impulsive mass of the stored fluid, which oscillates in synchrony with the tank structure. In contrast, the hydrodynamic force P_c results from the convective fluid mass and is the only component that vibrates with a different, and typically much larger, period.

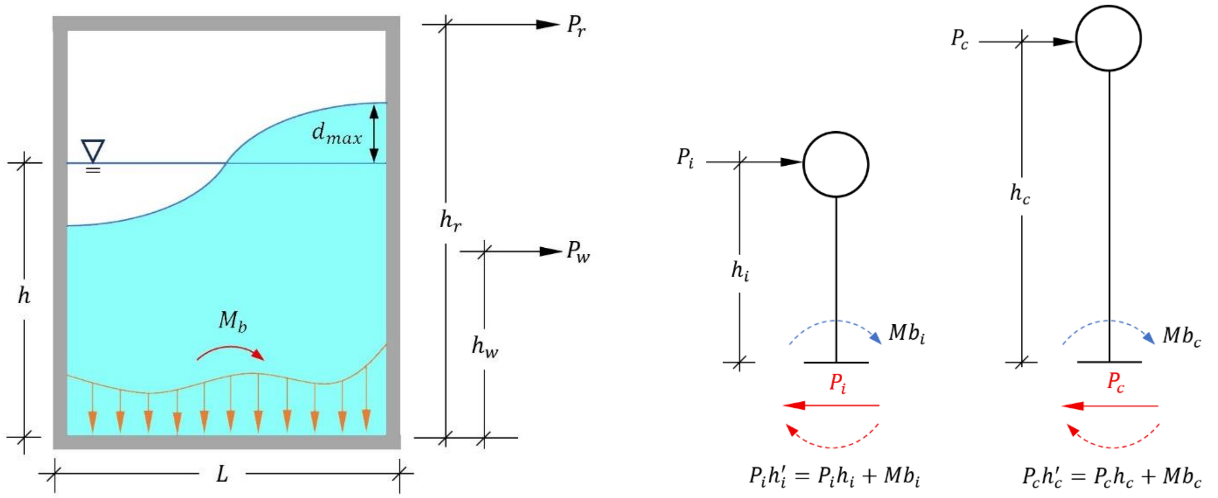


Fig. 3. Lateral and vertical forces applied to the tank.

$$P_w = C.W_w \tag{1}$$

$$P_r = C.W_r \tag{2}$$

$$P_i = C.W_i \tag{3}$$

$$P_c = C.W_c \tag{4}$$

In these equations, C is the seismic coefficient, as defined in Standard 2800 as follows:

$$C = \frac{ABI}{R} \tag{5}$$

In the equation above, the design basis acceleration ratio, A, is determined according to regional seismicity, as stipulated by Iranian Standard 2800. The parameter B is the structural response factor, which depends on the tank's natural period and site soil conditions. The Standard provides separate B factors for the impulsive and convective modes. The response factors for impulsive and convective modes are given by Eqs. (6) and (7), respectively. The Standard specifies a response factor of $B = B_{\mathrm{i}} = 2.0$ for computing lateral forces arising from wall motion (P_W) and roof motion (P_r).

$$B_{\mathrm{i}} = 2.0 \tag{6}$$

$$B_{\mathrm{c}} = 2\left(\frac{T_0}{T}\right)^{2/3} < 2 \tag{7}$$

where T is the fundamental natural period of the tank, defined as:

$$T = 1.81 K_t \sqrt{h} \tag{8}$$

In these equations, T_0 is the characteristic period of the site (as per the Standard), and h is the design liquid height.

The importance factor, I, is taken as a constant value of 1.2 for tank structures under all design conditions. Similarly, the response modification factor, R, is assigned a constant value of 3, applicable to both buried and non-buried tanks.

Once the lateral forces P_i and P_c and their respective points of application are determined, their combined resultant is distributed along the wall height as a trapezoidal force pattern. This distribution captures the variation of hydrodynamic pressure, as illustrated in Figure 4. The base shear and base overturning moment are computed using Eqs. (9) and (10). From these base values, the force intensities at the top (q_1) and bottom (q_2) of the wall are derived to define the seismic load profile. This trapezoidal load distribution is subsequently used for the seismic design of the tank.

$$V = \frac{q_1 + q_2}{2} hL = P_i + P_c \tag{9}$$

$$M_b = \left[\frac{q_1 h^2}{2} + \frac{(q_2 - q_1)}{6} h^2\right] L = P_i h_i + P_c h_c \tag{10}$$

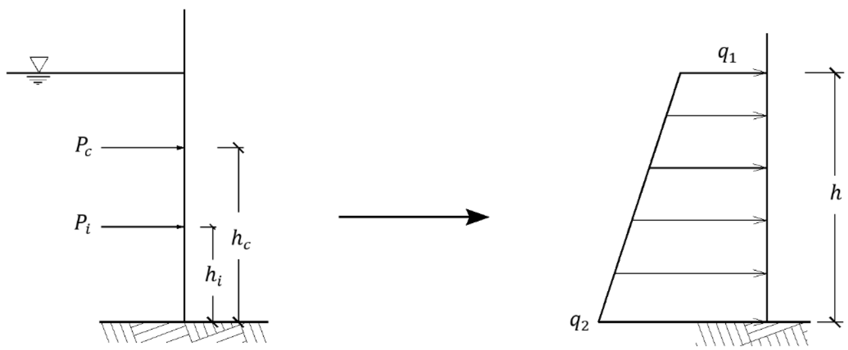


Fig. 4. Trapezoidal distribution of lateral hydrodynamic force on the wall.

The impulsive and convective masses, as well as the levels at which their corresponding forces are applied, are determined based on the height-to-length (or height-to-diameter) ratio of the tank—for rectangular and cylindrical tanks, respectively. These parameters are obtained from Figure 5 (for mass determination) and Figure 6 (for the effective height of force application), as shown below.

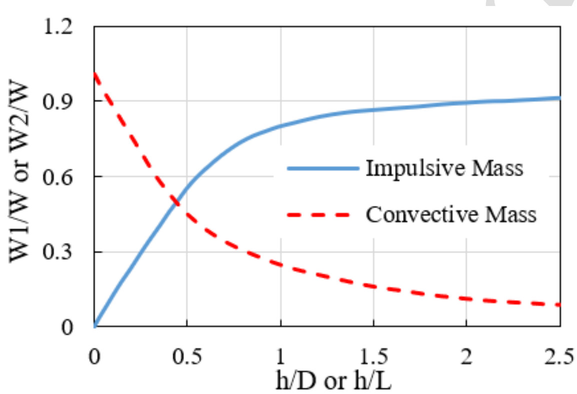


Fig. 5. Determination of impulsive and convective masses for rectangular and cylindrical tanks.

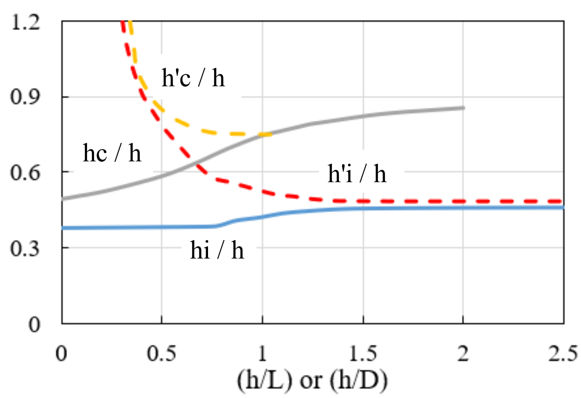


Fig. 6. Effective height of impulsive and convective masses for rectangular and cylindrical tanks.

3.4. Additional Wave Height

A freeboard—defined as the vertical distance from the still-water surface to the underside of the roof—must be provided to prevent the formation of a negative pressure zone due to wave impact against the roof structure. The freeboard shall exceed the maximum sloshing wave height, $d_{\max}$, which is computed using Eq. (11):

$$d_{max} = \left[\frac{0.417\left(\frac{ABI}{R}\right)}{1 - K_d\left(\frac{ABI}{R}\right)} \right] \tag{11}$$

The distribution factor, K_d, is determined from Publication 123 as a function of the tank's aspect ratio, h/L (or h/D for cylindrical tanks).

3.5. Design Load Combinations

Water storage tanks are designed using the ultimate strength method in accordance with the 1993 Edition. The load combinations applied are those listed in the following tables. A complete description of the notation used is provided in the Nomenclature section at the end of the paper.

Table 1. Load combinations without factors

Combination Name	Load Combination
Combo1	$D + L + F_S$
Combo2	$D + L + E_S$
Combo3	$Combo1 \pm T$
Combo4	$Combo2 \pm T$
Combo5	$Combo1 + EQ + FD$
Combo6	$Combo2 + EQ + ED$

Table 2. Factored load combinations

Combination Name	Load Combination
Combo1	$1.4D + 1.7L + 1.7F_S$
Combo2	$1.4D + 1.7L + 1.7E_S$
Combo3	$0.75(combo1 \pm 1.4T)$
Combo4	$0.75(combo2 \pm 1.4T)$
Combo5	$0.75(combo1 + 1.87F_d + 1.87EQ)$
Combo6	$0.75(combo1 + 1.87E_d + 1.87EQ)$

Table 3. Factored load combinations with reliability factor

Combination Name	Load Combination
Combo1	$1.82D + 2.21L + 2.21F_S$
Combo2	$1.82D + 2.21L + 2.21E_S$
Combo3	$0.75(Combo1) \pm 1.365T$
Combo4	$0.75(Combo2) \pm 1.365T$
Combo5	$0.75(Combo1) + 1.4025(FD + EQ)$
Combo6	$0.75(Combo2) + 1.4025(ED + EQ)$
Combo7	$0.75(Combo1) - 1.365T$
Combo8	$0.75(Combo2) - 1.365T$

3.6. Wall Reinforcements and Crack Control

The reinforcement design is driven by a combined assessment of two interacting load effects: the case of peak axial tension, T, along with its corresponding bending moment, and the case of peak bending moment, M, along with its corresponding tensile force. The governing design is selected as the more critical outcome derived from these interaction scenarios. The tensile steel area required under pure axial load is obtained using Eq. (12), whereas the flexural reinforcement is determined from Eq. (13). This procedure is consistently applied at key locations, such as the wall-to-base and wall-to-roof junctions, the wall mid-height, wall intersections, and the primary flexural steel in both the base slab and the roof.

$$A_s = \frac{T}{\phi f_y} \quad (12)$$

$$A_s = \frac{M}{\phi f_y d} \quad (13)$$

In the absence of a specific analytical procedure for determining crack widths under combined loading conditions, an interaction formula is adopted. This expression integrates the separate allowable steel stress limits for pure axial tension and pure flexure, thereby accounting for their combined influence:

$$\frac{f_{st}}{F_{st}} + \frac{f_{sb}}{F_{sb}} \leq 1 \quad (14)$$

3.7. Development Length and Splice Length

Accurate computation of development and splice lengths is fundamental to the structural integrity of the reinforcement system. The development length, L_d, denotes the minimum anchorage embedment necessary for a reinforcing bar to attain its full yield strength via bond with the adjacent concrete, thus avoiding pull-out failure. At locations where continuity requires bar overlapping, splice lengths are furnished to enable effective stress transfer from one bar to the next. These lengths are established following the provisions of the governing concrete design standard, with due consideration of bar diameter, concrete tensile strength, steel yield stress, coating type, and casting position (top or bottom bars). The development length is calculated using Eq. (15), as presented below. Precise determination of these parameters is crucial for preserving structural continuity and dependable load path transmission, particularly in zones that are vulnerable to the combined effects of tension and flexure.

$$L_d = 0.0594 \frac{A_b f_y}{\sqrt{f_c'}} \quad (15)$$

3.8. Limitations of the 1993 Edition

Although the 1993 edition of Publication 123 offered a practical framework for tank design, it exhibits several significant limitations that motivated the comprehensive revision in 2015, as outlined below:

- *Simplified Seismic Zoning:* The three-zone seismic classification does not adequately capture regional variations in seismic hazard, potentially leading to unconservative designs in some regions and overly conservative designs in others.
- *Constant Importance Factor:* The uniform importance factor of 1.2 for all tanks does not reflect the varying consequences of failure for different tank types and functions.
- *Uniform Response Modification Factor:* The constant R-value of 3 for all burial conditions fails to account for the beneficial effects of soil confinement on buried tanks.
- *Simplified Wall Mass Modeling:* The assumption of $\varepsilon = 1$ for wall mass contribution neglects the dynamic interaction between the wall and the contained fluid.
- *Linear Pressure Distribution:* The linear (trapezoidal) pressure distribution does not accurately represent the actual nonlinear pressure distribution in flexible tanks.
- *Geometry-Independent Approach:* The use of simplified, geometry- independent charts does not adequately capture the distinct dynamic characteristics of rectangular versus cylindrical tanks.

4. Seismic Regulations of Publication 123 (2015 Edition)

4.1. General Framework and Key Improvements

The 2015 edition represents a comprehensive revision of the seismic design provisions, incorporating lessons learned from recent earthquakes and advancements in international codes. The key improvements are organized around several major themes: refined seismic hazard characterization, improved modeling of fluid-structure interaction, recognition of burial effects, and adoption of performance-based design principles.

4.2. Equivalent Static Analysis

The equivalent static analysis method, based on Housner's theory and employing a two-degree-of-freedom model that captures the first vibration mode of the stored fluid, remains unchanged between the 1993 and 2015 editions of Publication 123.

4.3. Seismic Loading on Tank Walls

The seismic loading formulation on tank walls, as defined by Eqs. (16) – (19), was originally established in the 1993 edition of Publication 123 and has been largely carried forward into the 2015 revision, with some differences, particularly in the definition of the coefficient ϵ and the values of the response modification factor R and the structural response factor B, which are no longer treated as constants.

$$P_w = \frac{AB_iI}{R_i}[\varepsilon W_w] \tag{16}$$

$$P_r = \frac{AB_iI}{R_i}[W_r] \tag{17}$$

$$P_i = \frac{AB_iI}{R_i}[W_i] \tag{18}$$

$$P_c = \frac{AB_iI}{R_i}[W_c] \tag{19}$$

The design basis acceleration ratio, A, is established based on regional seismicity in accordance with Iranian Standard 2800. For large or critical tanks, a site-specific response spectrum is recommended where site-specific seismic hazard data are available; however, A shall not be taken below the minimum values specified in Standard 2800.

Based on previous studies, the vibration period of the rigid components of tanks, T_i, is typically less than 0.3 seconds. Consequently, as a simplifying assumption, the response factor for these rigid parts, B_i, may be taken as the maximum value stipulated in Standard 2800, corresponding to the local soil type and seismic hazard level. For the convective mass, the response factor B_c shall be taken as no less than 1.5 times the 5%-damped design spectral acceleration specified in Standard 2800. This conservative approach should be retained unless a refined calculation of T_i can reliably justify the use of a lower value. The horizontal vibration period of the convective mass, T_c, as well as the coefficient λ for both rectangular and cylindrical tanks, are determined from Eqs. (20) – (22). It is noted that the convective mode period is consistently longer than that of the impulsive mode. It is important to note that while the 2015 updates introduce several refinements relative to the previous edition, neither code edition includes provisions or discussion addressing soil-structure interaction (SSI).

$$T_c = \left(\frac{2\pi}{\lambda}\right)\sqrt{L\ or\ D} \tag{20}$$

$$\lambda = \sqrt{3.16g \times \tanh\left[3.16\left(\frac{H_L}{L}\right)\right]} \tag{21}$$

$$\lambda = \sqrt{3.68g \times \tanh\left[3.68\left(\frac{H_L}{D}\right)\right]} \tag{22}$$

Alternatively, Figures 7(a) and 7(b) provide the same information for rectangular and cylindrical tanks, respectively.

The importance factor (I) for tanks must be established based on the stored fluid, the tank's operational use, and its location, as prescribed by Iranian Standard 2800. For instance, under the 2015 provisions, both a municipal water storage tank serving post-earthquake firefighting and drinking water supply (classified as an essential facility) and an industrial tank storing a toxic or flammable critical fluid (classified as a hazardous facility) are assigned the same maximum importance factor of I = 1.4, illustrating that the highest seismic priority applies to tanks whose failure would pose either significant public safety risks or environmental hazards.

The effective mass coefficient of the walls, ε, which accounts for the inertial contribution of the tank wall itself, is determined from Eq. (23) for rectangular tanks and Eq. (24) for cylindrical tanks. Similarly, the ratios of the equivalent weights of the impulsive part (W_i) and the convective part (W_c) to the total fluid weight (W_L) for rectangular and cylindrical tanks are determined from Eqs. (24)-(25) and Eqs. (27)-(28), respectively. As a practical alternative, Figures 8(a) and 8(b) can be used to approximate these ratios.

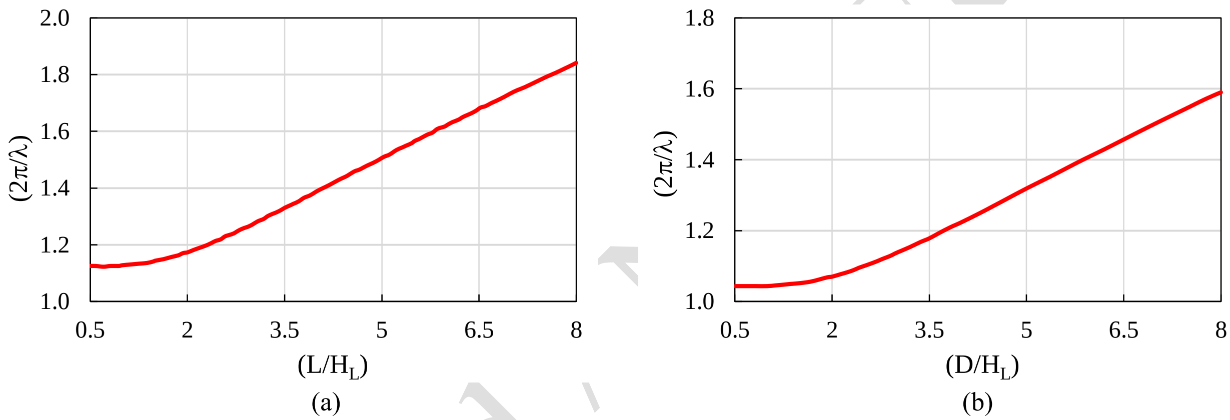


Fig. 7. Coefficient $\frac{2\pi}{\lambda}$ versus the $\frac{L}{H_L}$ and $\frac{D}{H_L}$ ratios: (a) rectangular tanks; (b) cylindrical tanks.

For rectangular tank model:

$$\varepsilon = \left[0.0151\left(\frac{L}{H_L}\right)^2 - 0.1908\left(\frac{L}{H_L}\right) + 1.021\right] \leq 1.0 \quad (23)$$

$$\frac{W_i}{W_L} = \frac{\tanh\left[0.866\left(\frac{L}{H_L}\right)\right]}{0.866\left(\frac{L}{H_L}\right)} \quad (24)$$

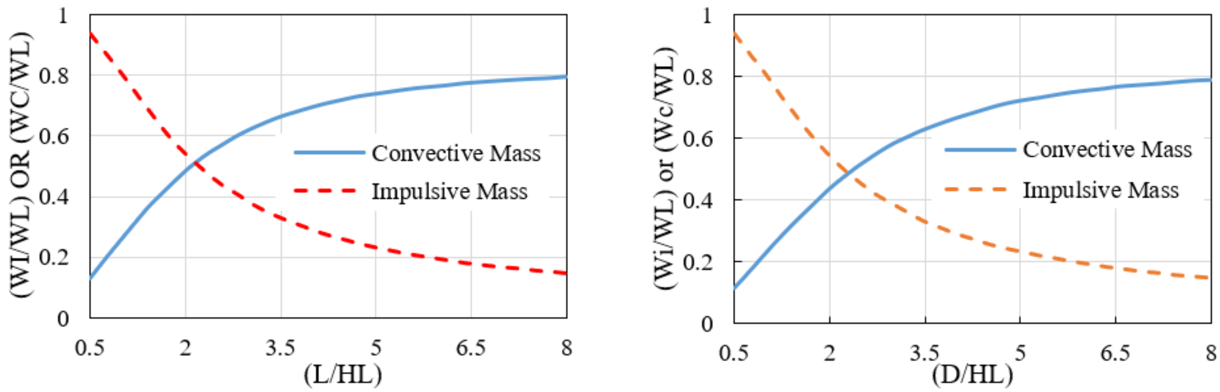
$$\frac{W_c}{W_L} = 0.264\left(\frac{L}{H_L}\right)\tanh\left[3.16\left(\frac{H_L}{L}\right)\right] \quad (25)$$

For cylindrical tank model:

$$\varepsilon = \left[0.0151\left(\frac{D}{H_L}\right)^2 - 0.1908\left(\frac{D}{H_L}\right) + 1.021\right] \leq 1.0 \quad (26)$$

$$\frac{W_i}{W_L} = \frac{\tanh\left[0.866\left(\frac{D}{H_L}\right)\right]}{0.866\left(\frac{D}{H_L}\right)} \quad (27)$$

$$\frac{W_c}{W_L} = 0.230\left(\frac{D}{H_L}\right)\tanh\left[3.68\left(\frac{H_L}{D}\right)\right] \quad (28)$$



(a) (b)

Fig. 8. Coefficients $\frac{W_i}{W_L}$ and $\frac{W_c}{W_L}$ versus the $\frac{L}{H_L}$ and $\frac{D}{H_L}$ ratios: (a) rectangular tanks; (b) cylindrical tanks.

The response modification factor, R, reflects the ductility, energy dissipation capacity, and inherent uncertainty of the structural system. For liquid-containing tanks, the R values for both the impulsive and convective components are determined from Table 4, based on the tank's support and burial conditions. A buried tank is defined as one in which the fluid level remains below the surrounding natural ground elevation. For semi-buried tanks, the R value is obtained through linear interpolation between the limits prescribed for buried and non-buried configurations. Under the new code edition, R is assigned as 4 for buried tanks, 3 for non-buried (above-ground) tanks, and an intermediate value for partially buried tanks, as presented in Table 4. This gradation reflects the beneficial effect of soil confinement in enhancing ductility and reducing seismic demand. In contrast, the earlier code edition employed a single, lower constant value of R for all burial conditions, which imposed higher seismic loads on tanks and led to a more conservative and less flexible design approach.

Table 4. Response modification factor (R) for tanks at the serviceability limit state

Tank model	R_i
Non-buried Tanks	3
Buried Tanks	4

The forces arising from horizontal tank vibration, as determined from Eqs. (16) – (19), are subsequently used to compute the corresponding bending moments at the wall base for structural design purposes, in accordance with Eqs. (29) – (32).

$$M_w = P_w h_w \quad (29)$$

$$M_r = P_r h_r \quad (30)$$

$$M_i = P_i h_i \quad (31)$$

$$M_c = P_c h_c \quad (32)$$

In these equations, h_r and h_w denote the heights from the wall base to the center of gravity of the roof slab and the tank walls, respectively. The parameters h_i and h_c represent the corresponding heights to the points of application of the lateral forces arising from the impulsive and convective fluid masses. These heights are determined without accounting for the effect of unbalanced pressure acting on the tank base.

The parameters h_i and h_c are determined using Eqs. (33)-(38), in conjunction with Figure 9(a) for rectangular tanks and Figure 9(b) for cylindrical tanks.

For rectangular tank model:

$$if\ \frac{L}{H_L} < 1.3333\ \rightarrow\ \frac{h_i}{H_L} = 0.5 - 0.09375\left(\frac{L}{H_L}\right) \tag{33}$$

$$if\ \frac{L}{H_L} \geq 1.3333\ \rightarrow\ \frac{h_i}{H_L} = 0.375 \tag{34}$$

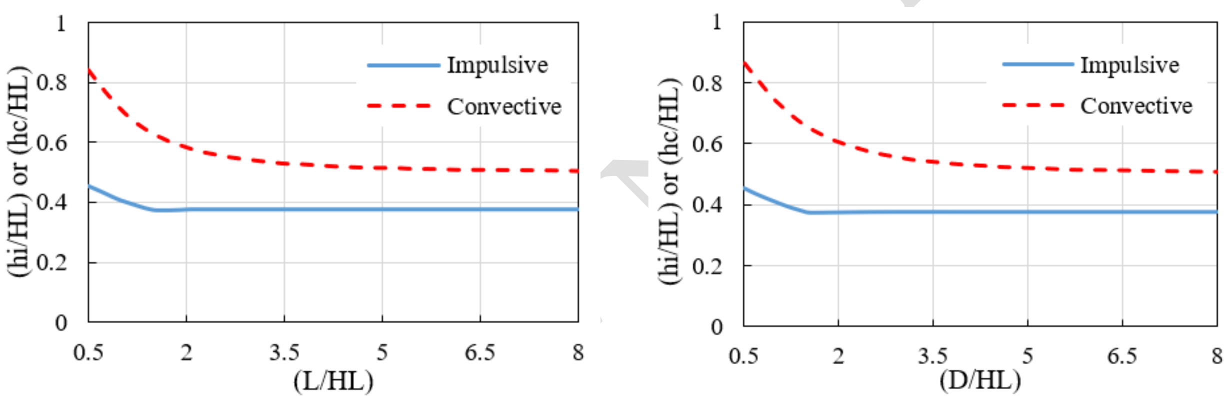
$$\frac{h_c}{H_L} = 1 - \frac{\cosh\left[3.16\left(\frac{H_L}{L}\right)\right] - 1}{3.16\left(\frac{H_L}{L}\right) \times \sinh\left[3.16\left(\frac{H_L}{L}\right)\right]} \tag{35}$$

For cylindrical tank model:

$$if\ \frac{D}{H_L} < 1.3333\ \rightarrow\ \frac{h_i}{H_L} = 0.5 - 0.09375\left(\frac{D}{H_L}\right) \tag{36}$$

$$if\ \frac{D}{H_L} \geq 1.3333\ \rightarrow\ \frac{h_i}{H_L} = 0.375 \tag{37}$$

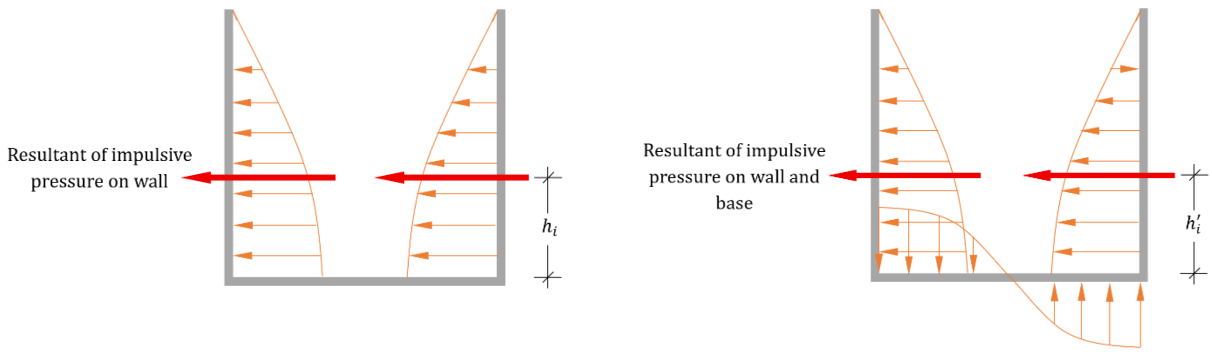
$$\frac{h_c}{H_L} = 1 - \frac{\cosh\left[3.68\left(\frac{H_L}{D}\right)\right] - 1}{3.68\left(\frac{H_L}{D}\right) \times \sinh\left[3.68\left(\frac{H_L}{D}\right)\right]} \tag{38}$$



(a) (b)

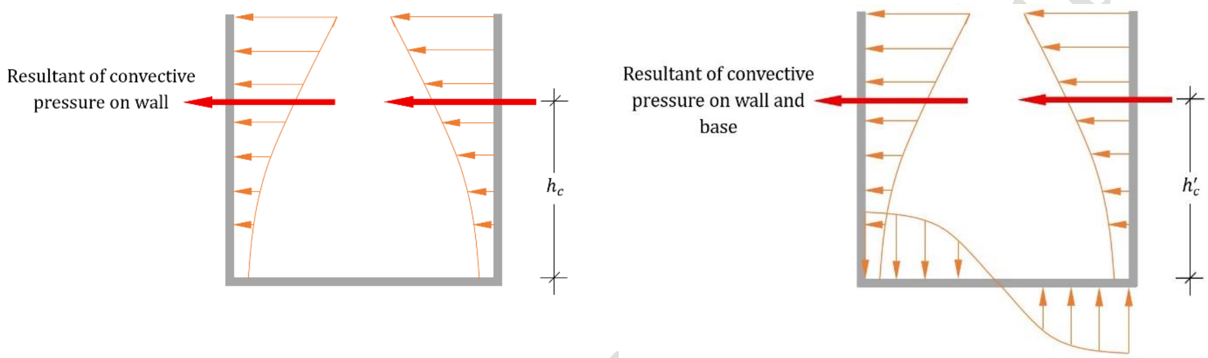
Fig. 9. Values of the coefficients $\frac{h_i}{H_i}$ and $\frac{h_c}{H_L}$, excluding unbalanced base pressure: (a) rectangular; (b) cylindrical tanks.

Once the resultant lateral forces arising from structural and fluid vibration, as well as their respective points of application, have been established, these forces must be suitably distributed along the height of the wall. Theoretical studies indicate that the hydrodynamic lateral pressure distribution follows a hyperbolic pattern, as illustrated in Figures 10 and 11. Nevertheless, for practical design purposes, a trapezoidal pressure distribution may be adopted with sufficient accuracy, provided that the location of the resultant force is preserved. Eqs. (39) and (40) are provided to facilitate this simplification. It should be noted that both the previous and the 2015 code editions assume a rigid foundation and neglect the vertical component of seismic acceleration, such that the analysis of wall stresses and foundation pressures remains limited to horizontal hydrodynamic effects.



(a) (b)

Fig. 10. Hydrodynamic pressure distribution due to the impulsive component in a liquid storage tank: (a) without and (b) with unbalanced floor pressure.



(a) (b)

Fig. 11. Hydrodynamic pressure distribution due to the convective component in a liquid storage tank: (a) without and (b) with unbalanced floor pressure.

$$P_{iy} = \frac{\frac{p_i}{2}\left[4H_L - 6h_i - (6H_L - 12h_i)\left(\frac{y}{H_L}\right)\right]}{H_L^2} \quad (39)$$

$$P_{cy} = \frac{\frac{p_c}{2}\left[4H_L - 6h_c - (6H_L - 12h_c)\left(\frac{y}{H_L}\right)\right]}{H_L^2} \quad (40)$$

4.4. Additional Wave Height

To prevent the development of negative pressure beneath the tank roof due to fluid sloshing during seismic events, or to avoid recurrent operational overflow, the available freeboard must be no less than 300 mm and must also exceed the maximum wave height, $d_{\max}$. For rectangular and cylindrical tanks, $d_{\max}$ is calculated using Eqs. (41) and (42), respectively. Where provision of the full calculated freeboard is not practicable, the roof slab shall be designed to withstand the residual negative sloshing pressure, taking into account the minimum operational freeboard.

$$d_{max} = 0.5\ LAB_cI \quad (41)$$

$$d_{max} = 0.5\ DAB_cI \quad (42)$$

4.5. Design Load Combinations

All structural elements of the tank must be designed to reliably resist every load combination with adequate reliability throughout their service life. Table 5 presents the design load combinations prescribed for the limit state method, along with the corresponding load factors as specified by the

Iranian Concrete Code. To facilitate direct comparison, Table 6 summarizes the factored load combinations for equivalent loading scenarios as prescribed in the 1993 and 2015 editions.

Table 5. Factored load combinations.

Combination Name	Load Combination
Combo1	$1.25D + 1.5L + 1.5F$
Combo2	$1.25D + 1.5L + 1.5H$
Combo3	$0.85D + 1.5H + 1.5F$
Combo4	$D + 1.2L + 1.2F + 1.2Es$
Combo5	$D + 1.2L + 1.2H + 1.2Es$
Combo6	$0.85D + F + 1.2Es$
Combo7	$0.85D + 1.2H + 1.2Es$
Combo8	$D + 1.2L + 1.2F \pm T$
Combo9	$D + 1.2L + 1.2H \pm T$
Combo10	$1.25D \pm 1.25T$

Table 6. Comparative summary of factored load combinations for equivalent loading scenarios.

Loading Scenario	1993 Edition (ACI 318-based)	2015 Edition (ACI 350-based)
Static loads only	$1.4D + 1.7L + 1.7F_S$	$1.25D + 1.5L + 1.5F$
Static + soil pressure	$1.4D + 1.7L + 1.7E_S$	$1.25D + 1.5L + 1.5H$
Soil + fluid pressure	Not explicitly provided	$0.85D + 1.5H + 1.5F$
Seismic + fluid pressure	$0.75(combo1 + 1.87F_d + 1.87EQ)$	$D + 1.2L + 1.2F + 1.2E_s$
Seismic + soil pressure	$0.75(combo2 + 1.87E_d + 1.87EQ)$	$D + 1.2L + 1.2H + 1.2E_s$
Seismic + fluid (alternative)	Not separately defined	$0.85D + F + 1.2E_s$
Seismic + soil (alternative)	Not separately defined	$0.85D + 1.2H + 1.2E_s$
Temperature + fluid	$0.75(combo1 \pm 1.4T)$	$D + 1.2L + 1.2F \pm T$
Temperature + soil	Not separately defined	$D + 1.2L + 1.2H \pm T$
Temperature only	Not separately defined	1.25D ± 1.25T

4.6. Wall Reinforcements and Crack Control

The minimum tensile reinforcement at potential plastic hinge locations near the wall-to-base slab connection shall be determined from the following Equation:

$$\rho_{min} = \frac{0.25\sqrt{f_c'}}{f_y} \quad (43)$$

Furthermore, the minimum reinforcement for hoop tension in the walls of cylindrical tanks must be calculated using:

$$\rho_{min} = \frac{0.05\, f_c'}{f_y} \quad (44)$$

The design of rectangular tanks requires particular attention to stress concentrations at wall corners and intersections. Supplementary horizontal reinforcement is essential in these regions, with its quantity and development length determined through a detailed three-dimensional structural analysis. It is important to note that pure bending is not usually the decisive design criterion for these walls. A more severe scenario arises when high tensile forces act in combination with bending. Therefore, to guarantee proper crack control and load transfer, the full calculated horizontal reinforcement must be placed continuously across the entire wall span.

Temperature and shrinkage reinforcement, consisting of two perpendicular grids, is placed in the planar tank elements to control thermal cracking (as per Edition 1993, Eq. (14)) and to ensure structural continuity where calculated steel requirements are negligible. Because serviceability governs tank design, this temperature steel is substantially more critical than it would be in typical structures.

4.7. Development Length and Splice Length

Development and lap splice lengths are typically established per the Iranian Concrete Code (ABA), with values dependent on the bar type and concrete strength grade. Notably, the code prescribes more conservative minimum lap splice requirements for certain reinforcement categories: no less than 55 bar diameters for temperature and shrinkage steel, 40 bar diameters for primary flexural reinforcement, and 60 bar diameters for hoop tension reinforcement in cylindrical tank walls.

4.8. Lessons Learned from Recent Earthquakes

Although no Iran-specific tank failures have been formally documented as direct drivers of the 2015 code revision, fundamental lessons learned from major international earthquakes, particularly the widespread tank damage during the 1960 Chile and 1964 Alaska earthquakes, catalyzed research into fluid sloshing, tank wall flexibility, and the low ductility of tank structures. These findings subsequently informed international standards such as ACI 350.3, from which Iranian Code No. 123 is derived, and thus indirectly shaped the provisions of the 2015 edition.

5. Comparative Analysis of Publication 123: 1993 Edition versus 2015 Edition

According to the items mentioned in the previous sections, fundamental revisions have been made in the new edition of Publication 123. This section provides a comprehensive comparison of the seismic regulations of the two editions, with particular attention to the practical implications of the changes.

5.1. Seismic Coefficient Components

The 2015 edition introduces substantial revisions to the seismic coefficient components, as outlined in the following discussion.

5.1.1. Design Basis Acceleration (A)

- *1993:* Three seismic hazard zones (high, medium, low) with coefficients of 0.35, 0.25, and 0.20.
- *2015:* Adopts Iranian Standard 2800 zoning—four hazard levels (very high, high, medium, low) with coefficients of 0.35, 0.30, 0.25, and 0.20.
- *Comparison:* The new edition provides a more refined hazard assessment by increasing the number of seismic zones from three to four. This refinement allows for more accurate characterization of regional seismic hazards, potentially reducing conservatism in moderate-hazard regions while maintaining appropriate safety in high-hazard regions.

5.1.2. Response Factor (B)

- *1993:* This edition assigns a constant response factor of $B_i = 2.0$ to tanks with periods under 0.3 seconds, while the coefficient B_c is calculated using the expression given in Eq. (7).
- *2015:* This edition distinguishes between two response factors: for the impulsive mass, B_i is taken as the maximum response factor from Standard 2800; for the convective mass, B_c is set to no less than 1.5 times the values of the 5%-damped design spectrum from the same standard.
- *Comparison:* In contrast to the earlier simplified approach, the 2015 edition computes distinct response factors for the convective and impulsive masses, with each value derived from the respective dynamic characteristics—specifically, soil type and tank period. This refinement reflects the recognition that impulsive and convective components exhibit fundamentally different responses to seismic excitation, with the convective mass being particularly susceptible to long-period ground motions.

5.1.3. Importance Factor (I)

- *1993:* Constant $I = 1.2$ for all ground-supported tanks.
- *2015:* I per Standard 2800, based on fluid type, tank usage, and site location.
- *Comparison:* The 2015 edition introduces a variable importance factor that reflects the tank's specific function and associated risk, thereby replacing the previous uniform approach. For instance, municipal water storage tanks designated for post-earthquake firefighting receive a higher importance factor than industrial tanks storing non-critical fluids, recognizing their essential role in emergency response.

5.1.4. Response Modification Factor (R)

- *1993:* Constant $R = 3$ for all ground-supported tanks.
- *2015:* R based on support conditions – Buried: 4, Non-buried: 3, Semi-buried: interpolated.
- *Comparison:* The new edition acknowledges the improved performance of buried tanks by assigning them a higher response modification factor, whereas the previous edition applied a uniform value regardless of support conditions. The higher R-value for buried tanks reflects their improved ductility and energy dissipation capacity due to soil confinement, resulting in reduced seismic demands and potentially more economical designs.

5.2. Effective Mass Coefficient (ε)

- *1993:* Constant $\varepsilon = 1.0$.
- *2015:* ε per Eqs. (23) and (26), accounting for wall inertia.
- *Comparison:* The 2015 edition provides a rational method to determine the wall's effective mass, marking a significant improvement over the previous simplistic assumption, which did not accurately model the wall's dynamic behavior. The formulation for the effective mass coefficient in the 2015 edition is consistent with the approach in ACI 350.3, particularly in its treatment of the impulsive mass contribution from the wall. The coefficient accounts for the dynamic interaction between the wall and the fluid, recognizing that the effective wall mass participating in seismic response depends on the tank geometry and aspect ratio. The parabolic expressions in Eq. (23) and (26) account for the reduction in effective mass as the tank becomes shallower in proportion to its length or diameter.

5.3. Design Load Combinations

- *1993:* Load combinations per ACI 318 (general building code).
- *2015:* Load combinations per ACI 350.3 (environmental engineering structures), aligned with national concrete code.
- *Comparison:* The comparative Table in section 4.5 (Table 6) reveals several fundamental shifts in the design philosophy between editions. First, the load factors in the 2015 edition have been reduced from the more conservative 1.4-1.7 range to 1.25-1.5, reflecting the adoption of ACI 350.3-specific load factors. Second, the 2015 edition introduces a distinct load combination for combined soil and fluid pressures (0.85D+1.5H+1.5F, Combo 3) that was absent in the 1993 edition. Third, the treatment of seismic loading has been substantially refined: the 1993 edition used a 0.75 factor with 1.87 multipliers, whereas the 2015 edition employs a more rational format with 1.2 factors and direct inclusion of dead and live loads. The 2015 edition also introduces alternative seismic combinations (Combos 6 and 7) that account for cases where reduced dead load may be critical for stability. Fourth, temperature effects are now treated with greater clarity and comprehensiveness through three distinct combinations (Combos 8, 9, and 10), compared to the 1993 edition's single combination. The introduction of Combo 10 (1.25D±1.25T) is

particularly significant, as it addresses temperature effects acting alone with dead load—a scenario relevant for empty or partially filled tanks during extreme temperature variations. Fifth, the expanded set of 10 combinations in the 2015 edition, compared to the 6 combinations in the 1993 edition (or 8 including reliability factor variations), reflects a more mature understanding of the complex loading conditions that concrete water storage tanks experience throughout their service life.

5.4. Additional Wave Height (Freeboard)

- *1993:* Empirical Eq. (11).
- *2015:* Dynamic sloshing theory per Eqs. (41) and (42).
- *Comparison:* The 2015 edition fundamentally revises the freeboard calculation by directly incorporating seismic parameters and tank geometry, yielding a rational, site-specific determination—a clear improvement over the previous empirical rule, which was unrelated to seismic demand.

5.5. Lateral Force Distribution

- *1993:* The impulsive (P_i) and convective (P_c) hydrodynamic forces were distributed as an equivalent linear trapezoidal load along the wall height using simplified, geometry-independent charts (Figs. (2) – (4)).
- *2015:* These forces are applied as distributed loads with hyperbolic profiles (Eqs. (39) and (40)), with separate provisions for rectangular and cylindrical tanks.
- *Comparison:* The hyperbolic distribution provides a more realistic representation of hydrodynamic pressure: impulsive pressure peaks near the base, while convective pressure is more evenly spread along the height—a distinction absent in the earlier linear approximation.

6. Discussion

6.1. Evolution of Seismic Design Philosophy

The evolution from the 1993 to the 2015 edition of Publication 123 reflects a broader paradigm shift in seismic design philosophy. The 1993 edition, consistent with the design practices of its time, employed a relatively simplified approach that prioritized computational simplicity and conservatism through the use of uniform factors and simplified models. The 2015 edition, in contrast, reflects the modern performance-based design philosophy, which aims to achieve more predictable and reliable performance through more sophisticated modeling and refined characterization of seismic demands. This conceptual evolution is evident in several key aspects of the code revisions:

- *From Uniform to Differentiated Factors*: The movement from uniform importance and response modification factors to differentiated values reflects an understanding that different tank types and configurations have distinct risk profiles and performance characteristics.
- *From Simplified to Refined Models:* The adoption of hyperbolic pressure distributions and geometry-specific formulations reflects the integration of research findings into design practice, enabling more accurate prediction of structural response.
- *From General to Specialized Load Combinations:* The shift from ACI 318 to ACI 350.3 load combinations acknowledges the unique serviceability requirements of liquid-retaining structures.

6.2. Assessment of Code Adequacy

While the 2015 edition represents a significant advancement, several limitations remain that should be addressed in future code development:

6.2.1. Soil-Structure Interaction

As noted in Section 4.3, neither edition of Publication 123 includes provisions for soil-structure interaction. This represents a significant gap, particularly for tanks founded on soft soils where SSI effects can substantially modify the dynamic response characteristics. Future code revisions should consider incorporating simplified SSI provisions, similar to those found in other international standards.

6.2.2. Vertical Excitation

Both editions neglect the vertical component of seismic acceleration. Research has demonstrated that vertical excitation can significantly influence wall stresses and foundation pressures, particularly for tanks with large aspect ratios. Future revisions should consider incorporating provisions for vertical excitation, especially for regions with significant vertical seismic hazards.

6.2.3. Two-Way Fluid-Structure Interaction

The current codes address only one-way fluid-structure interaction, where fluid forces are applied to the structure but structural deformation does not influence fluid motion. As demonstrated in previous section, fully coupled two-way interaction can lead to significantly different response predictions (Fedra Ashrafzadeh et al., 2025). Incorporating two-way interaction into design provisions would require substantial research and development but could lead to more accurate and reliable designs.

6.3. International Context

The 2015 edition of Publication 123 aligns more closely with international standards such as ACI 350.3 and Eurocode 8. Key areas of convergence include:

- Adoption of Housner's mechanical analog model with refinements for flexible tanks
- Recognition of the distinct dynamic characteristics of impulsive and convective components
- Differentiated response factors for buried versus non-buried tanks
- Specialized load combinations for liquid-retaining structures

However, differences remain, particularly in the treatment of soil-structure interaction and vertical excitation, where Iranian practice lags behind some international standards.

6.4. Implications of the Hyperbolic Pressure Distribution

The shift from a linear to a hyperbolic (nonlinear) pressure distribution in the 2015 edition has significant implications for tank design. The hyperbolic distribution, which is consistent with potential flow theory and experimental observations, concentrates the impulsive pressure more near the base of the tank compared to the linear distribution. This results in:

- *Redistribution of base shear:* While the total base shear may be similar, the distribution of shear along the wall height is modified, with higher shear demands near the base and lower demands near the top.
- *Increased overturning moments:* The lower center of effort for the impulsive force increases the overturning moment at the base, requiring stronger foundation design.
- *Reduced upper wall demands:* The reduced pressure in the upper portion of the wall results in lower moment and shear demands in the upper regions, potentially allowing for reduced reinforcement in these areas.
- *More accurate design:* The hyperbolic distribution provides a more physically accurate representation of the actual pressure distribution, leading to more efficient and reliable designs.

The technical justification for this change is based on more recent experimental and numerical studies of flexible tank walls under seismic excitation, which demonstrate that the impulsive component of hydrodynamic pressure does not vary linearly with height, especially in taller or more flexible tanks. Instead, the pressure increases more rapidly near the base and remains lower in the upper region, which is better captured by a hyperbolic shape derived from potential flow theory with wall flexibility accounted for.

6.5. Practical Design Implications

The cumulative effect of the changes introduced in the 2015 edition has significant practical implications for tank design:

6.5.1. Reinforcement Detailing

The changes in load combinations, response factors, and pressure distributions directly affect the required reinforcement amounts and detailing. For example:

- Higher importance factors for essential tanks may increase reinforcement requirements by up to 15-20%
- The more accurate pressure distribution may lead to higher reinforcement concentrations near the base
- Different reinforcement requirements for buried versus non-buried tanks reflect their distinct behavioral characteristics

6.5.2. Wall Thickness

The revised load combinations and more accurate pressure distributions may influence the required wall thickness. While the 1993 edition often led to conservative wall thickness designs, the 2015 edition allows for more optimized designs based on the actual loading conditions. However, the increased seismic demands for essential facilities may require thicker walls in some cases.

6.5.3. Construction Costs

The 2015 updates are expected to enhance safety and resilience, but may also imply increases in initial construction costs. The extent of cost implications depends on numerous factors, including tank geometry, site conditions, and specific design parameters. A thorough cost-benefit analysis would require detailed modeling of tank configurations, material properties, and design coefficients, which is beyond the scope of the present study.

6.6. Future Research Directions

Based on the analysis presented in this paper, several research directions are identified as priorities for future code development:

- Incorporation of soil-structure interaction provisions to account for the effects of foundation compliance on tank response
- Development of provisions for vertical seismic excitation to address the influence of vertical ground motion on wall stresses and foundation pressures
- Implementation of two-way fluid-structure interaction to capture the coupled effects of structural deformation on fluid motion
- Refinement of performance-based design frameworks for liquid storage tanks
- Comprehensive cost-benefit analysis to quantify the economic implications of enhanced design requirements

7. Conclusion

Water storage tanks represent critical infrastructure that must preserve operational integrity under seismic loading. This study provides a comparative assessment of the 1993 and 2015 editions of Publication 123, tracing the evolution of seismic design provisions and elucidating the fundamental enhancements introduced in the more recent code. The 2015 revision, prompted by empirical lessons from past earthquakes and convergence with international seismic standards, systematically addresses multiple deficiencies inherent in the earlier code. Notable refinements include an expanded seismic zonation framework, from three to four categories, enabling more precise regional hazard characterization; the replacement of a constant importance factor (1.2) with a variable coefficient contingent upon fluid type, tank function, and site location; and the explicit incorporation of burial conditions into the response modification factor, which previously applied a generic value irrespective of embedment. Additionally, a newly formulated effective mass coefficient now accounts for the inertia of the tank wall itself, a contribution that was effectively disregarded in the earlier edition.

Substantial revisions extend to the load combination framework, where the shift from ACI 318-based to ACI 350.3-based factors fundamentally transforms design demands. The 2015 edition expands the combinatorial space from 6 to 10 distinct load cases, introducing previously overlooked scenarios, most notably the standalone thermal combination ($1.25D \pm 1.25T$, Combo 10), which addresses temperature effects acting independently with dead load as a governing condition. Moreover, the 2015 code introduces seismic-dependent freeboard calculations, replaces linear pressure distributions with hyperbolic profiles differentiating impulsive and convective effects, and provides distinct provisions for rectangular versus cylindrical tanks, moving away from generic constants. Practically, these revisions translate into increased reinforcement requirements and a grade upgrade from AII to AIII steel, with bar sizing deliberately balanced to ensure ductility and avert brittle failure. Overall, the 2015 edition reflects a major evolution toward a more refined and reliable performance-based seismic design methodology, highlighting the essential role of contemporary codes in preserving tank resilience.

Nevertheless, the study does not quantify the trade-off between augmented initial construction costs and long-term seismic risk reduction, as such an evaluation would necessitate detailed parametric modeling beyond the current scope; this cost-benefit analysis is thus proposed as a priority for future inquiry. Furthermore, the authors identify the integration of fully coupled two-way fluid-structure interaction, wherein structural response reciprocally affects fluid motion, as a paramount research gap for Iran's seismic code development, necessitating advances in nonlinear analysis and performance-based frameworks, given that current provisions are restricted to one-way coupling.

8. Declaration

The authors acknowledge the use of an artificial intelligence-based language-editing tool solely for improving the English language, grammar, and readability of the manuscript. The authors remain fully responsible for the scientific content, analysis, interpretation, and conclusions presented in this manuscript.

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NOMENCLATURE

Symbol	Description	Symbol	Description
$\boldsymbol{A}$	Design basis acceleration ratio	L	Live load; Length of rectangular tank
$\boldsymbol{A_b}$	Rebar area	l_d	Development length
$\boldsymbol{B, B_i}$	Response factor (general, impulsive)	M	Bending moment
$\boldsymbol{B_c}$	Response factor for convective mass	M_b	Total seismic base moment
$\boldsymbol{C}$	Seismic coefficient	M_{bi}	Base moment from impulsive mass
$\boldsymbol{D}$	Dead load	M_{bc}	Base moment from convective mass
$\boldsymbol{d}$	Rebar diameter	P	Pressure (hydrostatic/hydrodynamic)
$\boldsymbol{d_{\max}}$	Maximum sloshing wave height	P_h	Hydrostatic pressure
$\boldsymbol{E_d}$	Dynamic soil pressure	P_i	Hydrodynamic force from impulsive mass
$\boldsymbol{E_s}$	Lateral soil pressure, Earthquake load	P_c	Hydrodynamic force from convective mass
$\boldsymbol{E_Q}$	Earthquake load	P_w	Lateral force of tank wall vibration
$\boldsymbol{F}$	Fluid load	P_r	Lateral force of tank roof vibration
$\boldsymbol{F_d}$	Hydrodynamic fluid pressure	P_b	Base pressure
$\boldsymbol{F_S}$	Static fluid pressure	q	Distributed load intensity
$\boldsymbol{F_{st}}$	Maximum tension stress (factored)	R	Response modification factor
$\boldsymbol{F_{sb}}$	Maximum bending stress (factored)	T	Temperature load; Period of vibration
$\boldsymbol{f_{st}}$	Maximum tension stress (without factor)	T_i	Period of impulsive mode
$\boldsymbol{f_{sb}}$	Maximum bending stress (without factor)	T_c	Period of convective mode
$\boldsymbol{f_y}$	Yield strength of reinforcement	U	Factored load in strength design
$\boldsymbol{f'_c}$	Compressive strength of concrete	W_i	Weight of impulsive fluid mass
$\boldsymbol{H}$	Soil load	W_c	Weight of convective fluid mass
$\boldsymbol{H_L}$	Liquid height in tank	W_L	Total weight of stored fluid
$\boldsymbol{h}$	Height; Depth		
$\boldsymbol{h_i}$	Height of impulsive mass	Greek Symbols	
$\boldsymbol{h_c}$	Height of convective mass	ρ	Reinforcement ratio
$\boldsymbol{h_w}$	Height to centroid of wall mass	σ	Stress
$\boldsymbol{I}$	Importance factor	ε	Effective mass coefficient of tank wall
$\boldsymbol{K}$	Soil pressure coefficient (e.g., K_a, K_0)	γ	Specific weight of fluid
$\boldsymbol{K_t}$	Stiffens of convective mass	λ	Sloshing frequency parameter
$\boldsymbol{K_d}$	Distribution factor 30	ϕ	Strength reduction factor