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Seismic analysis of tanks and vessels: A comprehensive review of international codes and guidelines

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ABSTRACT

This paper reviews and compares the seismic behaviour and design provisions for ground-supported tanks and vessels, with a focus on widely used international seismic codes, standards, guidelines, and books including NZSEE, API 650, API 620, ASCE 07, IITK-GSDMA, AIJ, and Eurocode 8 (EN 1998–4). Essential analysis considerations for tanks and vessels under seismic forces are examined, with emphasis on fluid-structure interaction. Key topics include the development of simplified mass-spring models, calculation of the base shear, and determination of the period of vibration for tanks and vessels. The study also explores the role of tank wall flexibility and highlights distinctions between ground-supported and elevated tanks, as well as vessels supported by various structural systems. In addition, it provides a review of non-structural components and non-building structures, aligning with the provisions outlined in Chapters 13 and 15 of ASCE 07. The paper further discusses common failure modes in tanks and vessels that have been documented in past earthquake events, offering insights into vulnerabilities and the seismic analysis improvements.

1. Introduction

Tanks and vessels are widely used across industrial and infrastructure systems to store liquids and gases such as water, oil, and chemical substances. Some of these tanks operate at atmospheric pressure, particularly those in the oil industry. Others are designed to resist moderately elevated internal pressures depending on their operational requirements.

Atmospheric-pressure tanks can be classified into several groups based on their location, construction material, support condition, wall rigidity, anchorage type, and aspect ratio. Tanks are commonly grouped into ground-supported, elevated, and leg-supported types. Ground-supported tanks may be placed entirely above ground, partially embedded, or fully buried. Elevated tanks are carried by structural systems that lift the tank above ground level. Spherical tanks, by contrast, are typically supported on steel legs that connect either directly to the foundation or to an intermediate supporting frame.

Based on their support conditions, tanks fall into two main categories: foundation-supported and structure-supported systems. Their bodies may be constructed from metal, reinforced concrete, prestressed or precast concrete, or masonry materials. Metal ground tanks are further classified into anchored and unanchored types. A self-anchored tank resists overturning mainly through its own weight and the hydrostatic weight of the liquid. In contrast, a mechanically anchored tank is connected to the foundation through anchors or straps, which provide additional resistance against wind- and earthquake-induced forces.

Based on their wall rigidity, tanks are classified as either flexible or rigid. They may also be categorised by aspect ratio as slender or broad/squat. Here, H denotes the tank height, D is the tank diameter, and L represents the height of the liquid inside the tank. However, no universal criterion exists for this classification. For example, O'Rourke and So [1] defined $H/D = 0.7$ as the boundary between slender and broad tanks, while Razzaghi and Eshghi [2] considered $H/D > 0.6$ as indicative of tall tanks.

Because H/D does not account for liquid height, recent studies

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Table 1
Reviewed codes, standards, guidelines, and design handbooks.

Full Name	Abbreviation/ Author	Type of Document	Country	Year
Seismic Design of Storage Tanks: 2009 (New Zealand Society for Earthquake Engineering)	NZSEE [30]	National Code	New Zealand	2007
Design of Structures for Earthquake Resistance - Part 4: Silos, Tanks and Pipelines	EN 1998-4 [31]	National Code	Europe	2006
Minimum Design Loads and Associated Criteria for Buildings and Other Structures	ASCE 07–22 [29]	Standard	USA	2022
Design Recommendation for Storage Tanks and Their Supports with Emphasis on Seismic Design (2010 Edition) (Architectural Institute of Japan)	AIJ [32]	Recommendation	Japan	2010
Guidelines for Seismic Design of Liquid Storage Tanks	IITK-GSDMA [33]	Guideline	India	2007
Seismic Analysis and Design of Liquid-Containing Concrete Structures (ACI 350.3–20) and Commentary	ACI 350.3–20 [34]	Standard	USA	2020
Design and Construction of Large, Welded, Low-pressure Storage Tanks	API 620 [35]	Standard	USA	2021
Welded Tanks for Oil Storage	API 650 [36]	Standard	USA	2020
ASME SECTION VIII - Rules for Construction of Pressure Vessels Division 1 and Division 2	ASME [37,38]	Standard	USA	2021
Seismic Evaluation and Design of Petrochemical and Other Industrial Facilities (Third Edition)	ASCE [39]	Design Handbook	USA	2020
Pressure Vessel Design Manual (Fourth Edition)	Moss, D. R., Basic, M. M. [40]	Design Handbook	USA	2012
Pressure Vessel Design Handbook (Second Edition)	Bednar, H.H. [41]	Design Handbook	USA	1990
Design of Pressure Vessels	Gaddam, S.R. [42]	Design Handbook	India	2020

recommend using *HL/D* as a more representative geometric measure, such as in the work of Miladi et al. [3].

Pressurised vessels are similarly categorised by their support conditions. They may rest on legs, skirts, lugs, or rings and can be supported either directly on a foundation or on another structure. Compared with atmospheric tanks, vessels are generally more slender, and their dynamic behaviour varies widely—from nearly rigid to highly flexible.

Because tanks and vessels play a critical role in industrial and infrastructure systems, accurate seismic analysis is essential. Past earthquakes in Italy [4–6], New Zealand [7–9], Japan [10–17], India [18,19] and Turkey [20–27] have shown that these structures are highly vulnerable.

Their seismic behaviour differs from that of conventional buildings. The stored liquid generates hydrodynamic pressures on the walls during shaking, and the structures themselves often have limited redundancy and ductility. Their response becomes even more complex when the flexibility of the supporting structure affects the overall tank–fluid system.

For these reasons, seismic codes assign tanks and vessels lower response-modification factors than buildings while requiring higher importance factors due to the severe potential consequences of failure. As a result, systems with similar dynamic characteristics may be designed for significantly larger seismic forces [28].

Unlike most previous studies, this paper examines tanks and pressure vessels together to enable a coherent comparison of their seismic responses, failure mechanisms, and code provisions. Although their mechanical behaviour differs, an international standard—such as ASCE 07 [29]—classifies both as non-building structures and apply similar design principles, with important distinctions related to fluid–structure interaction and internal pressure effects.

Accordingly, the manuscript focuses on how these codes evaluate seismic demands on tanks and vessels, rather than equating their detailed mechanical behaviour. The review first summarizes previous research, then discusses earthquake-induced failure modes, and finally compares the seismic provisions in the standards listed in Table 1.

While atmospheric tanks and pressure vessels share certain geometric and functional features, they also exhibit key differences that influence seismic assessment. These similarities and distinctions are highlighted to support the practical application of code-based seismic provisions.

2. Historical background and modelling

Seismic analysis of liquid storage tanks and vessels is a major concern in earthquake-prone regions. This literature review first summarises foundational studies on simplified tank–liquid models. It then examines later research that introduced additional complexities, including wall and foundation flexibility, anchorage conditions, and different support systems.

The section also reviews past studies on the common damage mechanisms observed during earthquakes and identifies key design aspects for these structures. Finally, it highlights major research contributions and points out remaining gaps that require further investigation to improve seismic resilience.

A significant distinction exists between atmospheric tanks and pressurised vessels. Atmospheric tanks, typically used for larger volumes of liquid, are generally much bigger than slender pressurised vessels. Consequently, most studies on sloshing effects have concentrated on atmospheric tanks. However, sloshing effects do occur in liquid-containing pressurised vessels, though they are not as significant as those observed in tanks.

2.1. Simplified analytical models

To analyse the seismic behaviour of tanks—including liquid–structure interaction effects—several simplified analytical models have been developed. Under earthquake excitation, the liquid mass is typically divided into two components: the impulsive part, which behaves as a solid mass and moves simultaneously with the tank body due to acceleration, and the sloshing (convective) part, which represents the liquid motion near the surface [43,44].

For tanks with height-to-radius ratios between 0.3 and 3, where H denotes the tank height and r is the tank radius, the initial impulsive and first convective modes collectively account for approximately 85–98 % of the entire liquid mass. The remaining liquid predominantly oscillates in high impulsive modes for tall tanks ($H/r > 1$) and in high convective

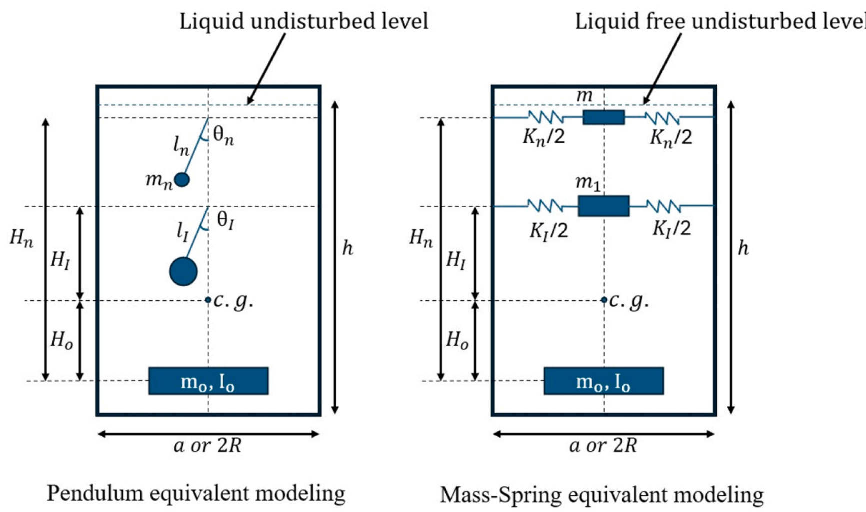


Fig. 1. Mass-spring system and pendulum equivalent modelling (adapted from [43]).

modes for wider tanks ($H/r < 1$). Considering only the initial impulsive and first convective modes yields sufficiently accurate results [45]. Although an infinite number of sloshing vibration modes can be mathematically defined, the first mode contains most of the vibrating mass and is therefore typically regarded as the fundamental sloshing mode in civil engineering seismic applications. Numerous studies have shown that this approximation closely reflects real-world behaviour [46–51].

Convective modes occur at frequencies significantly lower than the impulsive components. These two responses exhibit weak dynamic coupling, and the characteristics of each remain largely unaffected by variations in the other [52]. It is worth mentioning that in Section 14 of this paper, Table 13 presents the difference between the mass participation obtained from the first convective mode and that obtained from the first ten convective modes.

One of the most widely used approaches is the mass–spring analogy, also known as the Equivalent Mechanical Model (EMM) [43]. This method represents tank–liquid dynamics under horizontal excitation through a simplified mechanical system. In the model, the liquid inside the tank is replaced conceptually by an equivalent mass–spring system that captures the main features of the fluid–structure interaction.

The mass–spring system—or a set of pendulums (Fig. 1)—has frequently been used to model liquid motion, particularly sloshing behaviour. This approach was developed and refined through several studies by Graham [53,54], Westergaard [55], Housner [46–48,56–59], Werner and Sundquist [60], and Jacobsen, Hoskins, and Ayre [61–63]. While early studies assumed rigid walls and foundations, later research by Kena [64] and Balendra [65] showed that including structural flexibility significantly affects induced forces and pressures. Many additional studies have also examined the influence of different sloshing modes on tank dynamics [66–94].

The Housner [57] mass–spring model remains the basis of seismic design for storage tanks because of its simplicity. However, it is limited to linear responses, unidirectional excitation, and small vibration amplitudes. Under these conditions, the liquid surface oscillates in a planar manner, referred to as planar sloshing [67,95,96]. This assumption is valid when the seismic excitation is weak, short in duration, and when the tank geometry (H/R ratio), where H denotes the tank height and R is the tank radius, and filling level fall within normal ranges [96,97].

When the excitation is strong, broadband, or long-lasting, or when the tank is partially filled or equipped with a floating roof or flexible baffles, the free-surface motion deviates from the planar state and becomes nonplanar or chaotic [43,98]. Under such conditions, fluid–structure coupling intensifies, and phenomena such as rotary sloshing, breathing vibrations, and secondary resonances may occur—effects that the Housner [57] model cannot capture [99–101].

Therefore, the Housner [57] model is applicable only for preliminary or linear analyses with small-amplitude motions. For strong excitations or complex geometries, advanced multimode models or fully nonlinear Fluid–Structure Interaction (FSI) simulations are required to capture higher-mode contributions, geometric nonlinearities, and strong coupling effects [101–103].

Ching Yu et al. [104] reviewed an analytical solution for the seismic fluid–structure interaction of cylindrical tanks subjected to horizontal excitation. Their work reduced calculation errors found in earlier studies and highlighted the importance of precise modelling for understanding FSI responses.

Although simplified models are practical and widely used in design codes, they cannot capture the full nonlinear and coupled behaviour of tank–liquid systems under strong seismic excitation. Therefore, while they offer useful first-order estimates, more advanced numerical methods—especially fully coupled fluid–structure interaction analyses—are necessary for obtaining reliable predictions under realistic conditions.

2.2. Influence of wall and foundation flexibility

Several studies have investigated the effects of wall and foundation flexibility on the seismic behaviour of tanks. Most of these studies modified the mass-spring model to include wall flexibility. This line of research began with Edwards [105], who examined the effects of wall flexibility on hydrodynamic forces, and was continued by many researchers [46,48,49,56,58,106–111]. These studies demonstrated that in flexible-walled tanks, local accelerations may be higher or lower than those in rigid-walled tanks depending on the location and degree of flexibility [112].

In contrast, in rigid-walled tanks, the entire body experiences the same acceleration magnitude. Furthermore, these studies showed that wall flexibility mainly affects the impulsive mode of vibration and has little influence on the convective mode [113]. Important aspects of the mass-spring analogy, such as the ratio between the convective mass and the impulsive mass volume, as well as the tank's diameter-to-height ratio and its effect on these two mass components, have also been investigated within this framework.

Because many existing design codes idealise tank walls as rigid and rely on simplified mass–spring models, incorporating wall flexibility into future revisions of these codes is essential for achieving a more realistic representation of seismic behaviour. As mentioned earlier, the classical spring–mass analogy was originally developed under the assumption of a rigid foundation, which is not realistic in many practical situations. The seismic response of liquid storage tanks is highly influenced by the interaction between the structure, its foundation, and the supporting soil—known as soil–foundation–structure interaction [114].

Soil Structure Interaction (SSI) can significantly influence the seismic response of tanks [115], as it alters both the effective damping ratio and the natural period of vibration [113,116,117]. While wall flexibility primarily affects the impulsive component of motion, foundation flexibility and SSI considerations impact both the impulsive and convective components [113]. For large, stiff, and heavy tanks founded on soft or cohesionless soils—such as those in coastal and marine deposits—the site-dependent frequency content of ground motion can strongly influence their dynamic response. To address the effects of SSI, numerous researchers have investigated its influence on the seismic behaviour of liquid storage tanks [113,116–143].

Tsipianitis et al. [115] examined the role of SSI on the dynamic response of cylindrical steel tanks with various foundation conditions. Their study demonstrated that neglecting SSI can lead to unrealistic predictions and highlighted the necessity for detailed modelling of the contained liquid and the inclusion of base-isolation systems, particularly for tanks resting on soft soils.

2.3. Anchorage, uplift behaviour, and corrosion

Anchorage plays a critical role in the seismic performance of liquid storage tanks. The basic mass–spring model assumes rigid walls and a rigid foundation, requiring full anchorage of the tank to the foundation. However, full anchorage is often costly for large tanks.

To reduce costs, unanchored tanks are commonly used. These tanks rest on a ring wall without mechanical anchors and must resist uplift forces while accommodating bottom plate deformations [112].

Research on unanchored tanks can be grouped into several categories: experimental investigations such as static and shaking table tests [144–151]; analytical studies on the uplift behaviour (e.g., Clough [146], Wozniak [152], Leon [153], and Paolacci [154–161]); and studies of hydrodynamic pressure distribution under seismic excitation [162–166]. Additional studies have also examined the seismic behaviour of elevated tanks—both steel and concrete—as well as tanks supported on frames or buildings [167–172].

Bakalis et al. [173] analysed the uplift mechanism and seismic response of unanchored liquid storage tanks. Their analytical method identified major differences in stress distribution between anchored and unanchored tanks.

In addition to uplift and anchorage effects, corrosion is another important factor because it reduces stability and buckling capacity [174–176]. Analytical studies complement earthquake observations and help identify solutions—such as seismic isolation systems—that can improve overall resilience.

2.4. Common seismic damage mechanisms in storage tanks and vessels

Earthquake-induced damage to storage tanks and vessels can result in substantial economic losses and severe social and environmental consequences. Therefore, assessing their seismic vulnerability and implementing appropriate protective measures is essential. This section provides an overview of typical earthquake-induced damage mechanisms observed in such systems [9,20,30,112,177,178].

Damage to steel tanks has been documented in numerous major earthquakes, including the 1933 Long Beach, 1952 Kern County, 1964 Alaska, 1971 San Fernando, 1994 Northridge, 1999 Kocaeli, 2003 Bam, 2011 Tohoku, and 2016 New Zealand events [7,8,10,14,15,20,23,179–210]. These observations provide critical information for understanding seismic response and evaluating design standards.

Yazdanian et al. [7] conducted a comprehensive assessment of legged wine storage tanks affected by several significant seismic events in New Zealand, including the 2013 Seddon (Mw 6.5), 2013 Lake Grassmere (Mw 6.6), and 2016 Kaikōura (Mw 7.8) earthquakes. Their dataset included 1512 tanks affected by the 2013 events and 599 affected in 2016. Tank capacity, liquid level, and component damage were identified as key parameters governing seismic performance. Frame and leg elements were found to be the most vulnerable components, and tanks lacking anchorage to concrete slabs displayed erratic behaviour, often resulting in severe damage.

Frezzati et al. [211] examined the seismic guidelines for wine storage tanks in New Zealand by critically reviewing the NZSEE [30] guidelines and comparing them with Eurocode 8 Part 4 (EN 1998–4) [31] and API 650 [36]. Their findings indicated that the NZSEE [30] guidelines do not underestimate seismic demands relative to the other standards; however, they highlighted the need for improved detailing of critical connections and noted that assigning a higher Importance Level (IL2) could enhance the seismic resilience of wine tanks.

Dizhur et al. [177] investigated the post-seismic performance of winery facilities in the Marlborough region following the 2016 Kaikōura earthquake. During this event, more than 2 % of the region's wine production—approximately 5.3 million litres—was lost. Around 20 % of the total tank capacity sustained damage, affecting at least 1000 tanks, of which roughly 150 were irreparable. Observed damage varied with tank configuration (legged or flat), capacity, and fill level. In legged tanks, failures primarily occurred in foundations and supporting structures, whereas flat-bottom tanks—often of larger capacity—experienced shell buckling, damage to the top cone, and failure of connections. Ancillary components, such as access platforms and thermal insulation, also suffered damage. Although the overall performance of tanks during the 2013 and 2016 earthquakes was broadly similar, differences in design and construction quality led to distinct failure patterns.

Au et al. [9] studied the gap between NZSEE [30] guidelines and their practical implementation within the New Zealand wine industry, a discrepancy that became evident following the 2013 Seddon earthquake. They categorised typical damage mechanisms into three main groups. For plinth-mounted tanks, common failures included tensile rupture of anchor bolts due to steel failure, epoxy pullout, or concrete cone breakout; shear failure of shear bolts; and side-slippage resulting from inadequate friction between the tank base and plinth. Additional damage comprised localised deformation at the bottom-to-shell junction ("knuckle-squash") due to insufficient compression load paths, diamond-shaped or "elephant-foot" buckling caused by inadequate wall thickness, and localised buckling of the tank skirt near its support seat.

For base-frame-mounted tanks, failure modes included bending of legs in unbraced frames with semi-rigid connections, buckling of braces in braced frames due to excessive slenderness, and fracture or bending of adjustable legs caused by floor slope. Strap-joint deformation or welding failures—particularly in eccentric connections—sometimes resulted in rupture of the tank wall and leakage. Ancillary structures such as shared catwalks and pipelines also introduced unintended load paths, causing localized damage where they connected to tanks.

Complementing these case-based studies, Eshghi and Razzaghi [212] investigated two unconfined cylindrical steel tanks damaged during the 2006 Silakhor earthquake using near-site accelerometer records, numerical time-history analysis, and analytical relations including NZSEE 1986. Their results showed significant differences between various analytical relations in estimating maximum wave height, with NZSEE [30] providing more conservative predictions. While numerical shell deflection for wide tanks agreed with theoretical models and field observations, tall tanks exhibited axial stresses exceeding theoretical estimates, indicating possible shell buckling. Foundation flexibility was found to have little effect on vertical displacement, although axial stresses were higher for rigid foundations.

Rotter [213] provided a broader perspective on the structural behaviour and design of cylindrical metal bins, silos, and tanks under axial compression and internal pressure. Focusing on elephant-foot buckling as a critical failure mode, the study proposed design formulations to enhance the safety and reliability of such structures.

In a complementary numerical approach, Bovo et al. [214] evaluated

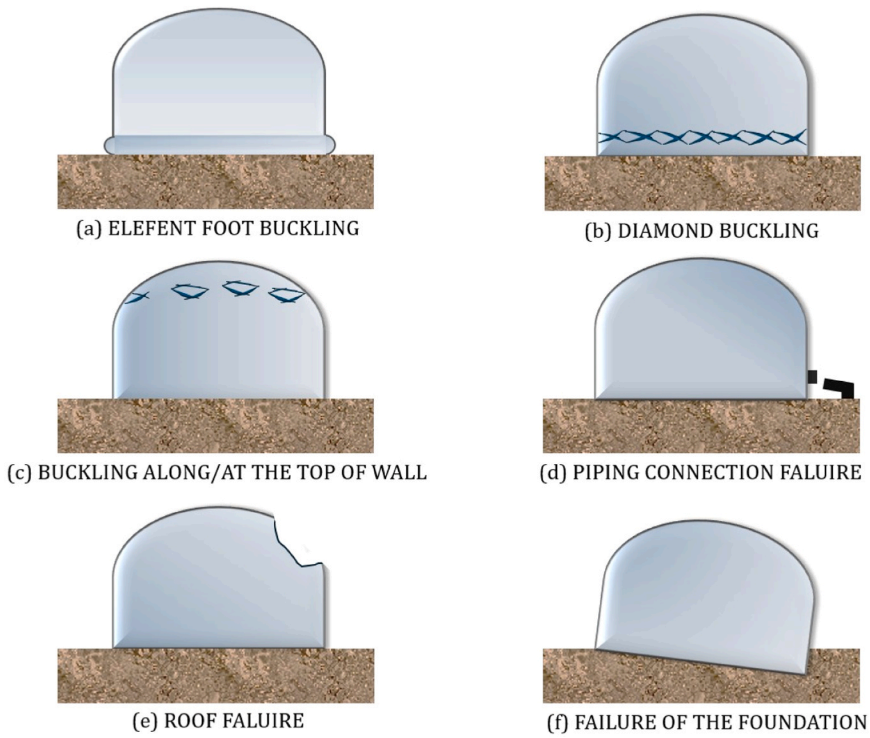


Fig. 2. Frequent observed damages in tanks in past earthquakes (adapted from [112]).

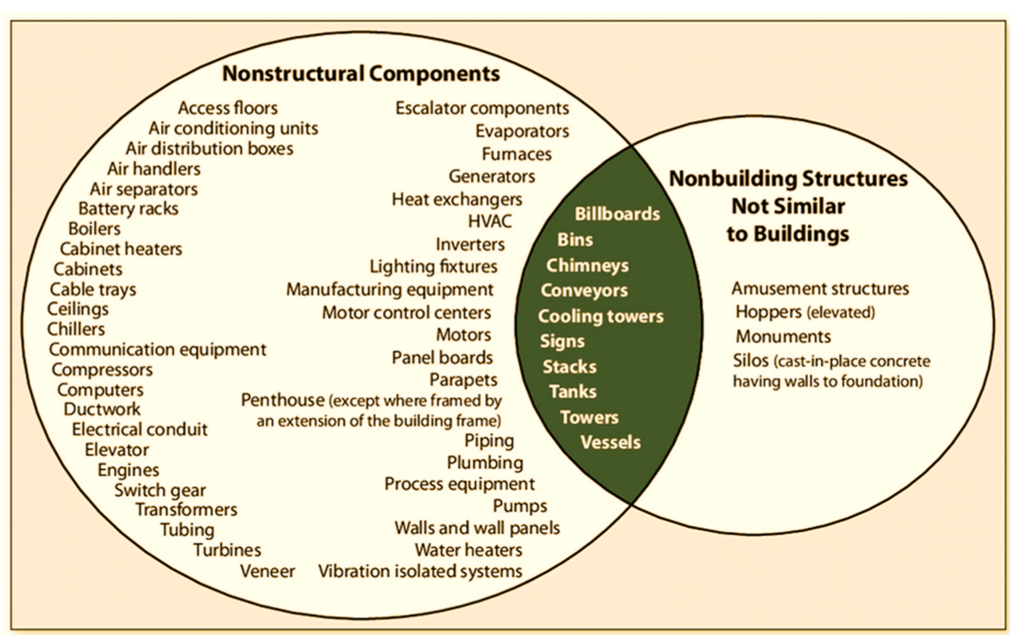


Fig. 3. Intersection of non-building and non-structural components from Bachman and Dowty [315].

the seismic vulnerability of stainless-steel legged tanks by performing Incremental Dynamic Analyses (IDAs) on finite element models of 140 tanks. Fragility curves based on lognormal cumulative distributions were developed for different limit states, demonstrating the significant influence of sloshing effects and soil–structure interaction on seismic response.

A review of existing design standards shows that only the NZSEE guideline [30] includes a dedicated chapter describing typical failure modes of atmospheric tanks based on post-earthquake observations. Additional discussions appear in Seismic Design of Tanks [112] and Seismic Evaluation and Design of Petrochemical and Other Industrial Facilities [39]. According to the first chapter of NZSEE [30], the most frequently observed types of earthquake-induced damage in tanks include:

- Shell buckling of the tank wall [140,213,215–227], including elephant-foot, diamond, and knuckle-swash patterns. Buckling mode depends on relative shell thickness: thicker shells typically exhibit inelastic (elephant-foot) buckling, whereas thinner shells are prone

to elastic (diamond-shaped) buckling [176,228–230] (See (a), (b), and (c) in Fig. 2). (Fig. 3).
- Damage to the tank roof and upper wall caused by waves generated from fluid sloshing (See (e) in Fig. 2),
- Tearing of the tank wall and stress concentration near the tank supports at the foundation or base,
- Sliding of ground-supported tanks when horizontal seismic forces exceed the available frictional resistance,
- Damage to rigid connections of pipes and attached equipment, weld cracking between the floor and shell, and differential settlement of the foundation due to partial uplift of the tank floor in self-restrained or partially restrained tanks (See (d) in Fig. 2),
- Foundation failure (See (f) in Fig. 2),
- Post-earthquake fires may occur due to connection failures or leakage of flammable fluids from the roof area, particularly in tanks with floating roofs.

The Seismic Evaluation and Design of Petrochemical and Other Industrial Facilities [39] manual identifies several additional damage modes, including:
- Splitting or leakage of bolted and riveted shells due to excessive hoop tension.
- Failure of non-ductile anchorage systems, leading to tearing of the shell or anchor rupture;
- Tearing at over-constrained attachments, such as stairways, ladders, or piping simultaneously fixed to both the shell and foundation. Similar tearing may occur in interconnected tanks experiencing differential motion.

It should be noted that diamond-shaped buckling patterns typically develop near the tank top under seismic loading, whereas bottom-shell diamond buckling is more characteristic of extremely thin-walled tanks, such as those commonly used in the beverage industry.

In addition to ground-supported tanks, leg-supported vessels exhibit distinct seismic behaviour. When properly designed and constructed, these systems can sustain high internal pressures and perform satisfactorily under moderate shaking. However, historical evidence shows that earthquake-induced damage primarily occurs in the supporting structures rather than in the vessels themselves [39]. Gonzalez [178], in his review of the 2010 Maule earthquake, documented typical damage patterns in leg-supported steel tanks. Because the shell is often connected directly to the supporting legs, local denting and buckling develop near the leg attachments and bases under strong shaking—a behaviour known as the "weak-shell/strong-leg" phenomenon. The main vulnerabilities include [39]:
- Unreinforced cutouts in the vessel skirt,
- Rigidly attached piping systems,
- Inadequate strength or ductility of the anchorage,
- Slender supporting piers,
- Overturning due to an insufficient number of legs or bracing members.

A comparative study of the NZSEE [30] and field observations from recent earthquakes in New Zealand (e.g. Seddon 2013, Kaikōura 2016) and other countries shows that although the code has been successful in identifying the main failure patterns, it shows limitations in representing the actual behaviour of tanks during earthquakes, particularly regarding component interaction, construction detailing, and unintended load paths. In field observations, failures have often occurred in a continuous and dependent manner, such that the failure of the supporting foundations or frames has usually been accompanied by a rupture of the shell and complete leakage of the contents [7] [177].

Also, damage to supporting components, platforms and common piping has led to secondary failures at the tank shell-wall joints [9]. These continuum behaviours indicate that the actual seismic performance of tanks is the result of the interaction between different failure mechanisms, while the NZSEE [30] models each failure mode separately without considering their simultaneous effect. Furthermore, the code usually ignores the influence of construction quality, type of connections, and unintended load paths caused by peripheral components such as pipes and joint platforms, while field observations have proven that these factors are among the most important causes of damage propagation in industrial tanks [9].

As a result, although NZSEE [30] is efficient at the conceptual level and identifies the types of failures, it needs to be revised in the sections related to the execution details, component interaction, and capacity-based design principles to fully match the actual behaviour of the structure in severe earthquakes.

3. Structural seismic mitigation systems and their treatment in design codes

Although seismic design standards aim to ensure satisfactory structural performance under specified earthquake levels, they cannot fully eliminate damage or prevent all adverse performance outcomes [112]. Field evidence shows that residual damage frequently stems from factors beyond the scope of design provisions, such as poor construction quality, defective materials, and inadequate detailing. To address these deficiencies, numerous studies have proposed a range of mitigation strategies [231–239]. Among these, the implementation of base isolation systems [118,119,128,239–275]—intended to decouple the tank from ground motion and reduce both impulsive and convective responses—represents the most extensively studied approach in this field.

Base isolation increases the fundamental vibration period and effectively shifts the structural response away from the dominant frequencies of typical ground motions. As a result, the transmitted accelerations and base shear forces are substantially reduced [235,257,267, 276–283].

The literature on isolated tanks spans a broad range of isolation devices and configurations including rubber-based isolators, sliding systems, and hybrid arrangements [118,284,285]. Experimental and numerical investigations consistently demonstrate that isolation substantially decreases impulsive hydrodynamic pressures and base shear compared with fixed-base tanks [118,285].

Analytical studies indicate that Lead–Rubber Bearing (LRB) systems substantially reduce impulsive hydrodynamic pressures while exerting minimal influence on convective responses, due to their much longer natural periods [128]. Both LRB and Friction Pendulum Bearings (FPB) are shown to be highly effective for cylindrical tanks, providing significant period elongation and energy dissipation through hysteretic deformation [252,286,287].

Despite their advantages, base-isolated liquid-storage tanks introduce several design challenges that extend well beyond simply reducing seismic demand. A critical issue is the accommodation of potentially large lateral displacements that develop at the isolation interface during strong ground motions. Such displacements necessitate careful detailing of attached systems—including piping networks, instrument lines, and auxiliary components—to prevent rupture or malfunction. Flexible couplings, expansion joints, and seismic separation gaps are typically required to maintain functional integrity during strong shaking. Furthermore, environmental actions such as wind loads, buoyancy effects, and tsunami-induced pressures must be incorporated into the isolation design, particularly for tanks located in coastal or offshore regions [128,256,262,263,288,289].

For large-diameter storage tanks, the design of an appropriate foundation system becomes increasingly complex due to the distribution of vertical loads across multiple isolators. To address load-path irregularities and maintain uniform stress distribution to the supporting soil, innovative foundation configurations such as continuous ring beams or interconnected pedestal systems have been proposed [234,290].

The long-term viability of isolated tanks has been demonstrated through numerous full-scale experiments and post-earthquake case histories, which consistently report reduced structural demand and improved performance relative to non-isolated tanks [239,285, 291–294]. However, despite these benefits, base isolation is still not universally adopted, partly because of higher initial costs and the specialized design expertise required [294]. Current research efforts focus on developing more cost-efficient devices, refining isolator characteristics for various tank geometries, and exploring adaptive or semi-active mechanisms capable of adjusting their mechanical properties in response to changing ground motion intensity or fluid–structure interaction conditions [295]. As these technologies mature, their use is expected to expand, contributing to the enhanced seismic resilience of critical storage infrastructure worldwide [259,296].

Tsipianitis et al. [297] investigated the influence of different damping modelling approaches on the seismic vulnerability of isolated liquid storage tanks equipped with Single Friction Pendulum Bearings (SFPBs). Their results showed that Rayleigh damping is the most appropriate method for modelling isolator damping in base-isolated steel tanks. In another study, Tsipianitis et al. [298] examined the seismic performance of base-isolated tanks fitted with secondary linear viscous dampers, showing that supplemental damping significantly improves seismic resilience—particularly at high Peak Ground Acceleration (PGA) levels—and identifying optimal damping values for enhanced performance.

The use of energy-dissipation devices [298–308]—along with dissipative anchors and connectors— [126,239,292,293] has emerged as an additional strategy for enhancing seismic performance. Energy dissipation systems provide an alternative or complementary means of controlling seismic response. Viscous fluid dampers, metallic yielding devices, and viscoelastic layers are commonly used to absorb part of the input energy, thereby reducing the stresses transmitted to the tank wall [118].

The integration of supplemental damping mechanisms into liquid storage tanks offers a protection strategy fundamentally distinct from period-modification approaches. By dissipating part of the input energy before it reaches the primary structure, these devices limit deformation and acceleration demands on both the tank shell and the stored liquid, improving the overall robustness of the system against strong earthquakes [115,233,308,309]. Various families of damping devices have been adapted for tank systems. Viscous dampers, which generate forces proportional to velocity, can simultaneously moderate impulsive structural behaviour and long-period sloshing. Metallic yielding devices—such as BRBs and ADAS components—provide hysteretic energy dissipation and have been shown to reduce overturning tendencies and base shear demands in cylindrical tanks [233,307,310]. Frictional damping units, which rely on sliding between surfaces, constitute another practical option, offering significant energy dissipation within relatively compact configurations. Their effectiveness depends strongly on tank proportions and support conditions: slender tanks may incorporate vertically arranged dampers as part of stiffening frames or outrigger systems, while wider tanks can utilize horizontal damping mechanisms near the base to limit uplift or enhance the performance of partially restrained systems [305,306].

In tanks equipped with floating roofs, dedicated damping mechanisms have been used to control differential motion between the roof and shell, reducing risks such as seal degradation or instability associated with frictional interactions. Because liquid-level variations alter the dynamic characteristics of tanks, some studies have proposed adaptive or semi-active damping systems capable of modifying their mechanical properties in real time, although such solutions require more complex control hardware and long-term maintenance strategies [304]. A key advantage of damping technologies is that they can often be integrated without major geometric modifications, making them suitable for retrofitting and for installations where isolation systems are impractical. Nevertheless, several design issues must be addressed: dampers introduce concentrated forces at their connections, can generate local stress concentrations, and often exhibit strongly nonlinear behaviour, which necessitates advanced analytical procedures to ensure accurate prediction of seismic performance [303,311].

Both laboratory experiments on scaled tanks and detailed numerical studies consistently show that damping devices can substantially reduce hydrodynamic pressures, sloshing amplitudes, and overall seismic demands. Field implementations—especially in high-risk facilities such as LNG terminals—provide additional evidence of their long-term performance and operational requirements [302,312]. Current research directions include developing more cost-effective damping solutions, designing systems capable of addressing multiple hazards simultaneously, and applying emerging materials such as shape-memory alloys or magnetorheological fluids to create devices with enhanced adaptability and energy absorption characteristics [298].

In parallel with these developments, a growing body of research has examined the limitations of traditional anchor-bolt systems. Although anchors are intended to prevent uplift, they contribute little to system ductility, and failures at anchor connections have been repeatedly reported. Studies show that conventional anchorage can lead to brittle response modes, and only a small number of bolts is needed to prevent excessive uplift, limiting their potential as energy-dissipating elements [126]. Innovative concepts—such as the NWS-TADAS device developed in response to observed anchor failures in recent earthquakes—have demonstrated stable hysteretic behaviour over multiple cycles, offering an effective alternative to rigid anchorage. Similarly, anchor bolts fabricated from shape-memory alloys have shown promise in reducing shell damage and anchorage distress during retrofits. Despite these advances, uncertainties remain regarding the role of mechanical anchorage in resisting sliding, suggesting that current code provisions may be either overly conservative or insufficiently detailed [291,300, 301]. It is worth mentioning that Section 14 of this paper discusses two types of anchorage systems—elastic anchorage and ductile tension-yielding hold-down bolts—and examines their influence on the design seismic force of tanks according to the NZSEE [30] provisions.

Other measures include increasing the thickness of bottom and annular plates, strengthening anchorage systems to improve ductility, and optimising tank geometry by adjusting the height-to-diameter ratio [112].

From a typological perspective, most investigations focus on atmospheric storage tanks, whereas fewer studies address elevated atmospheric [128,237,238,247,248,300,301] or pressurised and industrial tanks, such as LNG [235,242,262,266,275,291,307,308], CO_2 vessels [273], and petrochemical vessels [274].

Collectively, the literature demonstrates a gradual evolution from traditional base-isolated configurations toward integrated hybrid isolation–damping concepts in recent studies [231,234,273,299,304–307]. It reflects an emerging design philosophy that seeks to combine displacement reduction with enhanced energy absorption for superior seismic resilience and control of fluid–structure interaction effects.

Despite significant advances in isolation and damping technologies, international design provisions for liquid-storage tanks remain relatively conservative in their treatment of such systems. Documents such as API 650 [36], Eurocode 8 Part 4 [31], ASCE 7 (through its nonbuilding-structure provisions), and the NZSEE [30] recommendations primarily assume fixed-base behaviour and linear-elastic analysis, and they do not provide detailed tank-specific analytical procedures for fully base-isolated or heavily damped tank–fluid–soil systems. In many cases, the influence of isolation or supplemental damping can only be represented indirectly—typically by modifying effective damping ratios or design response spectra using the general rules for seismically isolated structures, rather than through explicit modelling guidance tailored to tanks. For example, Eurocode 8 Part 4 [31] refers the design of seismically isolated tanks and silos to the generic isolation provisions of Eurocode 8 Part 1, while its own annexed procedures for tanks remain based on linear single-degree-of-freedom impulsive–convective models. Likewise, API 650 [36] provides equations for impulsive and convective period estimation and base shear calculation, but Annex E expressly

Table 2
Support type categories for vessels.

Support type	Condition	
Skirt Supports	Cylindrical	
	Conical	
	Pedestal	
	Shear Ring	
Leg Supports	Braced	Cross Braced (Pinned and Unpinned)
		Sway Braced
	Unbraced	
Stub columns	Saddle Supports	
	Lug Supports	
	Ring Supports	
Combination Supports	Lugs and Legs	
	Rings and Legs	
	Skirt and Legs	
	Skirt and Ring Girder	

Table 3
Application range of each code/standard in terms of tanks/vessels shape.

Code	Geometry of Structure	Type of Structure
NZSEE [30]	Rectangular, Circular, Other Geometries.	Ground-Supported RC/PSC Tanks Ground-Supported Steel Tanks Elevated Tanks on Shaft-Type Tower Elevated Tanks on Frame-Type Tower
EN 1998-4 [31]	Rectangular, Circular	Underground Tanks
AIJ [32]	Circular, Spherical	Ground-Supported RC/PSC Tanks Ground-Supported Steel Tanks Elevated Tanks on Shaft-Type Tower Underground Tanks Spherical Leg-Supported Tanks
ASCE 07–22 (Chapter 15) [29]	Different Shapes	Non-Building Similar to Buildings Non-Building Not Similar to Buildings (e.g. Vessels, Boilers and Tanks)
ASCE 07–22 (Chapter 13) [29]	Different Shapes	Non-structural Components (e.g. Vessels, Boilers and Tanks Not within scope of chapter 15)
IITK-GSDMA [33]	Rectangular, Circular	Ground-Supported RC/PSC Tanks Ground-Supported Steel Tanks Ground-Supported Masonry Tanks Elevated Tanks on Shaft-Type Tower (Steel/RC/Masonry Shaft) Elevated Tanks on Frame-Type Tower (Steel/RC frame) Underground Tanks (Steel/ RC)
ACI 350–3–20 [34]	Rectangular, Circular	Ground-Supported RC/PSC Tanks Pedestal Mounted RC/PSC Tanks
API 620 [35] (Annex L)	Circular	Ground-Supported Steel Tanks (Max internal pressure 130)
API 650 [36] (Annex E)	Circular	Ground-Supported Steel Tanks (Max internal pressure 17)
ASME [37,38]	Different Shapes	Pressurised Vessels

states that tanks designed using base isolation or energy dissipation systems are beyond the scope of that annex. In a similar way, the NZSEE [30] storage-tank recommendations and the tank-related nonbuilding provisions of ASCE 07 [29] focus on conventional anchored or unanchored configurations and offer little explicit guidance on modelling coupled fluid–isolator–soil interaction.

4. Tanks and vessels geometry and type

Tanks and vessels can be constructed in various shapes and with different supporting conditions. The most common shapes for tanks are circular, rectangular, and, in some cases, spherical. Depending on their installation, they can be built as ground-supported, elevated, or underground tanks.

The most common vessel geometries include spherical, cylindrical, and non-uniform cylindrical forms in elevation. Vessels can also be classified according to the direction of applied pressure as being under internal or external pressure [313,314]. Table 2 presents different types of support systems used for vessels [40], while Table 3 outlines the application range of each code and standard with respect to tank and vessel shapes and types.

Table 4
Non-structural or non-building tanks and vessels recommendation from Bachman and Dowty [315].

Item	Chapter of ASCE 07 [29]	Description
Tanks	Chapter 13	Small tanks (1.524 m (5 feet) or less in diameter), shop fabricated and transported to the site in one piece
	Chapter 15	Large tanks (greater than 1.524 m (5 feet) in diameter) constructed at the job site
	Chapter 13 or Chapter 15	Large tanks supported within a building or on the roof may require seismic forces to be determined in accordance with Chapter 15, which refers to Chapter 13 (depending on their weight in relation to the supporting structures).
Vessels	Chapter 13	Short vessels (less than 3.048 m (10 feet) tall) typically fit within a building
	Chapter 15	Tall vessels (more than 3.048 m (10 feet) tall) are most likely to be considered as nonbuilding structures. Typically, Power Boilers Vessels, Typically Free-standing Vessels.
	Chapter 13 or Chapter 15	Vessels supported within a building or on the roof may require seismic forces to be determined in accordance with Chapter 15, which refers to Chapter 13, (depending on their weight and length in relation to the supporting structures).

Among the above-mentioned codes and standards, ASCE 07 [29] includes two separate chapters that address tanks and vessels. Chapter 15 of ASCE 07 [29] covers non-building structures and distinguishes between two categories: non-building structures similar to buildings and those not similar to buildings. Many types of industrial facilities fall within the scope of this chapter, including tanks and vessels, which are classified as non-buildings not similar to buildings and are specifically addressed in Clause 15.7 of ASCE 07–22 [29]. In addition, Chapter 13 of ASCE 07–22 [29], which covers non-structural components, provides detailed prescriptions for seismic force evaluation and design of vessels and tanks that do not fall within the scope of Chapter 15 of ASCE 07 [29]. Several studies [315,316] have been carried out to identify the key differences between non-structural components and non-building structures.

Large pressure vessels and large boilers should be analysed and designed to meet the seismic requirements of Section 15.7, which are much more comprehensive than those found in Chapter 13. The definition of “large” should be determined in consultation with the nonbuilding structures design community [316]. Table 4 presents non-structural or non-building tanks and vessels recommendation.

It is worth mentioning that certain cases exist where a non-building structure may be supported by another structure, forming a combined structural system. In some situations, such configurations may resemble non-structural components, and it is crucial to recognise that, in such cases, both the supporting element and the supported structure must be treated consistently — either as non-structural components or as non-building structures. Since a wide range of structures can fall into this category, Table 5 presents this classification specifically for tanks and vessels under different scenarios based on ASCE 07–22 [29]. Because rigid and flexible structures are treated differently and must be analysed according to different chapters of ASCE 07 [29], it is important to review the threshold values reported in previous studies and literature. In [40], vessels are categorised based on their fundamental mode of vibration into four groups:

- Vessels with vibration periods smaller than 0.3 s are considered rigid.
- Those with periods between 0.3 and 0.75 s are considered semi-rigid.
- Those with periods between 0.75 and 1.25 s are considered semi-flexible.

Table 5
Non-structural or non-building tanks and vessels in ASCE 07 [29].

Type of Structure		Condition					Analysis and Design With
Non-building Structures	Similar To Buildings	Moment-Resisting Frames (Steel or Concrete) Supporting Horizontal Vessels, Braced Frames (Cross-Braced or Chevron-Braced) Supporting Horizontal Vessels. (Such structures can be up to four or five levels high.)					Chapter 15
	Not Similar to Buildings	Horizontal Rigid Vessels, Supported on Short/Stiff Piers ($T_p \leq 0.06$ s)					Chapter 15
		Different Flat Bottom Tanks Supported at or Below Grade					Chapter 15
		Skirt-Supported Vertical Vessels, Spheres on Braced Legs, Horizontal Vessels on Long Piers					Chapter 15
		Combination Structures	$W_p \leq 0.25$ $(W_s + W_p)$	Rigid/Flexible		Tall Vertical Vessel or Tank Supported Above Grade on a Braced or Moment-Resisting Frame.	Chapter13
			$W_p > 0.25$ $(W_s + W_p)$	Flexible	$T_p > 0.6$ s		Chapter 15
				Rigid	$T_p \leq 0.6$ s		Chapter 13 (with Chapter 15 R value)
Non-Structural Components		Horizontal Vessels Supported on a Structure $W_p \leq 0.25$ $(W_s + W_p)$					Chapter 13
		Other Types of Electrical and mechanical equipment supported within a structure.					Chapter 13

T_p= Non-building structure fundamental period of vibration, W_p= Weight of non-building structure, R= Behaviour factor, W_s= Weight of supporting structure for non-building structure.

Table 6
Differences among API 620 [35], API 650 [36], and ASME VIII Division 1 and Division 2 [37,38].

	API 650 [36]	**API 620** [35]	**ASME VIII Division 1** [37,38]	**ASME VIII Division 2** [37,38]
Pressure	Up to 17.2 kPa (Atmospheric Pressure)	Up to 103.4 kPa	Above 103.4 kPa (High-Pressure)	
Temperature	Ambient Temperature	Down to −198°C	High and Low Temperatures	
Tank Shape	Cylindrical (Flat-Bottomed)	Spherical or Cylindrical (Not Flat-Bottomed)	Broad Range	
Materials	Carbon Steel, Stainless Steel	Carbon Steel, Stainless Steel, Nickel Alloys	Broad Range: Carbon Steel, Alloys	
Design Method	Design-By-Rule	Design-By-Analysis	Design-By-Rule	Design-By Analysis/ Design-By-Rule
Thickness	Thinner, Rule-Based	Thicker, Based on Stress Analysis	Broad Range	
Industry	Oil, Gas, Petrochemical Storage	Oil, Gas, Petrochemical Storage	Chemical, Power, Refineries (Pressure)	

- Vessels with periods greater than 1.25 s are considered flexible.

Additionally, ASCE 07 [29] defines 0.6 s as the threshold between rigid and flexible conditions. It should also be noted that several parameters can affect a vessel's fundamental period of vibration.

It is evident that pressure vessels exhibit different vibration periods when full compared to when empty. Furthermore, since these structures often operate under extreme temperature conditions, their modulus of elasticity may be affected, which in turn alters the vessel's vibration period. In addition, because ageing can influence the condition of a vessel, it is essential to consider whether the vessel is new or corroded when evaluating its vibration period [40].

Bender [41] highlighted that pressure vessels should be analysed and designed to resist earthquake forces. To evaluate these forces, simplified approaches, such as the equivalent lateral force method, can be used. The dynamic response of vessels—strongly influenced by their rigidity or flexibility—has a significant effect on the equivalent lateral seismic forces. A flexible vessel behaves as a distributed system with multiple degrees of freedom, leading to an indefinite number of vibration modes; however, in most practical cases, the first three modes of vibration dominate the response. In contrast, a rigid vessel subjected to an earthquake undergoes no deformation; instead, the entire structure experiences a horizontal inertial force acting through its centre of gravity.

There is a clear distinction between API 650 [36] and API 620 [35], which users should recognise when applying them. The main difference lies in the maximum internal pressure. API 650 [36] covers tanks operating at pressures close to atmospheric pressure (i.e., internal pressures not exceeding the weight of the roof plates), although higher internal pressures are permitted when additional requirements specified in Annex F of API 650 [36] are satisfied. In contrast, API 620 [35] applies to tanks operating at higher internal pressures—up to 103.4 kPa.

What distinguishes the ASME VIII [37,38] code from API 620 [35] is both its range of applications and its maximum internal pressure limits. API 620 [35], as indicated by its title (American Petroleum Institute), primarily covers the oil industry with limited maximum pressures, whereas the ASME [37,38] code applies to a broader spectrum of high-pressure vessels, including boilers, and chemical and petrochemical installations. Table 6 summarises the differences among these codes and their respective application ranges.

It is worth noting that while API 650 [36] and API 620 [35] each contain a dedicated Annex for seismic force evaluation, the ASME [37,38] code does not include a specific seismic evaluation procedure. However, the loading sections of ASME VIII Division 1 and Division 2 [37,38] require that vessels and their supports be analysed and designed for seismic forces in accordance with an accepted seismic code such as ASCE 07 [29]. Limited damage—such as buckling of the supporting skirt or shell due to a severe earthquake—is considered acceptable. Nevertheless, the anchor bolts are regarded as the most logical and effective components for absorbing earthquake energy and preventing major failure [317].

5. Material selection with a seismic approach

Tanks and vessels can be constructed from a wide range of materials. The most common materials used for tank construction are steel, reinforced concrete, and masonry, whereas vessels are primarily made of carbon steel, stainless steel, aluminium, nickel alloys, and composite materials [40], [42].

Although seismic codes establish minimum requirements for seismic demand evaluation and design criteria—such as wall thickness and

Table 7
Scope of application of seismic codes based on material selection.

Code	Material
NZSEE [30]	S, RC, PC
EN 1998-4 [31]	S, RC, PC
ASCE 07–22 [29]	S, RC, PC
AIJ [32]	S, RC, PC
IITK-GSDMA [33]	M, PC, RC, S
ACI 350–3–20 [34]	RC, PC
API 620 [35]	S
API 650 [36]	S, Al
ASME VIII [37,38]	Various Items

Table 8
International codes and design handbooks for tanks and vessels with focus on materials chapters.

Code/Book	Chapter #	Scope
NZSEE [30]	Not Exist	Material selection chapter does not exist
EN 1998-4 [31]	Not Exist	
ASCE 07–22 [29]	Not Exist	
IITK-GSDMA [33]	Not Exist	
API 620 [35]	Chapter 4	Exist, but its general provisions for the material selection
API 650 [36]	Chapter 4	
Pressure Vessel Design Manual (Illustrated procedures for solving major pressure vessel design problems) [318]	APPENDIX H, APPENDIX I	
ASME VIII Division 1 and Division 2 [37,38]	Several Chapters	
ASCE- Seismic Evaluation and Design of Petrochemical and Other Industrial Facilities [39]	Chapter 5	Seismic Provisions
AIJ [32]	Chapter 2	

seismic detailing—to ensure the safety of tanks and vessels during earthquakes, it remains essential to review the codes and references related to the materials specified in these standards.

From a seismic analysis standpoint, the materials used should exhibit ductility, possess adequate toughness, and remain functional at the design temperature of the structure, which may involve extreme thermal conditions. Table 7 summarises the scope of application of various seismic codes from the perspective of materials used.

In Table 7, the abbreviations M, S, RC, AL, PC, and P refer to Masonry, Steel, Reinforced Concrete, Aluminium, Prestressed Concrete, and Plain Concrete, respectively. For example, API 650 [36] covers the design of steel and aluminium tanks.

As previously noted, Table 7 addresses the application range of each code based on the type of material used in the tank but does not specify detailed requirements such as the grade of the material itself. Table 8 provides a more comprehensive examination of the various sections of the standards and related references to determine whether these documents include explicit requirements for selecting material types.

Chapter 4 of API 650 [36] and API 620 [35] presents general provisions for materials used in tanks (not from a seismic perspective). These codes provide guidelines covering plates, sheets, piping, forgings, flanges, bolting, welding electrodes, and gaskets. Owing to the numerous tables and detailed information contained in these standards, readers of this study are advised to consult the original sources when necessary.

AIJ [32] document — Design Recommendation for Storage Tanks and Their Supports with Emphasis on Seismic Design, 2010 Edition — furnishes material recommendations and specifications tailored to the structural design of various types of storage tanks and their supporting systems. According to these guidelines, the permitted materials include steel, aluminium, timber, concrete, and Fiber-Reinforced Plastic (FRP).

Chapter 5 of ASCE [39] deals with material selection and is particularly significant because it addresses the seismic evaluation of petrochemical structures. In this chapter, the primary objective of material selection for seismic applications is to ensure ductile performance of earthquake-resistant systems. This provision applies to leg-supported tanks and those supported by other structures, although it does not specifically address the material composition of the tank body itself.

The ASCE [39] recommendations primarily follow the seismic design guidelines for steel structures in high seismic regions, as provided by AISC 341 [319] and AISC 360 [320], summarised as follows:

- AISC 341 [319] permits the use of various structural steel specifications; however, not all section shapes are readily available, so verification of material availability is essential prior to specification.
- Bolted connections within seismic load-resisting system joints must comply with AISC 341 [319], utilising high-strength bolts in pretensioned bearing joints.
- Compliance with slip-critical requirements in accordance with AISC 360 [320] Section J3.8, using a Class A surface for faying surfaces in these joints, is mandatory.
- Weld filler materials for members and connections resisting seismic forces must produce welds with a minimum Charpy V-Notch toughness of 20 ft-lb (27 J) at 0 °F (−18 °C).
- Demand-critical welds require filler metals capable of providing Charpy V-Notch toughness of 20 ft-lb (27 J) at −20 °F (−29 °C) and 40 ft-lb (54 J) at 70 °F (21 °C) under specified conditions.
- For structures with service temperatures below 50 °F (10 °C), the qualification temperature must be reduced by 20 °F (11 °C) above the lowest anticipated service temperature.

Additionally, ASCE [39] explicitly states that the design, quality, fabrication, and erection of steel structures shall conform to the stipulations provided in the following references, depending on their applicability:

- IBC [321] (International Building Code),
- ASCE 07 [29] (American Society of Civil Engineers 7),
- AISC 360 [320] (American Institute of Steel Construction 360),
- AISC 341 [319] (American Institute of Steel Construction 341),
- AISI S100 (American Iron and Steel Institute S100),
- ASCE 8 [322] (American Society of Civil Engineers 8),
- ASCE 19 [323] (American Society of Civil Engineers 19).

Furthermore, the Handbook [40], specifically Chapter 11, provides tables describing consumable materials and their mechanical specifications; however, these provisions do not address seismic applications.

A review of the material-related provisions in international standards shows that, despite the critical influence of material behaviour on the seismic response of tanks and vessels—and despite extensive evidence of their vulnerability during earthquakes, especially when operating under high internal pressures or extreme temperature conditions—explicit seismic requirements for material selection are only minimally addressed. While current standards provide detailed and rigorous prescriptions for material strength, fabrication, and service performance, they offer very limited and often insufficient guidance regarding ductility, toughness, fracture resistance, and low-temperature behaviour under cyclic loading. This gap is significant considering the sensitivity of thin-walled liquid-containing structures to material-dependent failure mechanisms such as buckling, fracture, and fatigue under seismic actions.

6. Seismic loading

The first step in designing tanks and vessels for seismic resistance is to analyse and evaluate the seismic loads acting on the structure. These systems can be analysed using either the Equivalent Lateral Force (ELF) method or dynamic analysis [39]. Dynamic analysis includes several

Table 9
Analysis methods accepted by each code.

Code/Book	Analysis	
NZSEE [30]	ELF, PD	ELF= Equivalent Static Analysis
EN 1998-4 [31]	ELF, PD	PD= Pseudo Dynamic Analysis (Modal Spectrum Dynamic Analysis)
ASCE 07–22 [29]	ELF, PD, D	D= Dynamic Analysis
AIJ [32]	ELF, PD	
ACI 350–3–20 [34]	ELF, PD	
IITK-GSDMA [33]	ELF, PD	
API 620 [35]	ELF, PD	
API 650 [36]	ELF, PD	
ASME VIII [37,38]	Referred to ASCE 07 [29]	

approaches, such as time-history analysis, response spectrum analysis, and modal response spectrum analysis [39,40].

Modal spectrum analysis provides an efficient and straightforward procedure for evaluating the linear seismic response of structures. Unlike the ELF method, which considers only the fundamental vibration mode, modal spectrum analysis accounts for the contribution of multiple modes. However, because it typically produces lower seismic demands—particularly overturning moments—design codes such as ASCE 07 [29] require scaling of the obtained forces. According to Chapter 12 of ASCE 07 [29], if the seismic force obtained from spectrum analysis is less than 100 % of that determined by the ELF procedure, it must be scaled to match the ELF value; the required scaling may vary depending on structural conditions [39].

Although dynamic analysis methods—including time-history, response spectrum, and modal spectrum procedures—can provide improved insight into modal behaviour and force distribution, their accuracy for liquid-containing structures is highly dependent on the adopted modelling assumptions. Sensitivity to the representation of the impulsive and convective components of the fluid, fluid–structure interaction, boundary conditions, and the selected design response spectrum can lead to significant variability in the predicted seismic response, making dynamic analysis less straightforward than in conventional building structures.

For this reason, many design standards and guidelines (e.g., API 650/620 and portions of ASCE 07 [29]) recognise the ELF procedure—or pseudo-dynamic coefficient-based methods—as the primary and more reliable approach for routine design. Dynamic analysis is generally reserved for special situations such as mass or stiffness irregularities, interaction with adjacent structures, or other complex configurations. ASCE 07 [29] also requires that, when response spectrum analysis is used, the resulting forces must not be lower than those obtained from the ELF procedure; scaling is required if this condition is not met, further emphasising the uncertainties associated with dynamic analysis results.

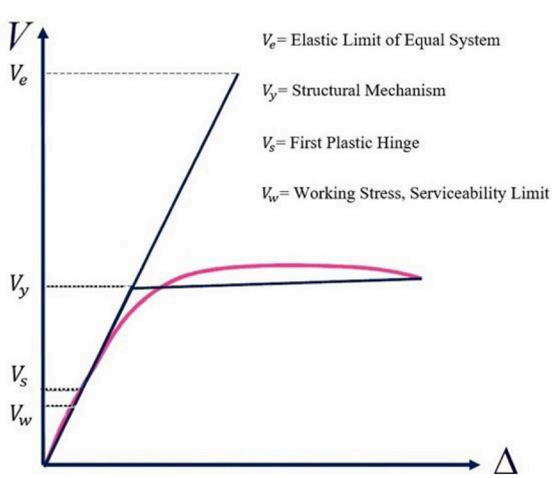
Therefore, although dynamic analysis can provide a more realistic distribution of seismic forces for certain cases, its strong dependence on modelling assumptions limits its applicability as a universal approach for all tank configurations. Consequently, most seismic design codes prioritise the ELF procedure, permitting or requiring dynamic analysis only when explicitly mandated or when justified by structural conditions.

Selecting the appropriate analysis procedure is essential for achieving an accurate seismic evaluation of tanks and vessels. While the ELF method is adequate for many cases, certain situations require dynamic analysis to obtain a more accurate representation of the seismic response and distribution of forces. Examples include specific vessel configurations listed in Table A1 (Appendix), for which dynamic analysis is mandated. Engineering judgment plays a key role in determining whether dynamic analysis is necessary—except where codes explicitly mandate it, such as in ASCE 07 [29], which requires dynamic analysis when mass or stiffness irregularities or interactions with other structures significantly affect the structural response. Table A1 (see Appendix) presents the cases where dynamic analysis is mandatory, while Table 9 summarises the accepted analysis methods permitted by different seismic codes.

The ELF method was originally developed for building structures and is largely empirical. Although petrochemical facilities (e.g., vessels) are often classified as regular structures, they are typically connected to piping systems with considerable mass. Such connections can induce interaction effects, making dynamic analysis more representative in many cases. Both ELF and dynamic analyses are generally performed using linear-elastic methods [39]. However, Chapter 15 of ASCE 07 [29] allows, for non-building structures, nonlinear response dynamic analysis in addition to ELF and linear dynamic analysis. In practice, however, nonlinear analysis is rarely applied to petrochemical tanks and vessels.

According to API 650 [36]/620 [35], equivalent static lateral forces are applied to a mathematical model of the tank, assuming a rigid wall and fixed-base structure. The pseudo-dynamic procedures in these codes rely on response spectrum analysis, considering two principal response modes of the tank and its contents: impulsive and convective. It is noted that dynamic analysis is neither mandatory nor explicitly included within the scope of these codes [35,36].

According to the AIJ [32], tank analysis for the impulsive mass can be performed using either the Modified Seismic Coefficient Method (ESF method) or Modal Spectrum Analysis. For foundation design, the



$$R_{LRFD} = \frac{V_e}{V_s} \qquad R_{ASD} = \frac{V_e}{V_w}$$

$$R_\mu = \frac{V_e}{V_y} \qquad \Omega_0 = \frac{V_y}{V_s}$$

$$Y = \frac{V_s}{V_w} \qquad \mu = \frac{\Delta_d}{\Delta_y}$$

$$R_{LRFD} = \frac{V_e}{V_s} = \frac{V_e}{V_y}\frac{V_y}{V_s} = R_\mu \Omega_0$$

$$R_{ASD} = \frac{V_e}{V_w} = \frac{V_e}{V_y}\frac{V_y}{V_s}\frac{V_s}{V_w} = R_\mu \Omega_0 Y$$

$$R_{LRFD} < R_{ASD} \qquad Y \cong 1.4$$

Fig. 4. Comparison of LRFD and ASD design approaches through strength ratios derived from the force displacement response.

Table 10
Design philosophy of each code.

Code/Book	Analysis
Seismic Design of Storage Tanks: 2009 (New Zealand Society for Earthquake Engineering)	LRFD
EN 1998-4 Design of Structures for Earthquake Resistance - Part 4: Silos, Tanks and Pipelines [31]	ULS
ASCE 07–22 [29]	ASD, LRFD
Design Recommendation for Storage Tanks and Their Supports with Emphasis on Seismic Design (2010 Edition)- (Architectural Institute of Japan)	ASD
IITK-GSDMA [33]	ASD
ACI 350–3–20 [34]	LRFD
ACI 350–3–01 [325]	ASD
API 620 (Design and Construction of Large, Welded, Low-pressure Storage Tanks) [35]	ASD
API 650 (Welded Tanks for Oil Storage) [35]	ASD
ASME VIII [37,38]	ASD

Modified Seismic Coefficient Method should be used. For liquid-containing tanks, the seismic loads associated with the convective mass must be obtained from the design velocity response spectrum, corresponding to the first natural period of the convective mass [324].

7. Design philosophy

In general, several design philosophies can be adopted in structural design. For tanks and vessels, the most common approaches are the Allowable Stress Design (ASD) and the Ultimate Limit State (ULS) or Load and Resistance Factor Design (LRFD). Fig. 4 illustrates the conceptual definitions of these two approaches and highlights the differences in design load levels and associated strength ratios.

In Fig. 4, V and Δ represent the base shear and the associated displacement values at different design levels (the subscripts e, y, s, and w correspond to elastic limit, structural mechanism, first plastic hinge, and working stress, respectively), while R represents the behaviour factor associated with the ASD and LRFD approaches. The figure also indicates the overstrength factor (Ω_o) and the ductility (μ). Before commencing the design process, it is essential to understand the magnitude of lateral seismic forces associated with each design philosophy. Table 10 presents a comparison of the fundamental design principles adopted by various seismic design codes.

The commentary in Chapter 1 of AIJ [32] clarifies that the adopted seismic design approach is the Allowable Stress Design (ASD) method, modified by the factor B, which represents the ratio of the structure's horizontal load-carrying capacity to its short-term allowable yield strength.

Similarly, Appendix E of API 650 [36] states that the appendix is based on the Allowable Stress Design (ASD) philosophy with its own specified load combinations. It also cautions against using load combinations from other design documents or codes, as such substitutions may require modification to ensure practical and realistic design outcomes. Additionally, the behaviour factor values obtained from the ASD method can be converted to those used in the LRFD method by dividing by 1.4, which enables a direct comparison of base shear coefficients and, consequently, the seismic base shear [34].

Chapter 5 of the New Zealand Society for Earthquake Engineering (NZSEE) [326] explains that the load combination in Equation 5.1 is derived from the Ultimate Limit State (ULS) seismic equations of the New Zealand Loadings Code (AS/NZS 1170) [326]. This equation applies a load factor of 1.0 to both Gravitational Load (G) and Live Load (Q) and incorporates vertical seismic acceleration directly into the earthquake load (E).

This chapter explicitly excludes a Serviceability Limit State (SLS)—or Working Stress Limit—earthquake load level, due to the limited ductility permitted and the satisfactory performance generally observed under frequent service-level earthquakes. However, it notes that special conditions may still require consideration of an SLS earthquake load combination.

8. Tank and vessel wall/foundation flexibility

In the case of tall vertical vessels (flexible vessels), structural flexibility must be considered using more rigorous approaches—such as dynamic analysis—to better capture the distribution of lateral seismic forces along the height of the structure. This is particularly important due to the influence of higher vibration modes and the dynamic interaction between the vessel and its supporting structure.

As discussed in previous sections, the thresholds distinguishing rigid and flexible vessels—and, more generally, non-building structures—are defined in design codes and literature (e.g., if the fundamental period T exceeds 0.06 s, the structure is considered flexible). As the tank aspect ratio (height/diameter) increases, the proportion of the tank contents exhibiting convective behaviour decreases, making the impulsive mode more dominant. Conversely, for tanks with a low aspect ratio, only about 30 % of the contents interact with the walls, while the remainder respond in various convective modes [30].

In fully filled and sealed tanks (e.g., wine tanks), the convective mode is absent, and all contents participate in the impulsive mode, which requires adjusting the impulsive mass height [30]. Because vessels are generally slender (with a small diameter-to-height ratio) and often filled and sealed, convective effects are typically negligible when they contain liquid. Consequently, their flexibility does not significantly

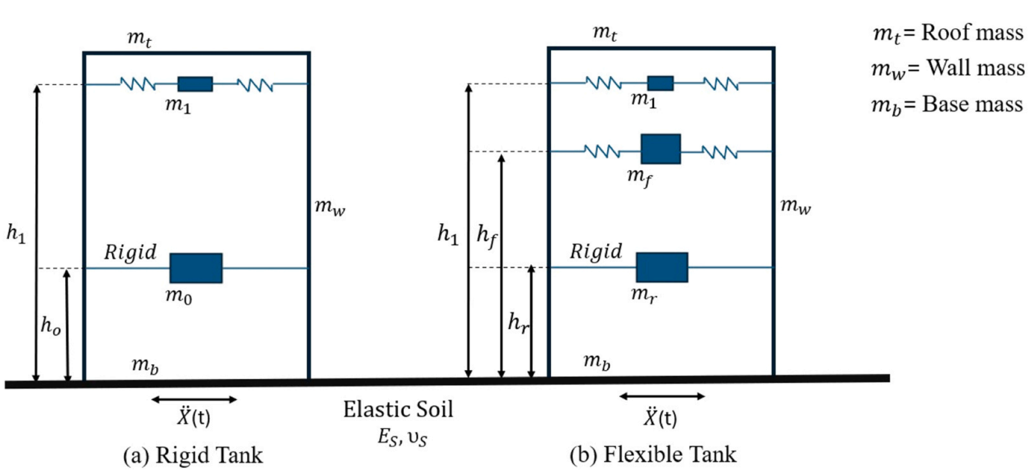


Fig. 5. Definition of wall flexibility by mass spring analogy from Priestley et. al. [109] study.

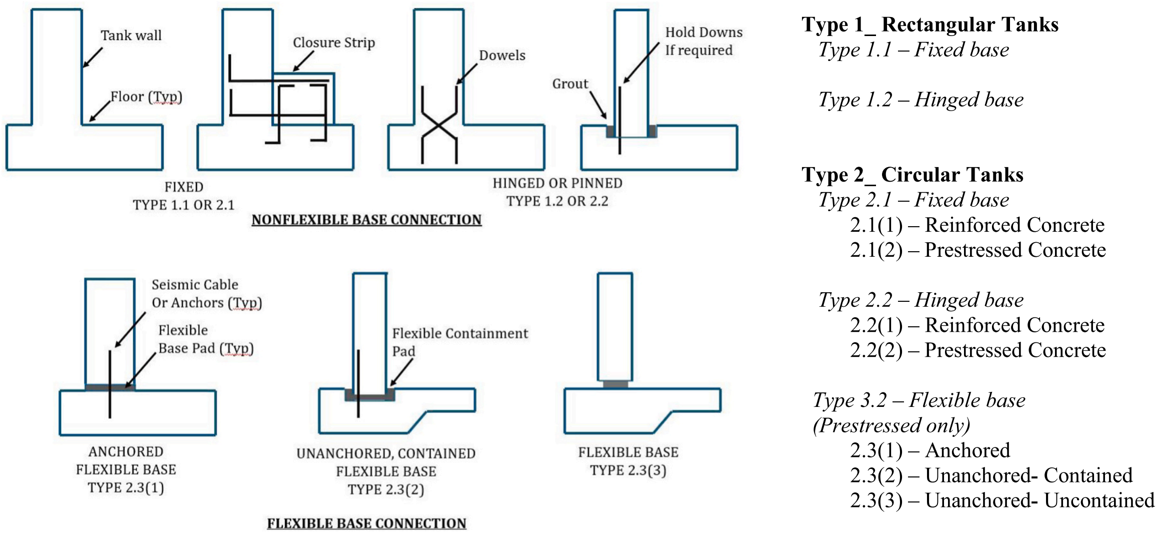


Fig. 6. Ground-supported Tanks Classification and Base Condition from ACI 350–3–20 (adapted from [34]).

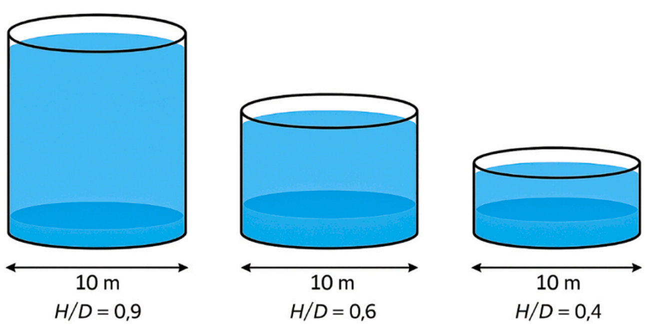


Fig. 7. Geometry and *H/D* Ratios of the Studied Cylindrical Steel Tanks.

Table 11
Anchorage and wall rigidity conditions accepted by each code.

Code/Book	Wall Condition	Foundation Condition	Ground-Supported Tanks Anchorage Condition
NZSEE [30]	Flexible Tanks, Rigid Tanks	Flexible Foundation,	Anchored, Mechanically Anchored,
EN 1998-4 [31]	Flexible Tanks, Rigid Tanks	Rigid Foundation	Unanchored, Self-Anchored,
ASCE 07–22 [29]	Rigid Tanks		Uplifting.
AIJ [32]	Rigid Tanks		
IITK-GSDMA [33]	Rigid Tanks	Rigid Foundation	
API 620 [35]	Flexible Tanks, Rigid Tanks		
API 650 [36]	Flexible Tanks, Rigid Tanks		

affect the dynamic response. Furthermore, the vessel shell is usually designed to withstand internal pressure, which increases its stiffness and reduces the impact of pressure redistribution under seismic excitation.

In contrast, the situation for tanks is more complex. When modelling a ground-supported tank, several flexibilities must be considered:

- Tank wall flexibility (Fig. 5),
- Tank bottom flexibility at the wall–base connection (Fig. 6), (Fig. 7)
- Foundation flexibility, and
- The ability of the tank base to uplift during earthquakes.

These flexibilities influence two key parameters: the distribution of hydrodynamic pressure on the tank wall and the natural vibration period (both impulsive and convective components), though their effects are not equal. Shell and foundation flexibility can lengthen the impulsive mode period, thus modifying the earthquake response according to its position on the response spectrum. For tall tanks on firm foundations, the influence of foundation flexibility is usually minor, though Soil–Structure Interaction (SSI) should still be evaluated [30].

The convective response in tanks is inherently flexible, with typical periods ranging from 2 to 10 s. For large tanks, NZSEE [30] notes that these periods are often between 6 and 10 s, meaning that additional flexibility from nonlinear effects has minimal influence on the period and damping of this response [36]. Analyses of flexible tanks show that wall flexibility has only a minor effect on the convective component of hydrodynamic pressure, allowing them to be treated similarly to rigid tanks in design [30].

For tanks with flexible walls, various codes and standards provide formulas for determining the hydrodynamic pressure distribution during earthquakes and the effect of wall flexibility on the structure's vibration period (see Tables A6 and A7 in the Appendix). In addition, the wall-to-base connection exhibits some rotational flexibility, which also influences both the pressure distribution and the natural vibration period. However, most reviewed codes address this effect only qualitatively. For example, IITK-GSDMA [33] states:

"For tanks with such flexible connections, the evaluation of the time period should appropriately consider the flexibility of the wall-to-base connection."

The commentary of this code further explains that wall flexibility affects the impulsive vibration period and refers to ACI 350.3–20 [34] for detailed guidance.

In unanchored ground tanks with a rigid base, partial detachment from the foundation can reduce hydrodynamic forces but increase vertical compressive stresses in the wall, potentially leading to buckling. In unanchored tanks with a flexible base, the rise in wall compressive stresses is smaller, but there is a risk of tank settlement in the supporting soil. It is therefore recommended that the tank ladder be disconnected from the ground; otherwise, the interaction effects with the tank must be included in the structural model. Table 11 summarises the codes that address these flexibility effects.

According to IITK-GSDMA [33], concrete tanks are categorised as rigid-walled, whereas steel tanks are classified as flexible. It further recommends applying rigid parameters to all tank types for simplicity [33]. While this approach simplifies the design process, it may not accurately capture the real-world dynamic performance. In contrast, NZSEE [30] acknowledges the inherent stiffness of concrete tank walls but cautions against assuming they are entirely rigid, suggesting that they exhibit some degree of flexibility. The NZSEE [30] also recommends adopting a flexible-tank design approach whenever there is uncertainty about rigidity, particularly for vertical cylindrical and rectangular tanks. This conservative approach emphasises the need to account for tank flexibility, especially when evaluating the interaction between soil behaviour and tank design under various seismic loading conditions [30].

Furthermore, after obtaining the equivalent base shear for each vibration mode, the resulting values must be combined to determine the total base shear. Different codes and standards adopt varying procedures for combining the horizontal impulsive, convective, and vertical impulsive components, as well as for defining and applying damping ratios. Table A2 (see Appendix) summarises the damping ratios and the combination methods used to evaluate base shear obtained from individual vibration modes.

Several modal combination methods are available, including the Direct Combination Method, the Square Root of the Sum of Squares (SRSS) Method, and the Complete Quadratic Combination (CQC) Method.

According to NZSEE [30], the total tank base shear resulting from the combined effects of impulsive and convective modes is determined by combining the peak spectral responses of each action using the SRSS method. However, for flexible tanks, NZSEE [30] adopts a more conservative approach, combining the horizontal rigid-impulsive and flexible-impulsive components using the absolute sum method instead of the SRSS method. This ensures that both the rigid and flexible components are fully included in the total horizontal impulsive response.

When it comes to damping ratios, the AIJ [32] provides a somewhat different interpretation compared with other codes. Experimental data from full-scale and model tanks indicate that higher vibration modes may exhibit damping ratios of 5 % or greater. Conversely, other studies suggest that using the same damping ratio as the first mode can produce a more realistic simulation of actual earthquake responses in tanks. This highlights that damping characteristics can vary significantly depending on structural configuration, tank geometry, and mode shape, leading to different recommendations for specific situations.

In contrast, the NZSEE [30] adopts a more prescriptive approach when defining damping ratios for different modes. It specifies an overall damping level of 0.5 % for the convective mode in all tank types. This relatively low damping value reflects the inherent behaviour of the convective mode, which typically dissipates less energy than the impulsive (structural) response. The damping ratio for the impulsive mode is not a fixed value but is instead evaluated based on several factors, including the tank material, foundation radiation damping, and soil hysteretic damping. This methodology ensures that the overall damping ratio realistically captures the complex interaction among the tank structure, foundation, and surrounding soil, which is critical for an accurate representation of the tank's seismic response.

A comparison of these two approaches reveals that while NZSEE [30] emphasises the mathematical combination of spectral responses using the SRSS method, it applies a more conservative combination rule—the absolute sum—for flexible tanks when calculating impulsive actions. In contrast, AIJ [32] acknowledges the variability in damping ratios for higher modes and suggests that, in certain cases, matching the damping ratio to that of the first mode may yield the most accurate simulation of real-world tank behaviour during earthquakes.

Overall, the NZSEE [30] code stands out for its explicit and detailed treatment of damping, as it accounts for material properties, foundation behaviour, and soil characteristics to define a specific damping value for the convective mode and a comprehensive evaluation framework for determining the damping ratio of the impulsive mode. In Section 14 of this paper, the influence of the damping ratio on the seismic response is presented and discussed.

9. Base shear

There are several approaches to applying earthquake excitation in structural engineering. Among these, the Equivalent Lateral Force (ELF) method is one of the most widely used in the seismic design of tanks and petrochemical equipment, including pressure vessels. In this method, an equivalent seismic lateral force, referred to as the base shear, is calculated and applied to the structure. The Base Shear depends on the mass of the structure, the seismic hazard level of the site, and therefore on ground acceleration and soil conditions, as well as on the dynamic characteristics of the structure—such as its natural period of vibration, damping ratio, ductility, and importance factor.

As discussed earlier, this paper focuses on tanks and vessels, which may be constructed under various conditions. A tank may be a large, ground-supported structure or a smaller unit mounted on a frame or building roof. Similarly, a vessel may be supported directly by a structure or installed on a building. Differentiating among these configurations is essential, since the base shear equations provided by design codes vary depending on the support condition.

A key distinction must also be made between non-structural components and nonbuilding structures, as previously discussed. Because non-structural components are typically attached to or supported by other structures, their seismic response can be amplified by dynamic interaction between the supporting and supported systems, as well as by the elevation at which the component is installed. Tables A3 and A4 (see Appendix) present the base shear equations for ground-supported tanks and other nonbuilding structures under Chapter 15 of ASCE 07 [29].

ASCE 07 [29] calculates the seismic force on non-structural components using a simplified approach that adapts the base shear concept—originally developed for entire structures—to individual components. This approach recognises that non-structural components experience amplified accelerations at higher building elevations and therefore introduces two correction factors:

- The component amplification factor (a_P), and
- The Ccomponent response modification factor (R_P).

The amplification factor (a_P) represents the tendency of a component to amplify ground motion; components that are more flexible or resonate with the building's vibration modes tend to have higher a_P values. The response modification factor (R_P) is analogous to the R factor used for building structures. It accounts for a component's ductility and overstrength, thereby reducing design forces for more ductile components.

Additionally, the height ratio (z/h) adjusts the design force based on the component's location within the building—components located higher experience greater seismic accelerations because upper stories sway more. The spectral response acceleration (S_{Ds}) represents the seismic demand corresponding to a specific site location, incorporating local soil properties and ground motion characteristics. The value of S_{Ds} is typically obtained from seismic hazard maps and serves as the baseline parameter for computing seismic design forces.

The ASCE 07 [29] equation for component seismic force is conservative, as it includes key variables influencing a component's seismic response without requiring detailed dynamic analysis. The intent is to provide engineers with a clear and adaptable framework for designing non-structural components to resist seismic loads. For essential facilities—such as hospitals and data centres—ASCE 07 [29] mandates a more rigorous design approach, which may include higher design forces or enhanced detailing requirements to ensure the continuity of operations after an earthquake.

Table A5 (see Appendix) summarises the procedure from ASCE 07 [29] Chapter 13 for calculating the base shear of vessels and tanks that fall under the non-structural component category.

10. Period of vibration for impulsive and convective modes

When applying the Equivalent Static Force (ESF) method, accurately estimating the period and mass of each vibration mode, along with identifying the locations of lateral force effects from these modes, is essential for computing the base shear and anchorage forces contributed by each mode. This section reviews and compares these parameters as defined in various design codes for tanks and vessels, highlighting differences in regulatory approaches to base shear estimation.

The tables presented in previous sections focused on evaluating the period of vibration and base shear for ground-supported tanks under different codes and standards. The current section extends this discussion to other types of liquid and gas containers and petrochemical structures, such as vessels, elevated equipment, and process columns. Table A8 (see Appendix) lists the formulae used to determine the period of vibration under various conditions. These expressions are generic and applicable across multiple code provisions.

Among the reviewed codes, nearly all include formulations for elevated tanks, with AIJ [32] specifically addressing spherical tanks supported on legs, while ASCE 07 [29] covers all structural configurations listed in Table A8. As previously mentioned, ASCE 07 [29] includes two relevant chapters for calculating the seismic base shear of these systems:

- Chapter 13, addressing non-structural components, and
- Chapter 15, addressing nonbuilding structures.

The corresponding relationships for base shear evaluation in these chapters are presented in Tables A3 and A4 (see Appendix). As discussed in Section 4, when designing an industrial component that falls within the overlap of these two chapters, it is critical to determine whether the element qualifies as a non-structural component or a nonbuilding structure to apply the correct procedure for seismic load evaluation. Furthermore, ASCE 07 [29] specifies behaviour factors (R values) for such structures, as shown in Tables A10 and A11 (see Appendix).

In seismic analysis, soil foundation deformation plays a major role, particularly when calculating the impulsive mode periods in both horizontal and vertical directions, as emphasised by NZSEE [30]. For flexible tanks, soil deformation must be included in estimating the periods associated with both the rigid impulsive mass (m_r) and the flexible impulsive mass (m_f). This underscores the importance of Soil–Structure Interaction (SSI), as foundation deformation can significantly influence the dynamic response, especially in the impulsive modes.

Additionally, NZSEE [30] requires that the mass of the tank shell, base, and any supporting foundation be included as part of the impulsive mass when computing impulsive mode periods. This results in a more accurate representation of the overall structural response by capturing the contribution of the entire system to dynamic behaviour. In contrast, for convective modes (i.e., liquid sloshing inside the tank), the effects of soil deformation and wall flexibility can be neglected, as they have minimal influence on the convective vibration periods compared to the impulsive ones.

While NZSEE [30] emphasises the inclusion of foundation mass and soil deformation for impulsive modes, it simplifies the analysis for convective modes by allowing the omission of soil effects. The same code also provides guidance for tanks with nonstandard geometries, recommending approximate analytical procedures based on established results for typical tank shapes and highlighting the need to estimate sloshing frequencies accurately in such cases.

In the AIJ [32], special attention is given to the uplift behaviour of unanchored tanks, accounting for the flexibility of both the bottom plate and shell wall. AIJ [32] distinguishes between two structural configurations:

- Flexible bottom plate with rigid shell, and
- Flexible bottom plate and flexible shell.

This distinction is critical for analysing the dynamic behaviour of unanchored tanks. In the first configuration—a flexible bottom with a rigid shell—the tank is assumed to rock during seismic excitation, but the shell remains undeformed. This simplification neglects energy dissipation and wall deformation, and therefore AIJ [32] does not provide an explicit expression for the rocking period (T_f). Instead, the analysis is typically performed using simplified mechanical analogies, such as hinged-base models or nonlinear time-history analyses. While this approach is conservative, it may reduce the accuracy of dynamic characterisation, particularly for slender or thin-walled tanks.

In the second configuration, both the bottom plate and the shell wall are considered flexible, providing a more realistic representation of actual seismic behaviour. In this case, the rocking period (T_f) is explicitly calculated, accounting for both uplift and shell deformability. The effective period of vibration (T_e) is then evaluated as a combination of the impulsive period (T_1) and the rocking period (T_f), expressed as:

$$T_e = \sqrt{T_1^2 + T_f^2} \tag{1}$$

Here, T_e represents the natural period associated with the impulsive mass (the portion of liquid moving with the tank wall), while T_f captures the contribution of base rocking and shell flexibility. The parameter λ, used in the formulation of T_f, is defined as a function of the tank aspect ratio (H_L/D), where H_L is the liquid height and D is the tank diameter.

11. Response modification and behaviour factors

In seismic design codes and standards, the design seismic forces are reduced using a parameter known as the behaviour factor, a fundamental concept in seismic design literature. This factor represents a structure's capacity to absorb and dissipate energy through inelastic deformation, reflecting its ductility, overstrength, and redundancy, which together define its seismic resilience.

Different codes address this reduction using two main approaches. Some specify explicit reduction factors according to the structural system type, while others apply the same philosophy implicitly by providing relationships that depend on structural properties such as ductility, damping ratio, and vibration period.

In ASCE 07 [29] and API [35] [36], this reduction is incorporated through the response modification factor (R). In contrast, Eurocode 8 (EN 1998–4) [31] and IITK-GSDMA [33] use the term behaviour factor (q and R, respectively). NZSEE [30] standard implicitly introduces a correction factor (C_f), defined as a function of the ductility factor (μ) and the damping ratio. Similarly, AIJ [32] defines a structural characteristic coefficient (D_s), which depends on the damping ratio and natural period, and accounts for the deformation of the tank shell and base plate.

One key distinction among these codes is the level of classification detail they provide. For example, ASCE 07 [29] and IITK-GSDMA [33] include detailed classifications of tanks by material (concrete, steel, masonry), anchorage condition (anchored or unanchored), and installation type (ground-supported, elevated, or underground), each with corresponding reduction factors. In contrast, NZSEE [30] provides a less detailed classification (e.g., no specific categories for concrete tanks), and EN 1998–4 [31] adopts an even more generalised classification.

Since tanks exhibit two main vibration modes—impulsive and convective—all these codes define reduction factors for both. However, a significant difference lies in how they handle the convective (sloshing) mode. The convective period, generally between 2 and 10 s, falls in the displacement-sensitive range, making it primarily governed by ground displacement rather than acceleration. Consequently, damping and ductility have only a minor effect on the reduction factor for this mode.

Therefore, EN 1998–4 [31] and IITK-GSDMA [33] adopt a unit value (1.0) for the convective sloshing mode, implying no reduction from energy dissipation mechanisms. Conversely, ASCE 07 [29] and API [35]

Table 12
Geometric and material properties of the studied tanks.

Tank ID	Location	Geometry	Diameter (m)	Height (m)	H/D	Material	Fluid Density (kg/m³)	Anchorage Condition*
T1	Above Ground	Cylindrical	10	9	0.90	Steel E = 210 GPa; fy= 275 MPa; ρ= 7860 kg/m³	1000	Code-dependent
T2				6	0.60			
T3				4	0.40			

[36] recommend values of 1.5 and 2, respectively. NZSEE [30] proposes variable values depending on the height-to-radius ratio (H/R), damping ratio, and soil conditions.

Another important distinction lies in the design philosophy adopted by each standard. Some use the Allowable Stress Design (ASD) approach, while others are based on the Load and Resistance Factor Design (LRFD) or Ultimate Limit State (ULS) method. The R values derived under the ASD level (e.g., API 650 [36]) are typically about 1.4 times higher than those for LRFD/strength-level design (e.g., ASCE 07 [29]).

In IITK-GSDMA [33], elevated tanks supported by steel frames are assigned a higher Behaviour Factor ($R = 2.5$), whereas tanks supported by masonry shafts have lower values ($R = 1.3$–1.5). This distinction reflects the greater ductility and energy-dissipation capacity of steel frame systems. Similarly, ASCE 07 [29] classifies Behaviour Factors according to structural type:

- Elevated tanks on Ductile Moment-Resisting Frames (SMRF): $R = 2.5$,
- Ordinary Moment-Resisting Frames (OMRF) with lower ductility: $R = 1.8$.

These values align closely with IITK-GSDMA [33], while NZSEE [30] adopts a ductility-based framework, assigning higher factors to tanks designed for inelastic deformation. Together, these similarities highlight a generally consistent design philosophy across major codes, with variations reflecting regional seismic performance objectives.

In NZSEE [30], the Behaviour Factor (μ) is defined according to the material and ductility capacity of the tank structure. For example:

- Steel tanks with primarily elastic response, $\mu = 1.25$,
- Unanchored tanks exhibiting limited ductile behaviour (e.g., elephant-foot shell buckling), $\mu = 2.0$.

The higher μ acknowledges the capacity for inelastic deformation without structural failure. NZSEE [30] also allows for increased μ values when specific ductile design criteria are met.

AIJ [32] defines the structural characteristic coefficient (D_s) for above-ground, vertical, cylindrical tanks under three base conditions:

1. Unanchored tanks,
2. Anchored tanks, and
3. Buckling of wall plates and supports in spherical tanks.

The coefficient D_s is derived as the product of two parameters:

$$D_s = D_h \cdot D_\eta \tag{2}$$

where D_h is determined by the structural damping, and D_η by the ductility. Both parameters are functions of:

- The damping ratio (h),
- The natural period (T_1)—representing deformation of the bottom plate and anchorage system,
- The modified period (T_e)—accounting for deformation of both the shell wall and bottom plate.

Table A9–A13 (see Appendix) summarise the response modification factors and behaviour factors for tanks and vessels across the reviewed codes and standards.

12. Importance factors

The importance factor (I) represents the consequence of failure for a tank or vessel and is assigned based on several parameters, including the type of stored contents, population risk, environmental exposure, value of adjacent properties, and national or community significance.

API 650 [36]

The API 650 standard defines three Seismic Use Groups (SUG), each corresponding to a different level of importance and post-earthquake performance requirement:

- SUG III ($I = 1.50$): Tanks that provide essential services and must remain fully operational after an earthquake. These include tanks critical for public safety, health, or life support, as well as those containing hazardous or significant substances.
- SUG II ($I = 1.25$): Tanks serving major facilities that store materials posing a substantial public hazard in the absence of secondary containment or safety measures.
- SUG I ($I = 1.00$): All other tanks not assigned to SUG II or III.

AIJ [32]

AIJ classifies tanks into four main categories according to storage capacity, stored material type, and post-earthquake functionality. The corresponding importance factors range from 0.6 to 1.2, with higher values representing tanks of greater social or safety significance.

IITK-GSDMA [33]

IITK-GSDMA standard divides liquid-containing tanks into three classes:

- Class 1 ($I = 1.75$): Tanks containing hazardous materials, where failure could cause serious risk to life or the environment.
- Class 2 ($I = 1.50$): Tanks storing essential or moderate-risk liquids, such as drinking water, low-flammability petrochemicals, or those required for emergency operations.
- Class 3 ($I = 1.00$): Tanks with low risk to human life or the environment. The code also refers to IS 1893 (Part 4) [327] for further classification guidance when applicable.

NZSEE [30]

NZSEE standard uses a more comprehensive risk-based classification, evaluating tanks against four criteria:

1. Life safety,
2. Environmental exposure,
3. Community significance, and
4. Adjacent property value.

Each criterion includes five consequence levels—*negligible, slight, moderate, serious,* and *extreme*. After individually assessing all four factors, the most severe consequence is selected to determine the overall importance level, which can range from 1 to 4.

Each importance level corresponds to a return period factor (R_u), defined as follows:

Table 13
Seismic action components and hydrodynamic response of the studied tanks (EN 1998-4 [31]).

Type	Anchorage Condition	Wall Rigidity	Rigid Impulsive (S)	Flexible Impulsive (S)	1st Convective (S)	Total Contained Fluid Mass (ton)	Rigid Impulsive Mass (ton)	Flexible Impulsive Mass (ton)	All Convective Mass (ton)	1st Convective Mass (ton)	Rigid Impulsive Base Shear (kN)	Flexible Impulsive Base Shear (kN)	All Convective Base Shear (kN)	1st Convective Base Shear (kN)	Anchorage Force (kN)
T1	M.A.	R	0.09	0	3.3	706.8	520.9	0	185.7	178.0	1744.8	0	168.9	148.2	865.4
		F		0.08			104.3	416.6			406.5	1306.7			918.0
	M.A.D.B.	R		0			520.9	0			1327.8	0			672.0
		F		0.08			104.3	416.6			309.3	1012.2			721.2
T2	M.A.	R	0.06	0	3.3	471.2	289.1	0	181.9	174.2	882.3	0	162.7	142.0	359.6
		F		0.05			53.0	236.0			197.1	662.8			356.8
	M.A.D.B.	R		0			289.1	0			724.5	0			300.6
		F		0.05			53.0	236.0			161.8	558.2			303.7
T3	M.A.	R	0.04	0	3.3	314.1	145.6	0	168.3	160.6	423.6	0	141.4	120.7	161.4
		F		0.03			25.8	119.7			97.3	314.4			152.3
	M.A.D.B.	R		0			145.6	0			365.2	0			141.4
		F		0.03			25.8	119.7			83.9	278.9			136.2

M.A. = Mechanically Anchored
M.A.D.B. = Mechanically Anchored with Ductile Bolt
R= Rigid
F= Flexible

- Level 1, $R_u = 0.5$
- Level 2, $R_u = 1.0$
- Level 3, $R_u = 1.3$
- Level 4, $R_u = 1.8$

For example, a tank containing highly hazardous substances located within 50 m of an area with over 100 occupants would be classified as "Extreme" (Schedule 1, UNPG I) and assigned the highest importance factor.

EN 1998–4 [31]

Eurocode 8, Part 4, classifies tanks based on the type of stored liquid and the associated life-safety, environmental, economic, and social risks. It defines three protection levels, each subdivided into three reliability classes, resulting in nine possible importance factor values ranging from 0.8 to 1.6.

ASCE 07 [29]

Since ASCE 07 [29] addresses tanks and vessels under two separate chapters—Chapter 13 (Non-structural Components) and Chapter 15 (Nonbuilding Structures)—it is critical to specify which provisions apply to a given system. In both chapters, the importance factor is determined based on the Risk Category of the tank or vessel.

Table A14 (see Appendix) summarises the prescribed importance factors for tanks and vessels covered under these two chapters.

13. Freeboard

Freeboard—defined as the vertical distance between the maximum liquid level and the top of a storage tank—is a critical parameter for minimising operational damage caused by sloshing waves. These waves arise when seismic excitation induces liquid motion, generating dynamic pressure on the upper shell and roof. The freeboard serves as a protective buffer, preventing both structural damage and product loss.

Several industry standards, including API 650 [36], NZSEE [30], and ASCE 07 [29], adopt similar design philosophies for freeboard calculation but differ in their treatment of sloshing wave height and its relationship to the required freeboard.

A key step in freeboard design is determining the sloshing wave height (δ_S), which governs the minimum required freeboard to contain convective motion. Sloshing waves result from the dynamic response of the liquid during seismic excitation. The NZSEE [30] recommends estimating δ_S using spectral modal analysis, considering at least the first two antisymmetric modes of liquid motion. This approach captures the most critical wave effects without necessitating excessive roof reinforcement. The wave height is commonly calculated using the equation:

$$\delta_S = 0.42 \quad D A_f \tag{3}$$

where D is the tank diameter and A_f is the seismic response factor.

Recent probabilistic studies have also contributed to improving freeboard assessment. For example, Merino et al. [328] performed a comprehensive probabilistic study on sloshing-induced freeboard demands in anchored cylindrical tanks. Using real-world geometric distributions, they generated a synthetic population of tanks subjected to recorded ground motions at different hazard levels. A mechanics-based surrogate model, calibrated against detailed finite-element simulations, was used to efficiently predict sloshing response. Their results showed substantial variability in sloshing wave height—especially in large-diameter ($R > 10$ m) and squat ($H/R < 2$) tanks—and indicated that existing API 650 [36] freeboard provisions are generally conservative. Based on these findings, the authors proposed risk-consistent safety factors to calibrate freeboard height according to acceptable risk levels.

The API 650 [36] standard also incorporates sloshing wave height into its freeboard provisions, stressing the need to accommodate convective liquid motion.

Table 14
Damping ratios, response coefficients, and minimum thickness requirements for the studied tanks (EN 1998-4 [31]).

Type	Anchorage Condition	Wall Rigidity	Imp. Damping Ratio %	Con. Damping Ratio %	qi	qc	Max Convective Wave (m)	Min Plate Thickness (mm)
T1	M.A.	R	2	0.5	1.5	1	0.36	6
		F						8
	M.A.D.B.	R			2.5	1		6
		F						7
T2	M.A.	R			1.5	1	0.35	5
		F						6
	M.A.D.B.	R			2.5	1		5
		F						5
T3	M.A.	R			1.5	1	0.32	4
		F						4
	M.A.D.B.	R			2.5	1		4
		F						4

M.A. = Mechanically Anchored
M.A.D.B.= Mechanically Anchored with Ductile Bolt
R= Rigid
F= Flexible

- For fixed-roof tanks, the roof must either be designed to resist the hydrodynamic pressures produced by sloshing or provide sufficient freeboard to prevent impact.
- This is particularly critical for hazardous liquid storage, where roof failure could cause severe environmental and safety hazards.

A primary difference between API 650 [36] and NZSEE [30] lies in how they distribute impulsive and convective forces. API 650 [36] permits treating the entire liquid content as impulsive mass when the tank is filled to capacity—requiring design modifications to accommodate this assumption. Conversely, NZSEE [30] recommends explicit spectral modal analysis for wave height estimation, without reducing convective mass based on ductility.

The API 650 [36] freeboard requirements also vary by Seismic Use Group (SUG):

- SUG I: Freeboard is optional.
- SUG II: Freeboard becomes mandatory when the spectral response acceleration ($S_{DS} \geq 0.33$ g), with a minimum height of 0.7 δ_S if secondary containment or roof strength is provided.
- SUG III: Requires a full freeboard height equal to δ_S to fully contain sloshing waves.

The ASCE 07 [29] standard follows similar design principles, explicitly addressing seismic zone requirements.

- For Risk Category IV (critical facilities), the full sloshing wave height must be considered in the design.
- For Risk Categories I, II, and III, simplified provisions may be applied, allowing use of a fixed natural period ($T_L = 4$ s) to reduce the freeboard requirement.

Both ASCE 07 [29] and API 650 [36] specify that if the provided freeboard is less than the calculated δ_S, the roof and its supports must be designed to withstand the full hydrodynamic pressures generated by sloshing.

Despite minor methodological differences, API 650 [36], NZSEE [30], and ASCE 07 [29] share the same design intent: to mitigate seismic sloshing-induced forces and protect tank integrity.

- API 650 [36] allows optional freeboard for low-risk tanks (SUG I).
- NZSEE [30] and ASCE 07 [29] impose stricter freeboard requirements for tanks in high-seismic zones or critical service categories.

All three standards emphasise safety, environmental protection, and functional continuity, particularly for tanks storing hazardous materials.

14. Comparative case studies

In this numerical benchmark, four standards (EN 1998–4 [31], NZSEE [30], API 650 [36], and ASCE 07 [29]) are implemented because these documents provide complete, explicit, and mutually comparable analytical procedures for ground-supported steel tanks. Other documents reviewed in earlier sections (AIJ [32] and IITK-GSDMA [33]) are included only in the qualitative comparison to maintain a focused and methodologically consistent numerical evaluation.

For the comparative code-based evaluation, three ground-supported cylindrical steel tanks were defined with a constant diameter of 10 m. The tank heights were selected as 9 m, 6 m, and 4 m, which correspond to height-to-diameter ratios of $H/D = 0.9$, 0.6, and 0.4, respectively. The purpose of choosing these three values was to cover a sufficiently broad range of geometric slenderness, allowing the seismic behaviour to be examined from relatively tall configurations to squat ones.

In all analyses, the tank wall was modelled using structural steel with a uniform shell thickness of 10 mm, consistent with the values used throughout Section 14. Water, with a density of 1000 kg/m^3 , was adopted as the stored liquid, allowing direct use of the impulsive and convective mass expressions from Tables A6 and A7 without material-dependent correction factors.

A single seismic input—the EN 1998–4 [31] Type-1 elastic spectrum for soil class C with ag = 0.20 g—was applied to all three benchmark tanks and across all standards. While the same spectrum was used, each code employed its own damping ratios, behaviour (response-modification) factors, impulsive/convective mass partitions, and base-shear formulations (Tables A2–A11), ensuring that the comparison reflects only procedural differences between standards (Table 12).

The three benchmark tanks share identical diameters ($D = 10$ m) and heights of 9 m, 6 m, and 4 m (T1–T3). Consequently, variations in the results presented in Tables 13–18 arise solely from the differing analytical assumptions embedded within each standard.

In addition, the minimum required shell thickness was independently calculated using EN 1993–1–6 and NZSEE [30] for comparison of their design philosophies.

The results presented in Tables 13 and 14 illustrate, in a unified manner, how the hydrodynamic behaviour of cylindrical steel tanks is shaped by geometry, anchorage condition, and wall rigidity under EN 1998–4 [31]. Across all three tanks, the distribution of impulsive and convective masses follows consistent physical trends. Rigid-wall tanks mobilise a larger impulsive mass, producing higher impulsive base shear and greater anchorage demand, whereas flexible-wall configurations reduce impulsive participation and shift a larger portion of the

Table 15
Seismic action components and hydrodynamic response of the studied tanks (NZSEE [30]).

Type	Anchorage Condition	Wall Rigidity	Rigid Impulsive (S)	Flexible Impulsive (S)	1st Convective (S)	Total Contained Fluid Mass (ton)	Rigid Impulsive Mass (ton)	Flexible Impulsive Mass (ton)	All Convective Mass (ton)	1st Convective Mass (ton)	Rigid Impulsive Base Shear (kN)	Flexible Impulsive Base Shear (kN)	All Convective Base Shear (kN)	1st Convective Base Shear (kN)	Anchorage Force (kN)
T1	M.A.E.	Rigid	0.09	0	3.31	706.8	520.9	0	185.7	178.0	2428	0	183.6	161.1	1188.4
	M.A.D.H										3668		392.9	344.8	1907.3
T2	M.A.E.		0.06		3.35	471.2	289.1		181.9	174.2	1141		176.8	154.4	459.8
	M.A.D.H										1610		378.4	330.3	726.8
T3	M.A.E.		0.04		3.48	314.1	145.6		168.3	160.6	519		153.7	131.2	196.7
	M.A.D.H										693		328.9	280.8	318.7

M.A.E. = Anchored (Elastically Supported)
M.A.D.H. = Anchored (Anchored with Ductile Tension Yielding Hold-Down Bolts)

hydrodynamic response into convective modes. This redistribution becomes more prominent as wall flexibility increases, revealing the sensitivity of slender tanks to shell deformability and demonstrating that rigid-wall assumptions may underestimate the convective contribution for tanks with lower stiffness.

The influence of tank geometry is clearly captured by the variation in H/D among T1, T2 and T3. Although the tallest tank (T1, $H/D = 0.9$) produces the largest absolute sloshing amplitudes because of its greater water depth, the relative proportion of convective mass increases steadily as H/D decreases. Consequently, the squat tank T3 ($H/D = 0.4$) develops the highest convective-to-total-mass ratio, consistent with the well-established behaviour of broad tanks where sloshing becomes a dominant response mechanism. T2 displays an intermediate distribution between impulsive and convective action, as expected for a medium-slender configuration.

Anchorage conditions further modify the balance between impulsive forces and transmitted base actions. Mechanically anchored tanks concentrate a larger portion of the seismic demand into impulsive base shear because uplift is restrained, while the introduction of ductile bolts introduces limited rotational flexibility, slightly reducing force transfer without altering the underlying response hierarchy. The tall tank exhibits the largest anchorage forces under rigid anchorage, reflecting the increased overturning sensitivity associated with deeper fluid columns, whereas squat tanks show smaller absolute demands but retain a high impulsive share of the total shear.

The base shear results reinforce these overall trends. For squat configurations such as T3, the impulsive component—though smaller in magnitude than in tall tanks—remains the dominant contributor to total shear, whereas in tall and more flexible tanks, the combined effect of flexible impulsive and convective components becomes increasingly significant. This highlights the importance of incorporating wall flexibility in the seismic assessment of slender tanks, particularly because EN 1998–4 [31] allows non-conservative underestimation of convective action when rigid-wall assumptions are used.

Table 14 complements the force-based results by relating damping ratios, response coefficients and minimum required shell thicknesses. The adopted damping ratios (2 % for impulsive and 0.5 % for convective components) result in distinct response coefficients: impulsive actions exhibit increased effective ductility when ductile bolts are used, while convective actions remain essentially elastic with a fixed coefficient of unity. Maximum sloshing heights follow the geometric trends of the tanks, with T1 and T2 generating larger wave amplitudes than T3 due to their increased liquid depth, which directly affects freeboard requirements and potential roof–liquid interaction.

The minimum shell thicknesses reflect the distribution of hydrodynamic demand. Required thickness increases with impulsive base shear, leading to the highest values for the tall anchored tank (up to 8 mm) and the lowest for the squat tank (as low as 4 mm). The modest reduction in required thickness for ductile-bolt anchorage corresponds to the reduced transmitted forces. Under EN 1993–1–6, the final thickness is governed by buckling checks—meridional, circumferential, and shear buckling—based on membrane stresses, elastic resistances, slenderness parameters, and interaction criteria, ensuring that none of the relevant failure modes becomes critical.

The results summarised in Tables 15 and 16 demonstrate how the NZSEE [30] provisions shape the hydrodynamic response, mass participation, base-shear distribution, damping characteristics, and required plate thicknesses of the studied tanks. In comparison with EN 1998–4, these results underline the influence of different national design philosophies, particularly in terms of anchorage forces, impulsive response, and wall-thickness demand.

Across all three tanks, the NZSEE [30] procedure consistently predicts higher seismic actions, most notably for anchored systems incorporating uplift-restraining devices. Tanks with elastically anchored supports (M.A.E.) mobilise substantial impulsive mass—especially in T1 and T2—but the introduction of ductile, tension-yielding hold-down

Table 16
Damping Ratios, Response Coefficients, and Minimum Thickness Requirements for the Studied Tanks (NZSEE [30]).

Type	Anchorage Condition	Wall Rigidity	Imp. Damping Ratio %	Con. Damping Ratio %	qi	qc	Max Convective Wave (m)	Min Plate Thickness (mm)
T1	M.A.E.	Rigid	14	0.5	0.55	0.92	0.39	7
	M.A.D.H.				0.32	0.43	0.83	9
T2	M.A.E.				0.55	0.92	0.38	6
	M.A.D.H.				0.32	0.43	0.81	7
T3	M.A.E.				0.55	0.92	0.35	4
	M.A.D.H.				0.32	0.43	0.75	5

M.A.E.= Anchored (Elastically Supported)
M.A.D.H.= Anchored (Anchored with Ductile Tension Yielding Hold-Down Bolts)

bolts (M.A.D.H.) further increases both impulsive base shear and anchorage forces. This is because NZSEE [30] explicitly accounts for the uplift-resisting capacity and energy-dissipating mechanism of the hold-down system, thereby activating a larger portion of the impulsive mass. For instance, in T1, the impulsive base shear rises from 2428 kN (M.A.E.) to 3668 kN (M.A.D.H.), while the anchorage force increases from 1188 kN to 1907 kN. Similar increases in T2 and T3 confirm the strong sensitivity of the NZSEE [30] methodology to anchorage stiffness.

Geometric effects remain evident in the convective components. The tallest tank, T1, produces the largest absolute convective mass and base shear due to its deeper fluid column, while the relative significance of convective behaviour increases progressively toward T3 as H/D decreases. Nevertheless, impulsive action remains the dominant contributor under NZSEE [30], contrasting with the more balanced distribution predicted by EN 1998–4 [31]. Even for the squat tank T3, convective base shear increases when hold-down bolts are introduced, indicating that uplift-restraining mechanisms can also influence sloshing behaviour by reducing effective wall flexibility.

Table 16 further highlights the differences between the two codes through the adopted damping ratios and response coefficients. NZSEE [30] uses a markedly higher impulsive damping ratio of 14 %—compared with 2 % in EN 1998–4 [31]—together with relatively small performance factors that counterbalance the reduction that would otherwise result from higher damping. Convective damping remains at 0.5 %, making sloshing modes sensitive to tank geometry. The resulting convective wave heights, such as values between 0.55 m and 0.32 m for T1, are generally greater than those predicted by the European code.

These seismic trends directly govern the minimum plate thickness requirements. Under NZSEE [30], thickness design is based on limiting compressive circumferential stress; the combined hydrostatic and hydrodynamic hoop stresses must remain below an allowable limit determined by yield strength, geometry, and seismic demand. Consequently, tanks designed according to NZSEE [30] require thicker wall plates than those designed under EN 1998–4 [31]. For example, T1 requires 7 mm for elastic anchorage and 9 mm when hold-down bolts are used, with similar patterns in T2 and T3. These higher thicknesses reflect the larger impulsive base shear and anchorage forces predicted by the NZSEE [30] approach.

The results summarized in Tables 17 and 18 describe the hydrodynamic action components, seismic base shear, damping ratios, and minimum plate thickness requirements for the three benchmark tanks when evaluated according to API 650 [36] Appendix E and ASCE 7. In contrast to EN 1998–4 [31] and NZSEE [30], both U.S. provisions rely on simplified hydrodynamic formulations in which impulsive and convective components are derived from empirical expressions using fixed modal parameters. As a result, the hydrodynamic quantities predicted by the two standards are nearly identical, and the observed differences in seismic demand stem mainly from their behaviour factors and damping assumptions rather than from variations in the spectral input, which remains identical across all analyses.

The base-shear results reflect the influence of these reduction factors. Although API 650 [36] and ASCE 07 [29] yield similar impulsive and convective mass participation, ASCE 7 generally predicts slightly higher base-shear values due to its smaller response-modification coefficients. For example, in Tank T1, the rigid impulsive base shear increases from 1057 kN under API 650 [36] to 1169 kN under ASCE 07 [29], whereas the convective base shear is nearly unchanged. The same pattern appears in T2 and T3, confirming that the differences between the two provisions originate from their reduction factors rather than from their hydrodynamic modelling approaches.

Anchorage forces exhibit a similar hierarchy. For T1, the anchorage demand increases from 525 kN under API 650 [36] to nearly 600 kN under ASCE 07 [29]. These values remain well below those predicted by NZSEE [30], which reflects the substantially lower uplift sensitivity of the U.S. provisions, yet they exceed some of the values from EN 1998–4 [31] where convective effects play a more pronounced role. This places API 650 [36] and ASCE 07 [29] in an intermediate position of conservatism relative to NZSEE [30] and EN 1998–4 [31].

Table 18 shows that both standards adopt uniform damping ratios of 5 % for impulsive components and 0.5 % for convective components. These damping assumptions lead to moderate spectral reductions, contributing to the close agreement in impulsive base-shear values between the two provisions. Their response-modification factors reflect the limited ductility expected from steel tanks under seismic loading, with API 650 [36] applying more conservative coefficients for convective modes, while ASCE 07 [29] permits slightly greater reductions.

The predicted convective wave heights are noticeably lower than those obtained from EN 1998–4 [31] and NZSEE [30]. For all tanks, sloshing amplitudes remain below 0.15 m, primarily because the simplified U.S. formulations produce shorter convective periods and consequently lower spectral amplification. As a result, freeboard demands and roof–liquid interaction effects are far less critical in API 650 [36] and ASCE 07 [29] than in the other international provisions.

A consistent observation across all provisions is that the fundamental impulsive and convective periods remain nearly identical, with Ti $\approx$ 0.09, 0.06, and 0.04 s for Tanks T1–T3 and Tc $\approx$ 3.3–3.5 s for the first convective mode. This similarity reflects the common mechanical analogues adopted by the four standards. Likewise, the total liquid masses—706.8, 471.2, and 314.1 tonnes for T1–T3—are identical across all tables, and the resulting convective masses are nearly the same among the codes. For EN 1998–4 [31], the convective-to-total mass ratio increases systematically from 26 % in T1–39 % in T2 and 54 % in T3, demonstrating the increasing relative significance of sloshing in squat tanks.

The most meaningful differences emerge when base-shear demands are compared. For the tall tank T1, NZSEE [30] predicts the highest rigid-impulsive base shear (2428 kN under M.A.E.), followed by EN 1998–4 [31] (1744.8 kN), ASCE 7 (1169.1 kN), and API 650 [36] (1057.1 kN). The same ranking is maintained for T2 (1141, 882.3, 658.9 and 617.2 kN) and T3 (519, 423.6, 329.3 and 314.4 kN). The NZSEE/EN ratio decreases from about 1.4 in T1 to roughly 1.2 in T3, indicating that NZSEE [30] is particularly conservative for tall tanks dominated by impulsive action.

These differences follow directly from the damping and behaviour factors adopted by each standard. EN 1998–4 [31] uses damping ratios of 2 % and 0.5 % for impulsive and convective modes, with behaviour

Table 17
Seismic action components and hydrodynamic response of the studied tanks (API650/ASCE07 [29]).

Type	Code	Anchorage Condition	Wall Rigidity	Rigid Impulsive (S)	Flexible Impulsive (S)	1st Convective (S)	Total Contained Fluid Mass (ton)	Rigid Impulsive Mass (ton)	Flexible Impulsive Mass (ton)	1st Convective Mass (ton)	Rigid Impulsive Base Shear (kN)	Flexible Impulsive Base Shear (kN)	1st Convective Base Shear (kN)	Anchorage Force (kN)
T1	API650	Anchored	Rigid	0.09	0	3.29	706.8	535.6	0	180.1	1057.1	0	75.7	525.6
T2				0.06		3.33	471.2	292.0		176.2	617.2		72.5	235.7
T3				0.04		3.47	314.1	141.3		162.4	314.4		61.6	106.8
T1	ASCE07			0.09		3.29	706.8	535.6		180.1	1169.1		101	599.2
T2				0.06		3.33	471.2	292.0		176.2	658.9		96.7	263.2
T3				0.04		3.47	314.1	141.3		162.4	329.3		82.1	119.8

factors $qi = 1.5–2.5$ and $qc = 1.0$. NZSEE [30] employs a much larger impulsive damping ratio (14 %) but compensates by assigning very small performance factors—$qi = 0.5$ and $qc = 0.92$ for elastic anchorage and $qi = 0.32$ and $qc = 0.43$ for ductile hold-down systems—leading to high base-shear values despite increased damping. API 650 [36] and ASCE 07 [29] adopt impulsive damping of 5 % and convective damping of 0.5 %, together with much larger reduction factors ($qi = 4$, $qc = 2$ for API 650 [36]; $qi = 3$, $qc = 1.5$ for ASCE 07 [29]). Under the same EN 1998–4 [31] input spectrum, these choices naturally position NZSEE [30] as the most demanding, API/ASCE as the least, and EN 1998–4 [31] in an intermediate role.

The relative contributions of impulsive and convective forces further distinguish the codes. In EN 1998–4 [31], the rigid impulsive base shear in T1 (1744.8 kN) greatly exceeds the total convective base shear ($\sim$169 kN), with similar patterns in T2 and T3. Even in the squat tank T3, impulsive action remains dominant, although the convective share grows. API 650 [36] and ASCE 07 [29] display an even stronger impulsive dominance, with convective base shears of 75–101 kN in T1 and 62–82 kN in T3, compared with impulsive values of 1057–1169 kN and 314–329 kN. NZSEE [30] yields somewhat larger convective base shears than EN 1998–4 [31], but the main source of its conservatism remains its impulsive component combined with an absolute-sum combination rule for flexible-wall tanks.

The differences among the provisions also extend to anchorage demand and minimum wall thickness. Under EN 1998–4 [31], switching from conventional mechanical anchorage (M.A.) to ductile anchor bolts (M.A.D.B.) in T1 reduces anchorage demand from 865.4 kN to 672.0 kN without altering the minimum required thickness for rigid-wall cases (6 mm). Under NZSEE [30], however, adopting ductile hold-down bolts increases both shell thickness and sloshing amplitude: for T1 the required thickness rises from 7 mm (M.A.E.) to 9 mm (M.A.D.H.), and the maximum convective amplitude increases from 0.39 m to 0.83 m. This behaviour follows NZSEE's [30] capacity-design philosophy, in which ductile anchorage is permitted only when the shell and freeboard offer sufficient overstrength. API 650 [36] and ASCE 07 [29] predict the smallest sloshing amplitudes—approximately 0.11 m in API 650 [36] and 0.14–0.15 m in ASCE 07 [29]—compared with 0.32–0.36 m in EN 1998–4 [31] and 0.35–0.39 m (up to 0.83 m in ductile configurations) in NZSEE [30]. Because all four provisions use the same EN 1998–4 [31] spectrum, the lower sloshing amplitudes in the U.S. standards directly reflect their larger convective behaviour factors.

Overall, the comparison across Tables 13–18 shows that all four codes employ similar physical representations of impulsive and convective behaviour, yet differ substantially in the reduction factors, damping assumptions, and combination rules that ultimately govern design forces. NZSEE [30] is the most conservative, particularly for tall tanks where impulsive action dominates; EN 1998–4 [31] provides a balanced treatment of rigid and flexible impulsive modes; and API 650 [36] and ASCE 07 [29], due to their larger reduction factors, predict substantially smaller base-shear and anchorage demands while still producing similar convective mass and period estimates.

15. Conclusions

This review provided a unified evaluation of the seismic behaviour of ground-supported tanks and pressure vessels across major international standards. Although both systems share fundamental fluid–structure interaction mechanisms, the analysis clearly shows that their seismic response, dominant failure patterns, and design requirements differ substantially. Storage tanks have long benefited from targeted research and evolving code provisions, while pressure vessels remain significantly underrepresented, lacking dedicated seismic design rules, consistent behavioural criteria, and comprehensive modelling guidance. This persistent gap represents one of the most critical shortcomings in the seismic design of industrial facilities.

Across the examined standards, discrepancies in seismic demand

Table 18
Damping ratios, response coefficients, and minimum thickness requirements for the studied tanks (API 650/ASCE 07 [29]).

Type	Code	Anchorage Condition	Wall Rigidity	Imp. Damping Ratio %	Con. Damping Ratio %	qi	qc	Max Convective Wave (m)
T1	API650	Anchored	Rigid	5	0.5	4	2	0.11
T2								0.11
T3								0.10
T1	ASCE07			5	0.5	3	1.5	0.15
T2								0.15
T3								0.14

predictions arise primarily from differences in seismic design philosophy—such as assumptions regarding damping, behaviour (R/q) factors, anchorage details, and load-combination schemes—rather than differences in the underlying hydrodynamic modelling. Evidence from past earthquakes reinforces this observation, repeatedly documenting vulnerabilities including shell buckling, uplift-related deformation, sloshing-induced roof failures, anchorage distress, and strong soil-–structure–fluid interaction effects. Classical linear procedures and simplified analytical analogies are unable to capture these complex behaviours, especially for pressure vessels, where slender geometry, elevated-temperature operation, ageing, corrosion, and interaction with piping systems introduce additional challenges that current standards do not explicitly address.

These observations lead to several key insights that summarise the overarching implications of this study:

- Despite similarities in impulsive–convective modelling across codes, seismic provisions for tanks and vessels diverge substantially in critical design parameters.
- Pressure vessels lack a coherent, dedicated seismic design methodology, revealing a major regulatory and research gap.
- The contrast between observed earthquake performance and code-based predictions highlights the need for behaviour-oriented, performance-based design approaches.
- Linear or equivalent-static procedures cannot adequately represent nonlinear uplift, higher-mode effects, three-dimensional sloshing, or coupled SSI–FSI mechanisms.

Because advanced nonlinear three-dimensional SSI–FSI models continue to progress rapidly, comparison with simplified code procedures remains inherently constrained; nevertheless, this limitation underscores the necessity for modernising existing design frameworks. To address these gaps, future research and code development should prioritise several specific directions:

- Development of performance-based seismic design methodologies tailored to both tanks and pressure vessels.
- Integration of validated 3-D SSI–FSI modelling strategies into practical design provisions.
- Establishment of dedicated seismic rules for pressure vessels that explicitly consider high internal pressure, temperature-dependent material behaviour, ageing, and corrosion.
- Clearer code-level guidance on seismic isolation systems, supplemental damping devices, and improved anchorage technologies.
- Experimental and numerical investigation of nonlinear uplift mechanisms, shell instability, and coupled dynamic response under strong ground motions.
- Systematic assessment of interaction effects among multiple tanks and vessels in dense industrial facilities.
- Formulation of a cohesive classification of acceptable material grades for tanks and vessels in high-seismic regions, ensuring consistent requirements for ductility, toughness, elevated-temperature performance, and long-term durability—an issue currently fragmented or absent across existing standards.

Taken together, these findings underline the need for future seismic standards to evolve toward a harmonised, behaviour-based, and analytically modern framework—one that recognises tanks and pressure vessels as distinct systems with different vulnerabilities and modelling requirements. Advancing in this direction is essential for improving the seismic resilience of industrial storage infrastructure and reducing the likelihood of catastrophic failures in future earthquakes.

CRediT authorship contribution statement

Marco Andreini: Writing – review & editing, Validation, Supervision, Resources, Project administration, Funding acquisition. **Simon Marsh:** Writing – review & editing, Validation, Investigation, Funding acquisition. **Amirhossein Nikpour:** Writing – review & editing, Writing – original draft, Validation, Methodology, Investigation, Formal analysis, Data curation, Conceptualization. **D'Aniello Mario:** Writing – review & editing, Writing – original draft, Visualization, Validation, Supervision, Resources, Project administration, Methodology, Investigation, Funding acquisition, Conceptualization. **Saverio La Mendola:** Writing – review & editing, Supervision, Resources, Project administration, Investigation, Funding acquisition. **Roberto Nascimbene:** Writing – review & editing, Writing – original draft, Visualization, Validation, Supervision, Methodology, Investigation.

Declaration of Competing Interest

I declare that the Authors do not have any competing financial interests or personal relationships that might have influenced the research and the relevant results described in the submitted manuscript titled “*Seismic Analysis of Tanks and Vessels: A Comprehensive Review of Global Codes and Guidelines*”.

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APPENDIX A

The tables summarizing the formulas and their definitions are provided in this appendix.

Table A1
Irregularities and Structures Recommended for Dynamic Analysis [36]

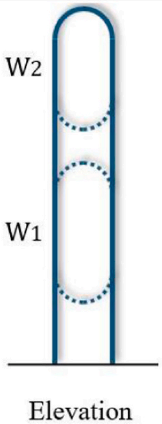
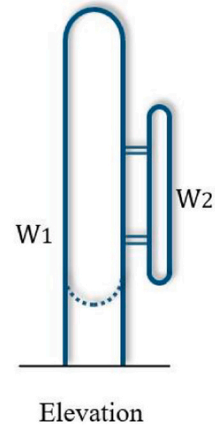
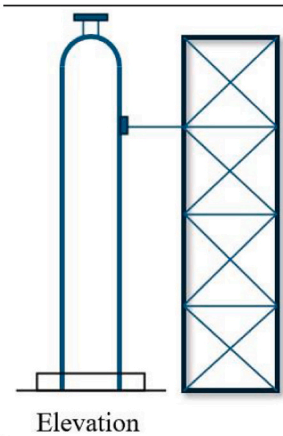
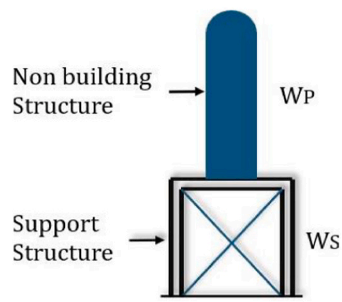
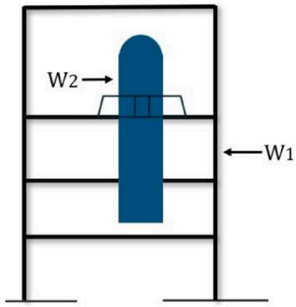
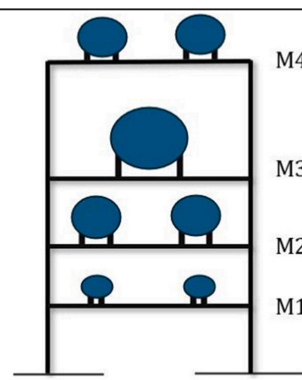
Structures sketch	Condition	Structures sketch	Condition
	Stacked vertical vessels with mass irregularity. (*W1*>1.5*W2 or W1*<0.67*W2*).		Vertical vessels with large, attached vessels, and *W2* ≥0.25*(W1 + W2)*
	Nonbuilding braced frame structures that provide nonuniform horizontal support to the equipment. Interaction should be considered.		Flexible nonbuilding structure that is supported on a relatively flexible elevated support structure and *Wp*≥0.25*(Ws + Wp)*.
	Nonbuilding structures with lug supported equipment and *W2*>0.25*(W1+W2)*. Interaction should be considered.		Nonbuilding structure with mass irregularity. *M3*>1.5*M2* or 1.5*M4*.

Table A2
Accepted Damping Ratios for Impulsive and Convective Modes and Mode Combination Methods

Code	Condition		Imp.%				Con. %	Modal Combination
				vs (m/s)				
NZSEE [26]			H/R	1000	500	200		
	Concrete Tanks & Stiff Steel Tanks (t/R = 0.002)		0.5	4	13	30	0.5	SRSS, DS*
			1	4	10	20		
			2	3	5	14		
			3	2	3	7		
	Steel Tanks (t/R = 0.001)		0.5	3	7	20		
			1	3	6	15		
			2	2	3	6		
			3	2	2	3		
	Flexible Steel Tanks (t/R = 0.0005)		0.5	2	4	12		
			1	2	4	9		
			2	2	3	4		
			3	2	2	3		
EN 1998–4 [27]			5				0.5	SRSS, CQC, DS
ASCE 07–22 [28]	Chapter 13		N/A				N/A	N/A
	Chapter 15		5				0.5	DS, SRSS (the modal periods are separated), CQC (significant modal coupling may occur)
AIJ [29]	Roof Condition	Free Surface (Fixed Roof)	5				0.1	SRSS
		Single deck (Floating Roof)	5				0.5	
		Double- Deck (Floating Roof)	5				1	
IITK-GSDMA [30]	Steel tanks		2				0.5	SRSS
	Concrete or masonry tanks.		5				0.5	
API 620 [32]			5				0.5	SRSS, DS
API 650 [33]			5				0.5	SRSS, DS

Table A3
Base Shear Formula Prescribed by Each Code

Code	Impulsive		Convective	
NZSEE [26]	$C_d(T_i) = C_h(T_i), Z, R_u, N(T_i, D), k_f(\mu, \xi_i).S_p$ $C_h(T_i)$= Basic seismic hazard coefficient; T = Natural period; R_u= Limit state factor; Z = Zone factor; R= Risk factor.S_p= Performance factor; $k_f(\mu.\xi_i)$= Correction factor that depends on ductility factor (μ) and damping factor (ξ_i).			
EN 1998–4 [27]	$(C_s)_i = \gamma_1 S_e$ or $(C_s)_i = \gamma_1 S_d$		$(C_s)_i = \gamma_1 S_e$	
	$S_e = \alpha S\left[1 + \frac{T}{T_B}(2,5\eta - 1)\right]$	For $0 \leq T < T_B$	$S_d = \alpha.\ S\left[1 + \frac{T}{T_B}(\frac{2,5}{q} - 1)\right]$	for $0 \leq T < T_B$
	$= 2.5\alpha S\ \eta$	For $T_B \leq T < T_c$	$= 2.5.\alpha.\ \frac{S}{q}$	for $T_B \leq T < T_c$
	$= 2.5\alpha S\ \eta(\frac{T_c}{T})$	For $T_c \leq T < 3$	$= 2.5\alpha\ \frac{S}{q}\ (\frac{T_c}{T})^{2/3} \geq 0,2\alpha$	for $T_c \leq T < 3$
	$= 7.5\alpha S\eta(\frac{T_c}{T^2})$	For $3 < T$	$= 39\alpha\frac{S}{q}\ (\frac{T_c{}^{2/3}}{T^{5/3}}) \geq 0,2\alpha$	for $3 < T$
	$\eta = \left(\frac{7}{2+\xi}\right)^{0,5}$			
	S_e= Elastic spectrum; S_d= Design spectrum; γ_1= Importance factor; α= Peak ground acceleration factor.S= Soil factor; η= Damping factor; ξ= Viscous damping ratio; q= Behaviour factor; T = Natural period.T_B and T_c= Periods at which constant-acceleration and constant-velocity range begin.			
ASCE 07–22Chapter 15 [28]	$(C_s)_i = \frac{S_{DS}I}{R}$	For $T_i \leq T_s$	$(C_s)_c = \frac{S_{D1}I}{T_c} \leq S_{Ds}I$	for $T_c \leq T_L$
	$= \frac{S_{D1}I}{RT_i}$	For $T_s < T_i \leq T_L$	$= \frac{S_{D1}IT_L}{T_c{}^2}$	for $T_c > T_L$
	$= \frac{S_{D1}IT_L}{RT_i{}^2}$	For $T_i > T_L$		
	$\geq 0.5S_1$			

(continued on next page)

Table A3 (*continued*)

Code	Impulsive		Convective	
	I= Importance factor; R= Response modification factor; T_i= Natural period of impulsive mode; T_c= Natural period of convective mode; S_{DS} and S_{D1}= Design spectra response coefficients; T_s=S_{D1} /S_{DS}; T_L = Transition period for long-period range; S_1= Mapped maximum considered earthquake spectral response acceleration at a period of 1 s			
API 620 [32]API 650 [33]	$(C_s)_i$=$S_{DS}\frac{I}{R_{wi}}$ =$QF_aS_a\frac{I}{R_{wi}}$≥ 0.007	S_1 < 0,6	$(C_s)_c$ =KS_{D1} $\frac{I}{T_c\ R_{wc}}$= $2.5KQF_aS_1\frac{IT_s}{T_c\ R_{wc}}$	for $T_c \le T_L$
	$= 0,5S_1\frac{I}{R_{wi}} = 0.625S_p\frac{I}{R_{wi}}$	$S_1 \ge 0,6$	$(C_s)_c$ =KS_{D1} $\frac{IT_L}{{T_c}^2\ R_{wc}}$= $2.5KQF_aS_1\frac{IT_sT_L}{{T_c}^2\ R_{wc}}$	for $T_c > T_L$
	S= Soil factor; I= importance factor; R_{wc} , and R_{wi}= response modification factors; F_a= Acceleration-based site coefficient at 0.2 s period; S_{DS} and S_{D1} = Design spectra response coefficients; S_1= Mapped maximum considered earthquake spectral response acceleration at a period of 1 s; T_c= Natural period of convective mode; T_L= Transition period for long-period range; K= Coefficient to adjust the spectral acceleration from 5 to 0.5 % damping, equal to 1.5			
AIJ [29]	$(C_s)_i$=$Z_SID_S\frac{S_{a1}}{g} \ge 0.3Z_SID_S$ $= D_hD_\eta$		$S_{vj} = 1,56T_jS_{aj} = 9,8$	For $T_j < T_G$
			$S_{vj} = 1,56T_GS_{aj} = 9,8\frac{T_G}{T_j}$	For $T_j \ge T_G$
	S_a1= Design acceleration response spectrum corresponding to the first natural period; g= Acceleration of gravity.Z_S= Seismic zone factor, which is the value stipulated in the Building Standards Law and the local government. I= Importance factor; D_S= Structural characteristic coefficient; S_vj= Design velocity response spectrum. S_aj= Design acceleration response spectrum; D_h= The coefficient determined by the damping of the structure. D_η= The coefficient determined by the ductility of the structure; T_j= j-th natural period of the structure (s). T_G= Critical period determined by the ground classification			

Table A4
Base Shear Formula Prescribed by Each Code

Code/Book	Impulsive		Convective	
IITK-GSDMA [30]	$\frac{Z}{2}\frac{I}{R}(\frac{S_a}{g})_i$	For hard soil sites (5 % damping) $(\frac{S_a}{g})_i = 2.5$ for $T_i < 0,4$ $(\frac{S_a}{g})_i = \frac{1}{T}$ for $T_i \ge 0.4$ For medium soil sites (5 % damping) $(\frac{S_a}{g})_i = 2.5$ for $T_i < 0,55$ $(\frac{S_a}{g})_i = \frac{1,36}{T}$ for $T_i \ge 0.55$ For soft soil sites (5 % damping) $(\frac{S_a}{g})_i = 2.5$ for $T_i < 0,67$ $(\frac{S_a}{g})_i = \frac{1,67}{T}$ for $T_i \ge 0.67$	$\frac{Z}{2}\frac{I}{R}(\frac{S_a}{g})_c$	For hard soil sites (0.5 % Damping) $(\frac{S_a}{g})_c = (1.75) \times 2.5$ for $T_c < 0,4$ $(\frac{S_a}{g})_c = (1.75)\times\frac{1}{T}$ for $T_c \ge 0.4$ For medium soil sites(0.5 % Damping) $(\frac{S_a}{g})_c = (1.75) \times 2.5$ for $T_c < 0,55$ $(\frac{S_a}{g})_c = (1.75) \times\frac{1,36}{T}$ for $T_c \ge 0.55$ For soft soil sites (0.5 % Damping) $(\frac{S_a}{g})_c = (1.75) \times 2.5$ for $T_c < 0,67$ $(\frac{S_a}{g})_c = (1.75) \times\frac{1,67}{T}$ for $T_c \ge 0.67$
	Z = Zone factor; I= Importance factor; R= Response reduction factor; $\frac{S_a}{g}$= Average response acceleration coefficient; Multiplying factor for 0.5 % damping shall be taken as (1.75); If time period is less than 0.1 s, the value of $\frac{S_a}{g}$ shall be taken as 2.5 for 5 % damping and be multiplied with appropriate factor, for other damping.			

Table A5
Prescribed Base Shear Formula for Vessels and Tanks under Chapter 13 of ASCE 07 [28]

ASCE 07–22 [28] Chapter 13	
Impulsive $F_P = 0,4a_p S_{DS} W_p \left(\frac{H_f}{R_\mu}\right)\left(\frac{C_{AR}}{R_{po}}\right) \geq 0,3S_{DS} W_p I_p$	Not greater than $F_P = 1,6 \quad S_{DS} W_p I_p$
$H_f = 1 + a_1\left(\frac{z}{h}\right) + a_2\left(\frac{z}{h}\right)^{10}$ $H_f = 1 + 2,5\left(\frac{z}{h}\right)$	$a_1 = \frac{1}{T_a} \leq 2.5$ $a_2 = \left[1 - (\frac{0,4}{T_a})^2\right] \geq 0$
$R_\mu = [1,1R/(I_e\Omega_0)]^{0,5}$	

F_P= Seismic design force; S_{DS}= Spectral acceleration, short period; I_p= Component importance factor.W_p= Component operating weight; H_f = Factor for force amplification as a function of height in the structure; R_μ= Structure ductility reduction factor; R_p = Component strength factor; h= Average roof height with respect to the base; I_e= Importance factor; C_{AR}= Component resonance ductility factor that converts the peak floor or ground acceleration into the peak component acceleration; z = Height above the base of the structure to the point of attachment of the component. For items at or below the base, z shall be taken as 0. The value of $\frac{z}{h}$ need not exceed 1.0; T_a=Lowest approximate fundamental period of the supporting building or nonbuilding structure in either orthogonal direction. For structures with combinations of seismic force resisting systems (SFRSs), the SFRS that produces the lowest value of Ta shall be used; R= Response modification factor for the building or nonbuilding structure supporting the component; Ω_0=Overstrength factor for the building or nonbuilding structure supporting the component.

Table A6
Convective and Impulsive Period of Vibration for Tanks

Code	Tanks Condition				Convective		Impulsive
NZSEE [26]	Circular		Horizontal	Rigid	Charts Exist (Figure C3.21)		$T_i = 2\pi\sqrt{\frac{m_0 + m_w + m_t + m_\theta}{K_x} + \frac{m_r h_0^2}{K_\theta}} K_x = \frac{8}{2-\upsilon_s} G_s R_b \alpha_x K_\theta = \frac{8}{3(1-\upsilon_s)} G_s R_b^3 \alpha_\theta$
				Flexible	Flexibility does not affect the Sloshing mode		$T_i = 2\pi\sqrt{\frac{m_r + m_\theta}{K_x} + \frac{m_r h_r^2}{K_\theta}} K_x = \frac{8}{2-\upsilon_s} G_s R_b \alpha_x K_\theta = \frac{8}{3(1-\upsilon_s)} G_s R_b^3 \alpha_\theta$
			Vertical	Rigid	$T_c = \frac{2\pi\sqrt{\frac{R}{g}}}{\sqrt{\lambda_i \tanh\left(\lambda_i \frac{H}{R}\right)}}$	$\lambda_1 = 1.841$	Similar to Horizontal Circular Rigid Tanks
				Flexible	Flexibility does not affect the Sloshing mode		$T_f = \frac{5,61\pi H}{k_h}\sqrt{\frac{\gamma_l}{Eg}}$
	Rectangular			Rigid	$T_c = \frac{2\pi\sqrt{\frac{L}{g}}}{\sqrt{\alpha_i \tanh\left(\alpha_i \frac{H}{L}\right)}}$	$\alpha_1 = \pi/2$	Similar to Horizontal Circular Rigid Tanks
				Flexible	Flexibility does not affect the Sloshing mode		$T_f = 2\pi\sqrt{\frac{d_f}{g}}$
		Other Shapes		Rigid	Charts Exist (Figure C3.21)		Similar to Horizontal Circular Rigid Tanks
API 650 API 620 [32] [33]		Circular	Vertical	Rigid	$T_c = K_s\sqrt{D} = \frac{0,578}{\sqrt{\tanh(\frac{3,68H}{D})}}\sqrt{D}$		$T_i = (\frac{1}{\sqrt{2000}})(\frac{C_i H}{\sqrt{\frac{t_u}{D}}})(\frac{\sqrt{\rho}}{\sqrt{E}})$
ASCE 07–22 [28]		Circular	Vertical	Rigid	$T_c = 2\pi\sqrt{\frac{D}{3,68g\tanh(3,68\frac{H}{D})}}$		$T_i = (\frac{1}{\sqrt{2000}})(\frac{C_i H}{\sqrt{\frac{t_u}{D}}})(\frac{\sqrt{\rho}}{\sqrt{E}})$

(continued on next page)

Table A6 (*continued*)

Code	Tanks Condition				Convective		Impulsive
Chapter 15							
AIJ [29]	Anchored or Unanchored*	Circular	Vertical	FlexibleBottom Pl.	$T_c = 2\pi\sqrt{\frac{D}{2\ \ \varepsilon_i\ g\tanh(2\varepsilon_i\frac{H_l}{D})}}$	$\varepsilon_1 = 1,841$	$T_1 = 2\pi\sqrt{m_t/K_1}$ (anchored)$K_1 = 48,7r_a^3k_1/H_l^2k_1 = \frac{16\rho}{9f_y}\sqrt{\frac{1,5\rho}{f_y}} = \frac{q_y}{\delta_y} = \frac{\frac{2}{3}t_f}{\frac{8}{3}t_f\frac{f_y^2}{E\rho}}$
				FlexibleBottom and side Pl.	$T_c = 2\pi\sqrt{\frac{D}{2\ \ \varepsilon_i\ g\tanh(2\varepsilon_i\frac{H_l}{D})}}$	$\varepsilon_1 = 1,841$	$T_e = \sqrt{T_1^2 + T_f^2}$ (Unanchored)$T_f = \frac{2}{\lambda}\sqrt{m_0/\pi E t_{1/3}}\lambda = 0,067(\frac{H_L}{D})^2 - 0,3\left(\frac{H_L}{D}\right) + 0,46$

T_i = Impulsive period of vibration, T_c = Convective (sloshing) period of vibration, m_0 = Rigid tank impulsive mass, m_w = Wall mass, m_t = Roof mass, m_r = Rigid impulsive mass, m_r= additional lumped mass participating in translation, h_0 = Height of rigid impulsive mass above base, equivalent to hr for a flexible tank, K_x = Horizontal translational stiffness factor of the foundation, K_θ = Rocking stiffness of the foundation, υ_s = Poisson's ratio of soil, G_s = Shear modulus of soil, R_b = Radius of foundation, α_x= Dimensionless horizontal translational stiffness factor that converts the static stiffness to dynamic stiffness, α_θ= Dimensionless rocking stiffness factor that converts the static stiffness to dynamic stiffness, L = Length, g = Gravity acceleration, H = Design liquid level, R = Radius, d_f = deflection of the tank wall on the vertical centre-line at the height of the impulsive mass, K_s = Sloshing period coefficient, t_u = Equivalent uniform thickness of tank shell, C_i = Coefficient for determining impulsive period of tank system, t_f = Thickness, f_y = Yield stress, H_L = Depth of the stored liquid, $t_{1/3}$ = Thickness of tank shell at a height of H/3.

Table A7

Convective and Impulsive Period of Vibration for Tanks

Code	Tanks Condition				Convective		Impulsive
EN 1998–4 [27]	Fixed based	Circular	Vertical	Rigid	$T_c = \frac{2\pi\sqrt{\frac{R}{g}}}{\sqrt{\lambda_i\tanh\left(\lambda_i\frac{H}{R}\right)}}$	$\lambda_1 = 1.841$	Similar to Horizontal Circular Rigid Tanks
		Rectangular		Rigid	$T_c = \frac{2\pi\sqrt{\frac{L}{g}}}{\sqrt{\alpha_i\tanh\left(\alpha_i\frac{H}{L}\right)}}$	$\alpha_1 = \pi/2$	Similar to Horizontal Circular Rigid Tanks
				Flexible	Flexibility does not affect the Sloshing mode		$T_f = 2\pi\sqrt{\frac{d_f}{g}}$
IITK-GSDMA [30]	Fixed based	Circular	Vertical	Rigid	$T_c = C_c\sqrt{\frac{D}{g}} = \frac{2\pi}{\sqrt{3,68\tanh(3,682\frac{h}{D})}} * \sqrt{\frac{D}{g}}$		$T_i = C_i\frac{h\sqrt{\rho}}{\sqrt{E}\sqrt{\left(\frac{t}{D}\right)}} = \left(\frac{1}{(0,46 - \frac{0,3h}{D} + 0,067\left(\frac{h}{D}\right)^2)\sqrt{\left(\frac{h}{D}\right)}}\right)\frac{h\sqrt{\rho}}{\sqrt{E}\sqrt{\left(\frac{t}{D}\right)}}$
		Rectangular		Rigid	$T_c = C_c\sqrt{\frac{L}{g}} = \frac{2\pi}{\sqrt{3,16\tanh(3,16\frac{h}{L})}} * \sqrt{\frac{L}{g}}$		$T_i = 2\pi\sqrt{\frac{d}{g}}$

T_c = Convective (sloshing) period of vibration, T_i= Impulsive period of vibration, T_f=fundamental natural period related to tank wall deformation without uplifting of bottom plate, R = Radius, g= Gravity acceleration, H = Design liquid level, L = Length, D = Diameter, h = Design liquid level, t = Thickness, ρ = Fluid density, d_f = deflection of the tank wall on the vertical centreline at the height of the impulsive mass.

Table A8
Period of Vibration for Vessels

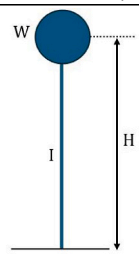
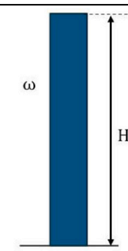
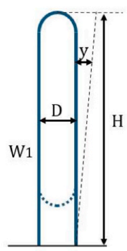
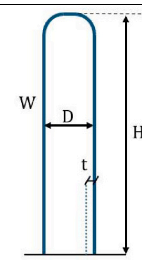
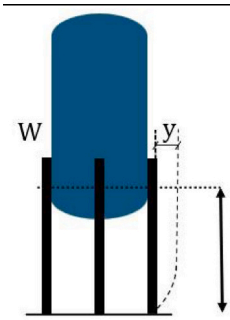
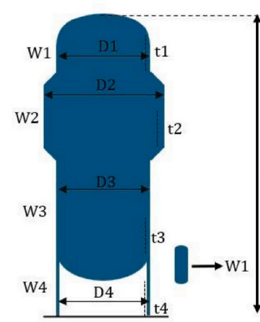
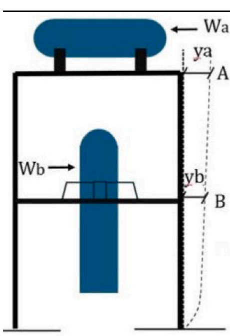
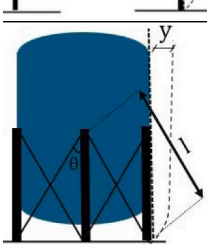
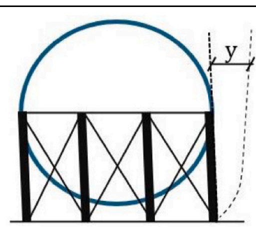
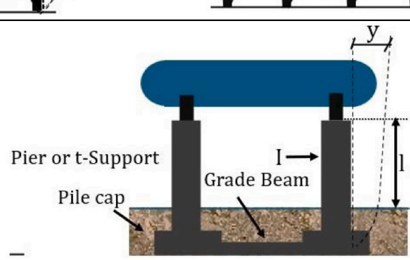
Structural System	Period of Vibration	Structural System	Period of Vibration
	$T=3.63\sqrt{\frac{WH^3}{EIg}}$ *W*= Weight of mass, *H*= Height of cantilever, *E*= Modulus of elasticity, *I*= Moment of inertia, *g*= Gravity acceleration.		$T=1.79\sqrt{\frac{\omega H^4}{EIg}}$ ω= Weight per unit height, *H*= Height, *E*= Modulus of elasticity, *I*= Moment of inertia. *g* = Gravity acceleration.
	$T=\frac{7,65}{10^6}\left(\frac{H}{D}\right)^2\sqrt{\frac{WD}{t}}$ *W*= Weight of mass, *H*= Height, *t*= Shell thickness, *D*= Diameter.		$T=\frac{7,78}{10^6}\left(\frac{H}{D}\right)^2\sqrt{\frac{12WD}{t}}$ *W*= Weight of mass, *H*= Height, *t*= Shell thickness, *D*= Diameter.
	$T=2\pi\sqrt{\frac{y}{g}}$ $y=\frac{2W_o l^3}{3NE(I_x+I_y)}$ I_x , I_y= Moment of inertia of legs, *N*= number of column legs, W_o= Weight of mass, *l*= legs length, *g*= Gravity acceleration,		$T=\left(\frac{H}{100}\right)^2\sqrt{\frac{\sum w\Delta\alpha+\frac{1}{H}\sum W\beta}{\sum ED^{3t}\Delta\gamma}}$ *H*= Overall height, *w*= Distributed weight of each section, *W*= Weight of each concentrated mass, *E*= Modulus of elasticity, *D*= Diameter, *t*= Shell thickness of each section, *α*, *β*, *γ*= The coefficients corresponding to a specific level are determined based on the ratio between the height of that level above grade and the total structure height. The terms *Δα* and *Δγ* represent the variations in *α* and *γ* values from the top to the bottom of each segment characterized by constant weight, diameter, and thickness. The coefficient *β* is calculated individually for each concentrated mass. *α*, *β*, and *γ* values are presented in the [36].
	$T=\sqrt{\frac{W_a y_a+W_b y_b+\sqrt{(W_a y_a-W_b y_b)^2+4W_a W_b y_{ab}^2}}{2g}}$ y_a = deflection at A, y_b = deflection at B, y_{ab} = deflection at B due to lateral load at A, W_a = Weight of mass, W_b = Weight of mass, *g*= Gravity acceleration.		
		$T=2\pi\sqrt{\frac{y}{g}}$ $y=\frac{W_o l sin\theta^2}{6EA}$ -Legs over 2.13 m (7 ft) should be cross braced [37], *E*= Modulus of elasticity, *A*= Cross section area, *l*= Length, W_o= Weight of mass, *g*= Gravity acceleration.	
		$T=0.32\sqrt{y}$ $y=\frac{W_o l^3}{3EI_m}$ I_m= moment of inertia of pier, legs, stack, etc., W_o= Weight of mass, *E*= Modulus of elasticity, *l*=Pier's length, *g*= Gravity acceleration.	

Table A9
Response Modification Factor and Behaviour Factor for Tanks Baes on NZSEE [26]

Type of base		Impulsiveζ%							Convectiveζ%
		0.5 $C_f(\mu, \zeta_i)$	1	2	5	10	15	20	0.5 $C_f(\mu, \zeta_i)$
	μ	Steel Tanks on Grade							
Elastically supported	1.25	0.92	0.88	0.83	0.72	0.62	0.58	0.55	0.92
Unanchored tank designed for uplift elephant foot shell buckling may occur under seismic overload	2.0	0.58	0.56	0.54	0.48	0.44	0.42	0.4	0.58
Unanchored tank designed for uplift and elastic diamond-shaped shell buckling mode	1.25	0.92	0.88	0.83	0.72	0.62	0.58	0.55	0.92
Anchored with nonductile hold-down bolts	1.25	0.92	0.88	0.83	0.72	0.62	0.58	0.55	0.92
Anchored with ductile tension yielding hold-down bolts	3.0	0.43	0.43	0.41	0.38	0.35	0.33	0.32	0.43
Ductile skirt pedestal	3.0	0.43	0.43	0.41	0.38	0.35	0.33	0.32	0.43
On concrete base pad designed for rocking	2.0	0.58	0.56	0.54	0.48	0.44	0.42	0.4	0.58
		Concrete Tanks on Grade							
Reinforced concrete	1.25	0.92	0.88	0.83	0.72	0.62	0.58	0.55	0.92
Prestressed concrete	1.0	1.75	1.57	1.33	1	0.8	0.71	0.67	1.75
		Tanks of other materials on Grade							
Timber	1	1.75	1.57	1.33	1	0.8	0.71	0.67	1.75
non-ductile materials (e.g. fibreglass)	1	1.75	1.57	1.33	1	0.8	0.71	0.67	1.75
Ductile materials and failure mechanisms	3	0.43	0.43	0.41	0.38	0.35	0.33	0.32	0.43
Elevated Tanks									

Table A10
Response Modification Factor and Behaviour Factor for Tanks

Type of base	Code	ASCE 07Chapter 15 [28]		IITK-GSDMA [30]		API [32] [33]		EN 1998–4 [27]	
	Mode	Imp.	Con.	Imp.	Con.	Imp.	Con.	Imp.	Con.
Ground-Supported RC/PSC Tanks									
Anchored Flexible base		3.25	1.5	2.5	1	N/A	N/A	q up to 1.5(Designed for Elastic Response)	1
Reinforced non-Sliding		2	1.5	N/A	N/A	N/A	N/A		
Unanchored and Contained Flexible		N/A	N/A	1.5	1	N/A	N/A		
Unanchored and Uncontained Flexible		1.5	1.5	1.5	1	N/A	N/A		
Fixed or hinged/ pinned base tank		N/A	N/A	2	1	N/A	N/A		
		Ground-Supported Steel Tanks							
Mechanically anchored		3	1.5	2.5	1	4	2	2.5 (With ductile anchored bolts)	1
Self-anchored		2.5	1.5	2	1	3.5	2	Greater than 1.5 but not larger than 2.0	
		Elevated Tanks							
RC pedestal		2	1.5	N/A	N/A	N/A	N/A	q up to 1.5(For DCL Class Substructure)q greater than 1.5(For DCM / DCH Class Substructure - The upper limit values of q defined in EN 1998–1:2004)	1
Braced/ unbraced legs		3	1.5	N/A	N/A	N/A	N/A		
Tank supported on RC shaft		N/A	N/A	1.8	1	N/A	N/A		
Tank supported on RC frame		N/A	N/A	1.8–2.5	1	N/A	N/A		
Tank supported on a steel frame		N/A	N/A	2.5	1	N/A	N/A		
Tank supported on masonry shaft		N/A	N/A	1.3–1.5	1	N/A	N/A	N/A	N/A

Table A11
Response Modification Factor for Tanks and Vessels Baes on ASCE 07 [28] Chapter 15

Type of Structure		R	Ω_0	
Elevated Tanks, Vessels, Bins	On Symmetrically Braced Integral Legs(not similar to buildings)	3	2	2.5
	On Unbraced Integral Legs/ Asymmetrically Braced Integral Legs (not similar to buildings)	2	2	2.5

(continued on next page)

Table A11 (*continued*)

Type of Structure		R	Ω_0	
Horizontal, Saddle-Supported Welded Steel Vessels		3	2	2.5
All Steel and Reinforced Concrete Distributed Mass Cantilever Structures Not Otherwise Covered Herein, Including Stacks, Chimneys, Silos, Skirt supported Vertical Vessels; Single-Pedestal or Skirt-Supported	Welded steel	2	2	2
	Welded steel with special detailing	3	2	2
	Prestressed or reinforced concrete	2	2	2
	Prestressed or reinforced concrete with special detailing	3	2	2
All Other Self-Supporting Structures, Tanks, or Vessels Not Covered Above or By Reference Standards That Are Not Similar to Buildings		1.25	2	2.5
Reinforced Concrete Tabletop Structure (Not Similar to Buildings) Supporting Elevated Tanks, Vessels, Bins.		2	2	2

Table A12
Response Modification Factor for Tanks and Vessels Baes on ASCE 07 [28] Chapter 13

Type of Structure	C_{AR}		R_{po}	Ω_{0P}
	Supported at orBelow Grade Plane	Supported Above Grade Plane by a Structure		
Atmospheric Tanks, Boilers, Bins	1	1	1.5	2
Pressure Vessels Not Supported on Skirts(not within the scope of Chapter 15)	1	1	1.5	2
Pressure Vessels Skirt-Supported (not within the scope of Chapter 15)	1.8	2.2	1.5	1.75

Table A13
Response Modification Factor for Tanks and Vessels Baes on AIJ [29]

Type of Tank	Type of base	structural characteristic coefficient
Above-ground, Vertical, Cylindrical Storage Tanks	Unanchored Tanks	$D_s = \dfrac{1.42}{1+3h+1.2\sqrt{h}} \dfrac{1}{\sqrt{1+24(\frac{T_1}{T_e})^2}}$
	Anchored tanks	$D_s = \dfrac{1.42}{1+3h+1.2\sqrt{h}} \dfrac{1}{\sqrt{1+\frac{3.3t\sigma_y{}^2}{l_a p a^{\sigma_y}}(\frac{T_1}{T_e})^2}}$
	Buckling of Tank Wall Plate	$D_s = \dfrac{1.42}{1+3h+1.2\sqrt{h}} \dfrac{1}{\sqrt{1+3(\frac{T_f}{T_e})^2}}$
Support structures for Spherical Tanks	$\alpha = 0.75$ for $N_T < 2N_c$ $\alpha = 1$ for $N_T \geq 2N_c$	$D_s = \dfrac{1}{\sqrt{1+4\alpha\eta}}$ $\eta = \dfrac{\delta_m - \delta_y}{\delta_y}\alpha\eta \leq 3$

D_h = The coefficient determined by the damping of the structure, D_η = The coefficient determined by the ductility of the structure, D_s = Structural characteristic coefficient, h= Damping ratio, T_1 = Natural period of the storage tank when only the bottom plate and anchor bolts (or anchor straps) deform (s), T_e = Modified natural period of the storage tank considering deformation of the wall plate as well as the bottom plate and anchorage (s), σ_y = Yield stress of the bottom plate (N/mm2), a^{σ_y} = Yield stress of anchor bolt or anchor strap (N/mm2), l_a = Effective length of anchor bolt or anchor strap (mm), t= Thickness of the bottom plate (mm), p = Static pressure imposed on the bottom plate (N/mm2), Y_r = The ratio of yield stress to ultimate tensile strength of annular plate, T_f = Fundamental natural period related to tank wall deformation without uplifting of the bottom plate, η= Mean cumulative plastic deformation ratio of the supporting structure, α = Correction factor for η considered the reduction of the strength of the bracings, N_C = Short-term allowable compressive strength of bracings (kN), N_T = Short-term allowable tensile strength of the bracings (kN), δ_m = Horizontal displacement of the column head when a supporting column reaches the short-term allowable strength (m), δ_y = Horizontal displacement of the column head when a bracing reaches the short-term allowable strength (m).

Table A14
Importance Factor Based on ASCE 07 [28] Chapter 13 and Chapter 15

Class	Definition	Seismic Importance Factor	
		I_e (for Structures)	I_p (for Components)
I or II	General (Normal, Special)	1	1
III	Sufficient hazardous materials	1.25	1.5
IV	Essential or containing a quantity of hazardous materials exceeding a prescribed threshold	1.5	1.5

Appendix A. Supporting information

Supplementary data associated with this article can be found in the online version at doi:10.1016/j.istruc.2026.111276.

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