

Article

On the Seismic Response of Elevated Steel Water Tanks Using a Lever-Arm Steel Slit Damper

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Abstract

The seismic behavior of an above-ground steel liquid storage tank supported on steel columns and retrofitted with a compact lever-arm amplification mechanism combined with a steel slit damper (SSD) is investigated in this study. The lever-arm mechanism amplifies the relative displacements transmitted to the SSD, enabling efficient energy dissipation through a compact device. Both the conventional and the retrofitted tank are subjected to nonlinear time history analyses (NLTHA), with soil–structure interaction (SSI) explicitly incorporated into the numerical models. The comparison between the two configurations reveals that the retrofitted tank achieves notable reductions in column drift and hydrodynamic wall pressures. The conventional tank is found to be particularly susceptible under soft-soil conditions, while the lever-arm/SSD system demonstrates consistent mitigation performance across all examined soil profiles. The numerical results indicate that the proposed system not only minimizes structural damage but also achieves an average quantitative reduction of 26% in fluid sloshing wave heights compared to the conventional bare frame. These outcomes establish the proposed passive control scheme as a viable and effective retrofit strategy for the seismic upgrading of elevated steel liquid storage tanks at sites with diverse geotechnical characteristics.

Keywords: liquid storage tank; steel slit damper; lever-arm system; nonlinear time history analysis; soil–structure interaction; site response analysis



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1. Introduction

Elevated steel liquid storage tanks are critical infrastructure components in water supply networks, industrial facilities, and the agri-food sector, where their continued operation during and after seismic events is essential. These structures are inherently vulnerable to earthquake loading due to their dynamic configuration: a heavy mass concentrated at the top of a slender, flexible support system produces low natural frequencies and large lateral displacements under ground shaking [1,2]. The presence of stored liquid adds further complexity, as the dynamic interaction between the fluid and the structure generates both impulsive and convective (sloshing) response components that must be carefully accounted for in any realistic seismic assessment [3].

The seismic behavior and design of elevated tanks have attracted considerable research attention in recent years. Regarding reinforced concrete configurations, an optimization

study demonstrated the strong nonlinearity of the seismic design problem and its sensitivity to column height and ground acceleration level [1], while a separate investigation highlighted the importance of the response reduction factor and the need for nonlinear analysis procedures when assessing the inelastic performance of such structures [2]. On the analytical side, a comprehensive review of fluid–structure interaction formulations for base-supported cylindrical tanks established that omitting structural flexibility in the analysis can lead to a significant underestimation of seismic demands [3]. For elevated steel tanks specifically, studies evaluating passive control strategies have yielded mixed conclusions: one investigation found that a seesaw-based system offered only limited benefits for the tank typologies considered [4], whereas a separate study showed that a curved steel damper interposed between the vessel and its supporting frame could substantially reduce both base shear and wall deformations [5].

Seismic isolation has also been explored as a means of reducing earthquake demands on liquid storage tanks of various geometries and support conditions. Research on conical elevated tanks demonstrated that lead-rubber and elastomeric bearings, when properly sized and positioned, can effectively control the distribution of seismically induced forces and displacements throughout the structure [6]. In the industrial context, full-scale shaking table experiments on legged wine storage tanks confirmed that flat sliding bearing systems reduce leg forces significantly compared to fixed-base configurations [7]. Field evidence from the 2010 Maule earthquake in Chile further underscored the consequences of inadequate seismic provisions for this tank typology, with widespread failures attributed to poor structural detailing, insufficient redundancy, and deficient anchorage [8]. For spherical elevated tanks, numerical studies evaluating energy-dissipative bracing systems, including viscous dampers, buckling-restrained struts, and friction devices, identified the relative merits of each approach in reducing dynamic response indicators [9].

Despite these contributions, the use of displacement-amplification mechanisms for the passive control of elevated steel tanks has not been examined. Such mechanisms operate by magnifying the relative deformation imposed on a supplemental energy dissipation device, thereby improving its efficiency without increasing its physical size. A recently proposed system combining a small-sized double lever-arm mechanism with a steel slit damper was evaluated for multi-story steel frames and shown to produce early device yielding, high cumulative energy dissipation, and meaningful reductions in story drift demand [10]. In recent years, an extensive body of literature has been dedicated to analyzing the complex structural vulnerabilities, hydrodynamic pressure distributions, and specific failure modes observed in industrial facilities and elevated liquid containment systems during major seismic events. Within this framework, passive energy dissipation devices, particularly metallic yield and slit dampers, have been widely recognized as highly effective structural solutions due to their stable hysteretic behavior and substantial energy dissipation through controlled plastic deformation. Recent comprehensive investigations [11] have provided extensive insights into advanced multi-phase energy dissipation systems, thoroughly evaluating a novel brace-type damper that utilizes coupled friction and metallic yielding mechanisms to control multi-stage yielding regimes under severe cyclic responses. Concurrently, state-of-the-art research [12] has expanded the applications of energy-dissipation technologies beyond pure seismic actions, demonstrating the efficiency of lateral damping buffers and fluid-structure interaction dynamics in protecting critical infrastructure elements against hazardous fluid-induced impacts, such as debris flow loads. Integrating these foundational findings regarding advanced mechanical modeling and alternative lateral energy-dissipation applications into the present study allows for a more holistic understanding of how displacement-amplifying passive control systems can be systematically optimized to mitigate multi-hazard lateral risks in elevated liquid storage

configurations. The applicability of this concept to elevated liquid storage tanks, however, remains an open question.

A related aspect that has received limited attention in tank design is the role of local site conditions in shaping seismic demand. Site-specific ground response analysis involves multiple sources of uncertainty, including input motion selection, soil stratigraphy, and nonlinear soil properties, all of which influence the surface acceleration characteristics used as structural input [13]. Empirical evidence from urban site response studies has shown that amplification factors can vary widely within a single city, with soft near-surface deposits producing the largest amplifications and posing the greatest risk to structures founded on them [14]. These findings have direct relevance to the seismic assessment of elevated tanks, yet SSI and site amplification effects are rarely treated in a unified framework in the existing literature.

The present study addresses this gap by examining the seismic response of an elevated steel liquid storage tank, supported on steel columns, before and after retrofitting with the lever-arm/steel slit damper system [10] (Figure 1). The amplification mechanism transmits magnified displacements to the damper, allowing efficient energy dissipation through a compact device. To quantify the seismic performance enhancement, both the conventional and retrofitted tanks were subjected to NLTHA, with SSI explicitly incorporated to account for the influence of soil flexibility on structural response. Site response analyses carried out for several geotechnical profiles are used to generate surface ground motions that reflect realistic local amplification conditions, and these motions serve as input to the structural models. The study evaluates reductions in base shear, column drift, sloshing height, and seismic-induced hydrodynamic pressures on tank walls, with particular focus on the behavior of the unprotected and retrofitted tank under soft-soil excitation.

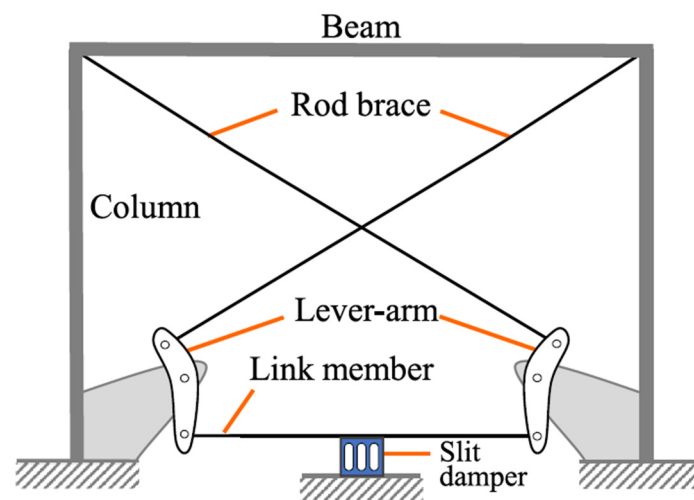


Figure 1. Lever arm slit damper system [10].

2. Description of the Structural System and Numerical Model

2.1. Elevated Steel Tank

The elevated steel tank under study is shown in Figure 2. The tank is rectangular in shape with dimensions $5.30 \text{ m} \times 5.30 \text{ m}$, its height is 4.00 m , and it has no roof. The tank is rigidly supported upon a 3D steel frame comprised of four columns and four beams of a standard rolled SHS section. The height of the columns of the space frame is 6.00 m , and the clear span of each bay is 5.00 m . The steel grade of both the tank shell and the frame members is S275. The thickness of the tank shell is uniform and equals 15 mm . The section of all members of the space frame is SHS $300 \times 300 \times 17.5$.

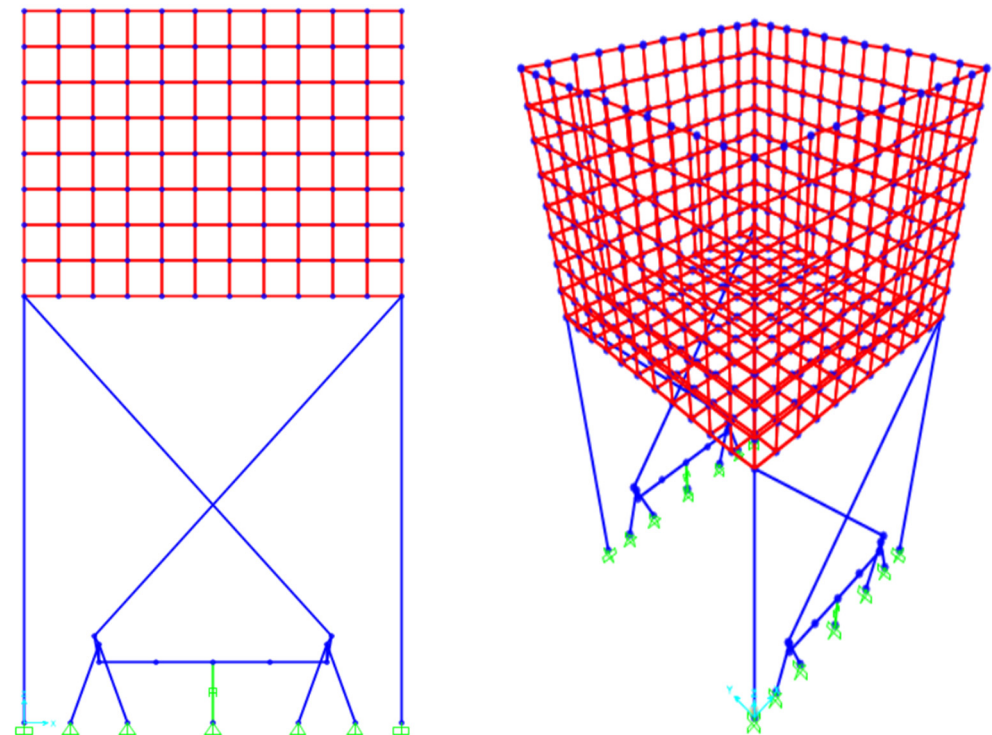


Figure 2. Water tank with lever arm steel slit damper system.

2.2. Lever-Arm System with Steel Slit Damper

The proposed passive control system consists of double small-sized lever-arm members, a link member (steel bar), a steel slit damper (SSD), and a pair of intersecting rod braces, all arranged within the bay of the supporting steel frame. The system is based on the configuration originally proposed by Tagawa et al. [10] for multi-story steel frames and is here adapted for the elevated tank application. As shown in Figure 3, the rod braces connect the top of each lever-arm to the beam–column joints at the top of the supporting frame, while the link member connects the bottom ends of the lever-arms to the upper end of the slit damper. The lower end of the slit damper is anchored to the base of the frame. Support members, arranged diagonally from the base of the frame to the rotation centers of the lever-arms, provide stable pivot points about which the lever-arms rotate, without connecting to the columns of the supporting frame.

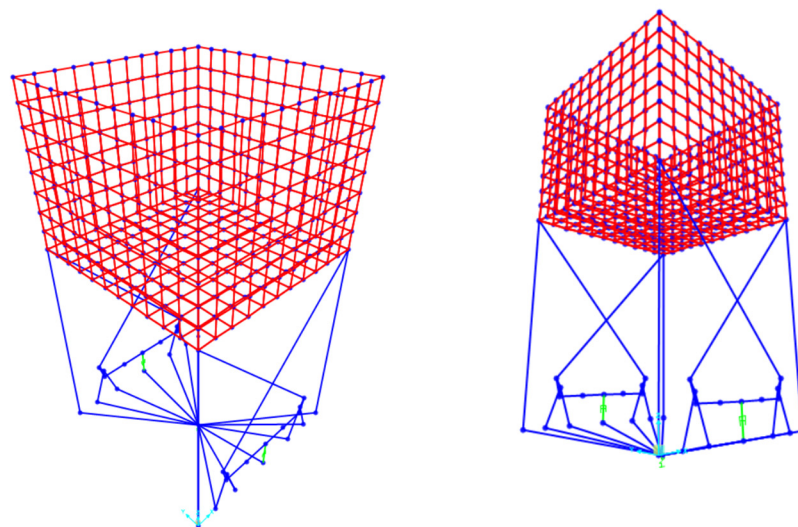


Figure 3. Water tank with lever arm slit damper system founded on flexible soil.

The amplification factor β of the lever-arm is defined as the ratio of the lower to the upper arm length, i.e., $\beta = b/a$, where a and b denote the lengths of the upper and lower arms, respectively. In the present model, the lever-arm height is set to 0.882 m, consistent with the configuration reported in [10]. An amplification factor of $\beta = 2.0$ is adopted, achieved by setting $a = 250$ mm and $b = 500$ mm. The lateral strength of the system is given by

$$F = \beta \cos\theta \cdot F_{Du} \quad (1)$$

where θ is the horizontal inclination angle of the rod brace, and F_{Du} is the shear strength of the slit damper. The displacement amplification factor, defined as the ratio of damper deformation to story drift, is expressed as $f = \beta \cos\theta$, indicating that a larger lever-arm ratio produces proportionally greater damper deformation and thus enhanced energy dissipation for a given story displacement.

2.3. Rod Braces

The rod braces are solid circular steel members with a diameter of 90 mm (D90), consistent with the full-scale analytical model of [10]. Although solid steel rods can, in principle, sustain both tensile and compressive forces, they are designed to act as tension-only elements throughout the seismic response. This is achieved by introducing an initial pretension into the braces prior to the seismic excitation, ensuring that they remain in tension at all times and thus eliminating the risk of buckling under compressive loading. In the SAP2000 [13] model, the rod braces are modeled as frame elements with a circular cross-section of diameter 90 mm, assigned a tension-only property so that no compressive force can develop. The pretension is applied as an initial load condition before the nonlinear time history analysis is initiated.

To ensure that the solid circular rod braces function effectively as slender tension-only elements throughout the seismic event, a controlled initial pre-tension force T_0 is introduced. The determination of T_0 is governed by two competing structural design considerations: Minimum Tension Criterion (Serviceability): The pre-tension must be large enough to prevent the compression-side rod from going slack during structural drift under wind or moderate seismic actions. The minimum force is determined using the kinematic relationship:

$$T_{0,min} = A_b E_s \left(\frac{\Delta_{SLs} \cos\theta}{L_b} \right)$$

where A_b is the cross-sectional area of the rod, E_s is the steel elastic modulus, Δ_{SLs} is the target serviceability lateral drift, θ is the brace inclination angle, and L_b is the total length of the brace.

Maximum Compression Limit (Frame Capacity): To address the risk of introducing high pre-existing compressive forces into the primary columns and horizontal beams under gravity loads, a strict upper bound is enforced. The design limits the initial pre-tension to a small fraction of the rod's capacity:

$$T_0 \leq 0.10 f_y A_b$$

where f_y is the yield strength of the rod steel.

By applying this optimized boundary condition, the pre-tension force for the $\phi 30$ mm solid rods was engineered at a nominal value of 45 kN. Comprehensive gravity-load analysis checks confirmed that this pre-tension level introduces an additional axial compressive demand on the main columns of less than 3.5% of their design buckling capacity, $P_c / \phi^* P_n \leq 0.035$. Therefore, the adverse pre-stressing effects on the vertical load-

resisting frame are structurally negligible while successfully preventing brace slackness during dynamic shaking.

2.4. Numerical Modeling of the Steel Slit Damper in SAP2000

The steel slit damper is modeled in SAP2000 [15] as a nonlinear link element using the Wen (Bouc–Wen) plasticity model, which captures the hysteretic energy dissipation behavior of the device under cyclic loading. The steel slit damper is modeled in SAP2000 using the non-linear Plastic (Wen) link element, which is based on the uncoupled hysteretic formulation originally proposed by Wen (1976) [16]. The governing force–displacement relationship of this link element is expressed as

$$F(t) = \alpha K_D d(t) + (1 - \alpha) F_{Du} z(t)$$

where K_D is the elastic stiffness, F_{Du} is the yield strength, α is the post-yield stiffness ratio, and $z(t)$ is an internal hysteretic variable bounded by ± 1.0 . The evolution of the hysteretic variable $z(t)$ is defined by the following differential equation

$$\dot{z} = \frac{\dot{d}}{d_y} [A - |z|^n (\gamma \cdot \text{sgn}(\dot{d} \cdot z) + \beta)]$$

In the standard SAP2000 implementation, the dimensionless shape parameters are automatically fixed at $A = 1.0$ and $\beta + \gamma = 1.0$ to satisfy thermodynamic consistency and eliminate parameter redundancy. The transition between the elastic and post-yield loading paths is entirely regulated by the yielding exponent n , which is set to 1.0 in this study to represent a smooth elastoplastic backbone curve matching experimental findings for mild steel dampers.

Because structural mild steel slit dampers exhibit highly stable hysteretic loops with substantial energy dissipation capacity and minimal cyclic degradation prior to low-cycle fatigue failure, phenomena such as stiffness degradation, strength degradation, and pinching are omitted in this macro-model. The complete calibrated parameter set assigned to the axial degree of freedom (U1) of the link element is summarized below.

The link element is oriented along the axial (U1) direction, corresponding to the shear deformation of the physical damper as transmitted through the link member. The two governing parameters of the spring element are the elastic stiffness K_D and the yield (shear) strength F_{Du} . A post-yield stiffness ratio of 0.02 is adopted, representing the nearly perfectly plastic behavior of the slit plate struts after yielding. The yield exponent is set to 1.0, corresponding to an elastoplastic backbone curve. The damper properties adopted in the present model are derived from [10]. Specifically, the elastic stiffness is $K_D = 151.1 \text{ kN/mm}$, and the yield strength is $F_{Du} = 602 \text{ kN}$. The complete set of damper properties used in the reference study [10], covering all story levels of the five-story frame, is reproduced in Table 1 for reference.

Table 1. Seismic motions.

No.	Date	Record Name	Comp.	Station Name	PGA (g)
1	4 October 1987	Whittier Narrows	NS	24399 Mt Wilson-CIT Station	0.158
2	4 October 1987	Whittier Narrows	EW	24399 Mt Wilson-CIT Station	0.142
3	20 September 1999	Chi-Chi, Taiwan	056-N	HWA056	0.107
4	20 September 1999	Chi-Chi, Taiwan	056-N	HWA056	0.107
5	1 October 1987	Whittier Narrows	NS	24399 Mt Wilson-CIT Station	0.186
6	1 October 1987	Whittier Narrows	EW	24399 Mt Wilson-CIT Station	0.123
7	9 February 1971	San Fernando	N069	127 Lake Hughes #9	0.157

Table 1. *Cont.*

No.	Date	Record Name	Comp.	Station Name	PGA (g)
8	9 February 1971	San Fernando	N159	127 Lake Hughes #9	0.134
9	17 January 1994	Northridge	NS	90019 San Gabriel-E. Gr. Ave.	0.256
10	17 January 1994	Northridge	EW	90019 San Gabriel-E. Gr. Ave.	0.141
11	20 September 1999	Chi-Chi, Taiwan	NS	TAP103	0.177

The lever-arm members and support members are modeled as rigid frame elements (truss elements with very high axial stiffness). The link member is modeled as a standard frame element.

2.5. Numerical Modeling of the Tank Structure and Fluid Tank Interaction

The tank structure is represented using standard thin shell elements with a uniform wall thickness of 15 mm, rigidly connected to the supporting steel frame, designed according to Eurocode 3 [17]. To account for fluid–tank interaction, the simplified two-mass analogy of Malhotra et al. [18], subsequently incorporated into EC8 [19], is employed. This approach divides the total liquid mass into an impulsive component, which moves together with the tank wall and controls the short-period structural response, and a convective component, which oscillates independently and governs the long-period sloshing behavior. Each component is associated with a characteristic height above the tank base at which its inertial effect is applied, and both masses are uniformly distributed along the shell perimeter to the nodes located at or nearest to these heights. For the tank geometry ($R = 2.65$ m, $H = 4.00$ m), the non-dimensional ratios were obtained from the Malhotra et al. [18] formulation. For the application of the Malhotra hydrodynamic model, the rectangular tank geometry is converted into an equivalent cylindrical configuration. Consequently, an equivalent radius of $R = B/2 = 2.65$ m is adopted, where B represents the tank width parallel to the direction of seismic excitation, to compute the impulsive and convective parameters. To model the dynamic fluid–structure interaction within the elevated steel tank, the simplified two-mass mechanical analogue proposed by Malhotra et al. is utilized [18]. This approach characterizes the fluid behavior by partitioning the total liquid mass, denoted as m , into two distinct response components: an impulsive mass m_i that moves synchronously with the flexible tank shell, and a convective mass m_c that accounts for the independent low-frequency sloshing oscillations of the liquid surface. For the application of this hydrodynamic formulation, the rectangular tank geometry is converted into an equivalent cylindrical configuration with a calculated equivalent radius, R . The operational depth of the stored fluid is represented by the total liquid height, H . The dynamic inertial effects of the fluid are introduced into the structural numerical model by applying the impulsive and convective masses at their respective calculated elevations above the tank base, designated as h_i and h_c . The impulsive fraction amounts to $m_i/m = 0.686$, acting at a normalized height of $h_i/H = 0.439$, while the convective fraction is $m_c/m = 0.314$, located at $h_c/H = 0.69$. To assess the effect of fill level on structural response, two water heights are considered: 2.50 m, representing a partially filled condition, and 3.50 m, corresponding to a near-full tank. The impulsive and convective masses and their respective heights are re-evaluated for each case following the Malhotra et al. [18] procedure. It is worth noting that the full tank height cannot serve as the operational fill level in the absence of a roof, as sloshing would result in overflow.

3. SSI Effects

To evaluate the tank’s vulnerability under unfavorable geotechnical conditions, the structure is analyzed under soft-soil conditions, which are explicitly defined here as a Class

D ground profile in strict accordance with Eurocode 8. This specific soil class represents loose-to-medium cohesionless soils or soft-to-firm cohesive deposits. In this study, the deposit is modeled as a uniform 30 m thick layer characterized by a low shear wave velocity of $v_s = 180$ m/s and a mass density of $\rho = 1900$ kg/m³, resulting in a fundamental site period of $T = 4 H/V_s = 0.67$ s. These parameters define a highly flexible, compliant stratum where soil–structure interaction (SSI) and local site amplification phenomena are most pronounced.

To simulate the seismic response of the elevated steel tank on flexible ground, where significant soil–structure interaction (SSI) effects are expected, the mechanical analogue proposed by Mulliken and Karabalis [20] is adopted. In this approach, the foundation together with the surrounding soil is replaced by a frequency-independent system of springs, dashpots, and masses. In accordance with the soil classification of EC8 [19], the tank is assumed to rest on class D soil, characterized by a shear wave velocity $v_s = 180$ m/s and a mass density $\rho = 1900$ kg/m³, conditions under which SSI effects are most pronounced (EC8 [19]). The mat foundation is rectangular, $7.0 \text{ m} \times 7.0 \text{ m}$ in plan and 1.0 m thick, and its design complies with the ANSI/AWWA [21] provisions for cross-braced multicolumn elevated tanks [22]. For this foundation geometry, the spring–dashpot–mass parameters of the analogue are fully uncoupled.

The soil parameters and foundation geometry are combined to derive the necessary mechanical quantities. The elastic shear modulus is obtained from $G = \rho \cdot v_s^2 = 1.9 \times 180^2 = 61,560$ kN/m². Applying the reduction coefficient $\text{coeff} = 0.50$ specified in EC8 [19], Table 4.1 for class D soil, the effective shear modulus reduces to $G_{\text{eff}} = G \times \text{coeff} = 30,780$ kN/m². The equivalent circular foundation radius is evaluated as $D = \sqrt{(B \cdot A)/4} = \sqrt{(7.0 \times 7.0)/4} = 3.5$ m, and the corresponding effective shear wave velocity becomes $v_{\text{sef}} = \sqrt{(G_{\text{eff}}/\rho)} = \sqrt{(30,780/1.9)} = 127.3$ m/s.

The static stiffness coefficients of the equivalent circular footing are subsequently computed as follows: the vertical stiffness $K_{\text{ver}} = 4.7 \cdot G_{\text{eff}} \cdot D / (1 - \nu) = 4.7 \times 30,780 \times 3.5 / 0.7 = 723,330$ kN/m; the horizontal stiffness $K_{\text{hor}} = 9.2 \cdot G_{\text{eff}} \cdot D / (2 - \nu) = 9.2 \times 30,780 \times 3.5 / 1.7 = 583,009$ kN/m; the rocking stiffness $K_{\text{rock}} = 4 \cdot G_{\text{eff}} \cdot D^3 / (1 - \nu) = 4 \times 30,780 \times 42.875 / 0.7 = 7,541,100$ kN·m/rad; and the torsional stiffness $K_{\text{tor}} = 8.31 \cdot G_{\text{eff}} \cdot D^3 = 8.31 \times 30,780 \times 42.875 = 10,966,645$ kN·m/rad. The associated radiation damping coefficients follow as: $C_{\text{ver}} = 0.8 \cdot D \cdot K_{\text{ver}} / v_{\text{sef}} = 15,912$ kN·s/m; $C_{\text{hor}} = 0.163 \cdot D \cdot K_{\text{hor}} / v_{\text{sef}} = 2613$ kN·s/m; $C_{\text{rock}} = 0.6 \cdot D \cdot K_{\text{rock}} / v_{\text{sef}} = 124,422$ kN·s·m/rad; and $C_{\text{tor}} = 0.127 \cdot D \cdot K_{\text{tor}} / v_{\text{sef}} = 38,299$ kN·s·m/rad. Within the finite element simulation framework, the soil–structure interaction (SSI) phenomena are rigorously incorporated through a localized substructure approximation, implemented via a specialized six-degree-of-freedom non-linear link element strategically positioned at the exact geometric centroid of the mat foundation plan (Figure 3). This advanced substructure approach utilizes frequency-independent lumped parameters—specifically synthesized spring–dashpot–mass mathematical pairs—to dynamically replicate soil flexibility, macro-boundary soil non-linearities, and radiation damping effects into the half-space. This formulation provides a computationally efficient yet highly accurate alternative to computationally prohibitive, fully coupled finite element-boundary element models, while comprehensively capturing the global and local inertial demands. To ensure rigid kinematic coupling, this central link element is interconnected to the base nodes of the four heavily braced structural columns through infinitely stiff frame elements. These rigid links successfully enforce the kinematic boundary conditions of the concrete mat foundation, transferring complex column axial forces, shears, and moments directly to the underlying soil interface without introducing artificial flexibility into the numerical model. Consequently, the superstructure interacts with the compliant geotechnical strata exclusively through this single multi-directional node, allowing the vertical, lateral, rocking, and torsional compliance of the

ground to fully participate in the overall dynamic and hysteretic response of the elevated tank configuration.

To capture the holistic three-dimensional dynamic response of the elevated water tank, all nonlinear time history analyses were executed under multi-directional seismic excitation. For each of the 11 ground motion records, three orthogonal accelerogram components were applied simultaneously to the numerical model: the two horizontal components were targeted along the principal structural axes (X and Y), while the corresponding vertical acceleration component was applied as-is at its full authentic magnitude along the vertical Z -axis. This multi-component approach ensures that the high-frequency vertical interactions, which are critical for elevated liquid-containment structures, are fully integrated into the performance evaluation of the framework.

4. Results of NLTH Analysis

A carefully curated suite of 11 highly representative far-fault earthquake accelerograms was systematically assembled to execute the comprehensive non-linear time history analyses (NLTHA) of the elevated steel liquid containment system. The selected strong-motion records strictly represent far-field tectonic settings characterized by broadband frequency content, but are intentionally selected to lack near-fault pulse-like components or velocity fling steps, thereby ensuring an unbiased assessment of long-duration dynamic effects and structural fatigue. These ground motion records, which are analytically detailed in Table 1, were meticulously gathered in strict compliance with the Eurocode 8 (EC8 [19]) seismological criteria for the selection of real accelerograms for non-linear dynamic scaling, spanning four distinct historical seismic events downloaded from the PEER database [23]: Whittier Narrows (1987), Chi-Chi (1999), San Fernando (1971), and Northridge (1994). Both orthogonal horizontal components (NS and EW) for each distinct station were fully integrated into the analyses, providing an exceptionally diverse spectrum of strong-motion durations, peak ground accelerations (PGA), and energy contents. All scaled records were subsequently applied as direct dynamic base excitations to the 3D structural models, with the non-linear integration algorithms concurrently accounting for both material plastic yielding and large-displacement geometric non-linearities (P-Delta effects). This mathematical rigor guarantees that the inelastic performance of the supporting frame, the yielding sequences of the columns, and the progressive hysteretic activation of the compact passive control mechanism are continuously captured at every discrete time step throughout the entire duration of the seismic shaking. To quantify the seismic performance enhancement, both the conventional and retrofitted tanks were subjected to NLTHA, with SSI explicitly incorporated to account for the influence of soil flexibility on structural response.

Before being used as structural input, the recorded motions were propagated through a representative geotechnical profile to obtain surface accelerograms that capture local soil amplification effects. The soil column adopted for the site response analyses comprises a uniform 30 m thick layer with a shear wave velocity of $v_s = 180$ m/s and a mass density of $\rho = 1900$ kg/m³, resting on a rigid halfspace. These properties are consistent with EC8 Class D ground and produce a fundamental site period of $T = 4 H/V_s = 4 \times 30/180 = 0.67$ s. One-dimensional equivalent-linear site response analyses were conducted for all 11 input motions, and the derived surface time histories served as seismic excitation for every structural model examined.

The term ‘soft-soil excitation’ used throughout this study refers specifically to the modified, highly amplified surface time histories that serve as the direct dynamic base inputs for the structural models. Rather than applying raw bedrock accelerograms, the 11 selected far-fault input motions were numerically propagated upward through the 30 m Class D soil column using one-dimensional equivalent-linear site response analyses.

Because the flexible soil filters and alters the frequency content of the seismic waves, the resulting surface motions feature elongated periods and amplified acceleration amplitudes that closely match the low natural frequencies of the elevated tank. This soft-soil excitation imposes severely elevated displacement demands on the structural frame, providing a rigorous environment to test the energy-dissipation efficiency of the compact lever-arm/SSD system.

To evaluate the inelastic performance and damage distribution of the supporting frame, the progressive formation of plastic hinges is monitored at each analysis time step. The color-coded performance markers in the hinge distribution plots correspond to the canonical acceptance criteria established by ASCE 41-17: IO [24] (Immediate Occupancy) represents minor structural damage where the columns retain nearly all of their pre-earthquake strength and stiffness, allowing safe immediate re-entry. LS (Life Safety) indicates significant structural damage with some margin against total collapse, where structural components have experienced significant degradation, but major structural failures are prevented. CP (Collapse Prevention) defines the limit state where the structural matrix continues to support gravity loads but retains zero residual lateral resistance, meaning any additional lateral displacement could trigger catastrophic structural collapse.

The explicit inclusion of the vertical ground motion component introduces significant high-frequency vertical oscillations that superimpose onto the gravitational load state. Under pure horizontal shaking, column axial forces are primarily governed by the global overturning moments, resulting in symmetric tension–compression structural actions. However, the simultaneous activation of the vertical component induces severe fluctuating axial force variations across all supporting columns. Despite these rapid axial force fluctuations—which momentarily alter the columns’ plastic hinge capacities—the compact lever-arm/SSD system maintains its high energy-dissipation efficiency. Because the lever-arm mechanism is kinematically activated by the relative lateral story drift (horizontal shear), its mechanical hysteretic performance remains uncoupled and structurally isolated from the vertical shaking modes. The results confirm that the frame equipped with the SSD successfully controls top joint displacements and structural damage, proving its resilience even under the severe combined demands of simultaneous horizontal and vertical seismic excitations.

4.1. Liquid Height 2.5 m

Figure 4 presents the distribution of plastic hinges in the conventional elevated steel tank with a fixed base under seismic excitation, for a liquid height of 2.5 m. Plastic hinging is concentrated at the base of the four supporting columns, with hinges reaching the Immediate Occupancy (IO) performance level according to ASCE41-17 [24], as indicated by the blue markers in the legend. The tank shell and upper frame members remain elastic throughout the analysis, confirming that inelastic demand is localized in the column bases under fixed-base conditions.

Figure 5 illustrates the plastic hinge pattern for the retrofitted tank equipped with the lever-arm/SSD system, also under fixed-base conditions and a liquid height of 2.5 m. The introduction of the passive control system significantly redistributes and reduces inelastic demand. The column bases still exhibit limited plastic activity, but the overall extent of yielding is markedly reduced compared to Figure 4. The presence of the lever-arm mechanism and the associated bracing elements absorbs a substantial portion of the seismic energy, keeping the structural response within more favorable performance levels.

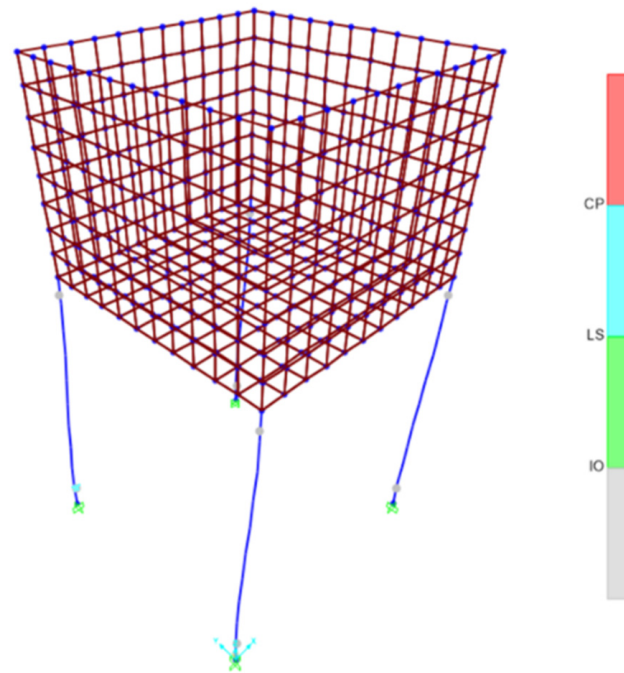


Figure 4. Formation of plastic hinges conventional water tank (fixed).

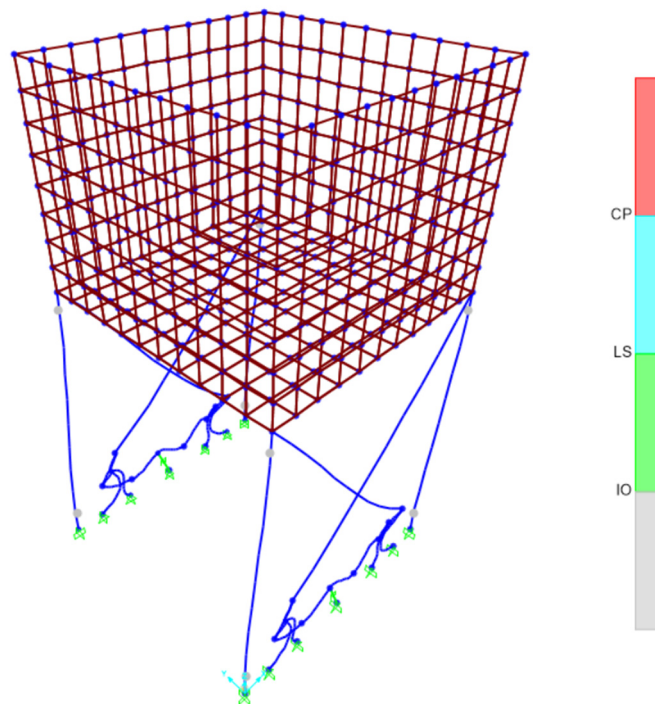


Figure 5. Formation of plastic hinges water tank with SSD (fixed).

Figure 6 shows the plastic hinge formation for the conventional tank when soil–structure interaction (SSI) is explicitly accounted for, with a liquid height of 2.5 m. Compared to the fixed-base case of Figure 4, the SSI model produces a notably reduced number of plastic hinges at the column bases, with demand remaining at the IO level according to ASCE41-17 [24]. The flexibility of the Class D soil ($V_s = 180$ m/s) lengthens the effective period of the system, which, in combination with the frequency content of the input motions, results in lower inertial forces transmitted to the structure.

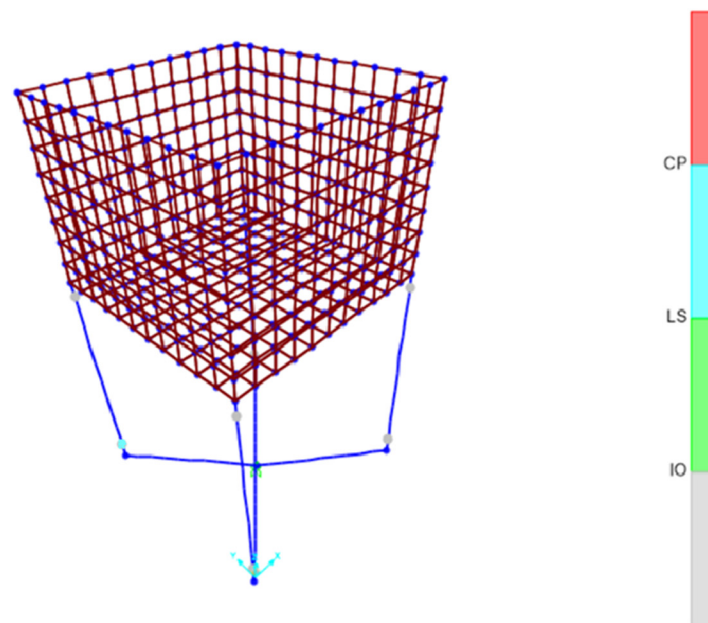


Figure 6. Formation of plastic hinges conventional water tank (SSI).

Figure 7 depicts the hinge pattern for the SSD-equipped tank, including SSI effects at a liquid height of 2.5 m. The combined effect of the passive control system and the soil flexibility yields the most favorable performance among the four configurations examined for this fill level. Plastic hinging is virtually eliminated from the column bases, with only marginal inelastic activity observable at a single location near the foundation, confirming the high efficiency of the lever-arm/SSD system under soft-soil excitation.

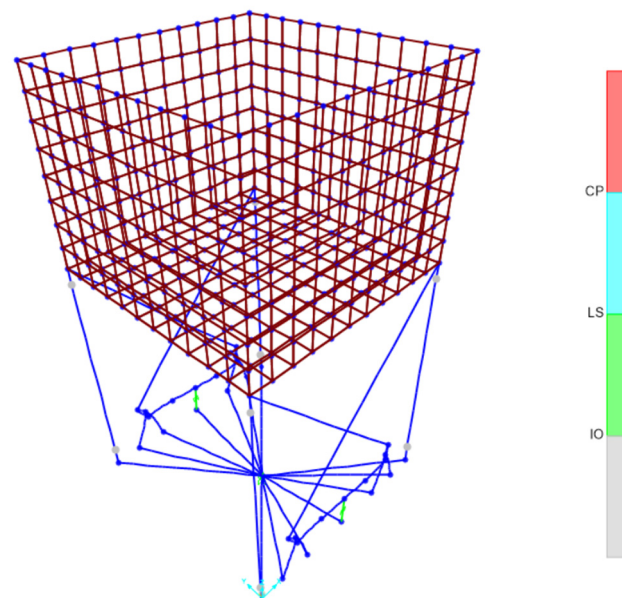


Figure 7. Formation of plastic hinges water tank with SSD (SSI).

Figure 8 presents the base shear–displacement relationships for the conventional and SSD-equipped tanks under fixed-base conditions at a liquid height of 2.5 m. Both curves exhibit a near-linear, elastic response with displacements ranging between approximately -0.03 m and $+0.03$ m and base shear values between roughly -250 kN and $+250$ kN. The two curves are nearly coincident, indicating that for this fill level and base condition, the stiffness of the retrofitted system is comparable to that of the conventional tank, and the response remains essentially in the elastic range.

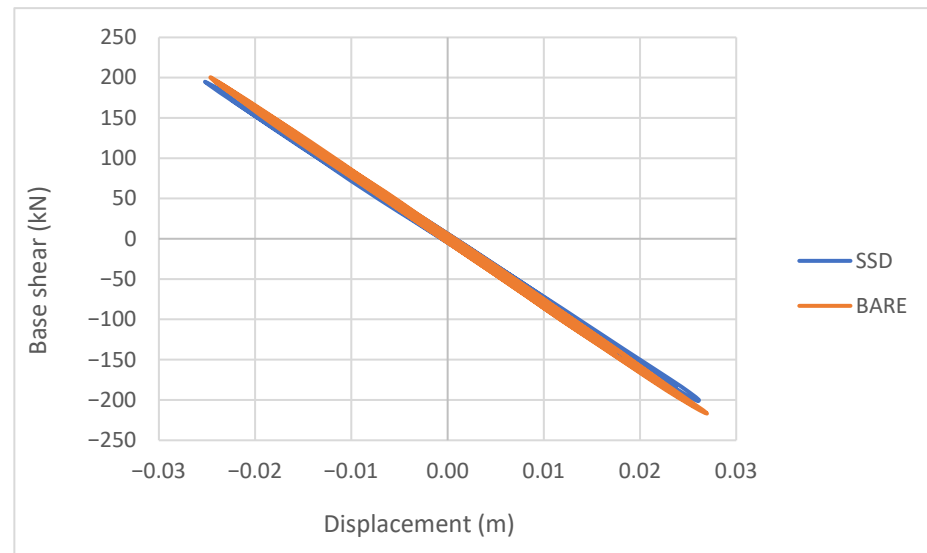


Figure 8. Base shear vs. displacement (fixed).

Figure 9 shows the hysteretic base shear–displacement loops under SSI conditions for the same fill level. In stark contrast to Figure 8, the SSI model produces substantially larger displacements (up to approximately ± 0.3 m) and base shear amplitudes reaching roughly ± 400 kN for the conventional tank. The SSD-equipped tank (blue loops) exhibits tighter, more stable hysteresis with reduced peak displacements compared to the conventional tank (orange loops), demonstrating the energy dissipation capability of the lever-arm/SSD system when soil flexibility amplifies the structural demand.

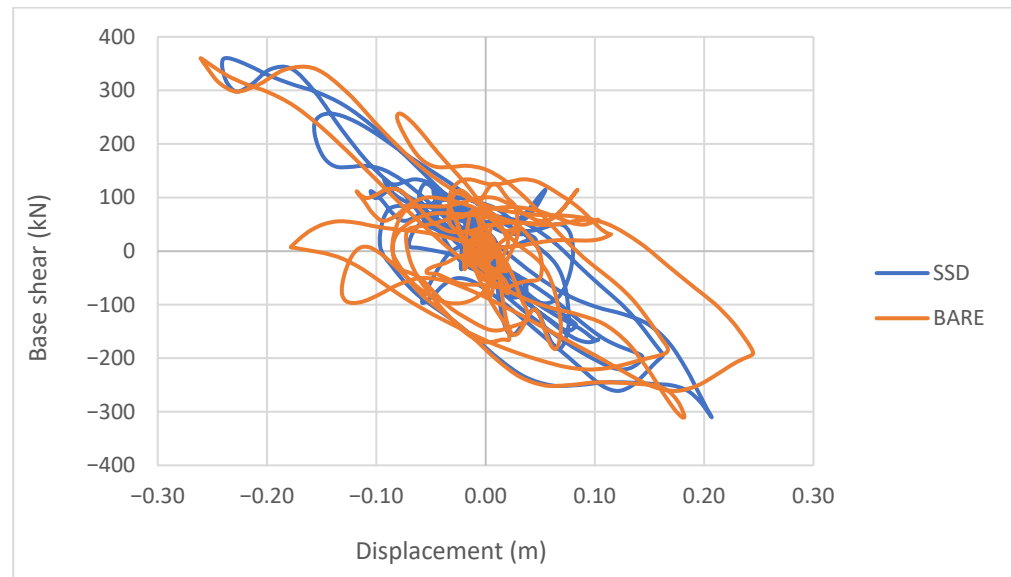


Figure 9. Base shear vs. displacement (SSI).

Figure 10 is a spider/radar chart comparing the maximum top joint displacements across the 11 earthquake records for the conventional (orange) and SSD (blue) tanks under fixed-base conditions at a liquid height of 2.5 m. The displacement values are generally small, with the outer ring at 0.2 m. For nearly all records the SSD configuration produces slightly smaller displacements than the conventional tank, though the differences are modest given the predominantly elastic response observed in Figure 8.

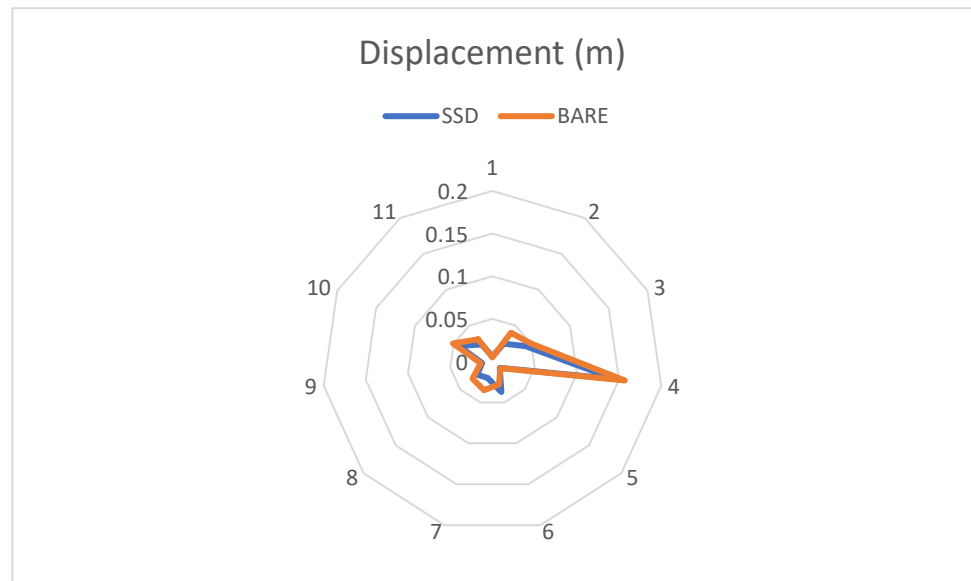


Figure 10. Top joint displacement of water tank (fixed).

Figure 11 presents the radar chart of top joint displacements for the SSI case at a liquid height of 2.5 m. The outer ring extends to 0.4 m, reflecting the significantly amplified demands due to soil flexibility. The conventional tank (orange) dominates across most of the 11 records, reaching values close to 0.4 m for the most critical ground motion (record 4), while the SSD tank (blue) consistently produces smaller peak displacements, confirming the effectiveness of the passive control system under soft-soil conditions.

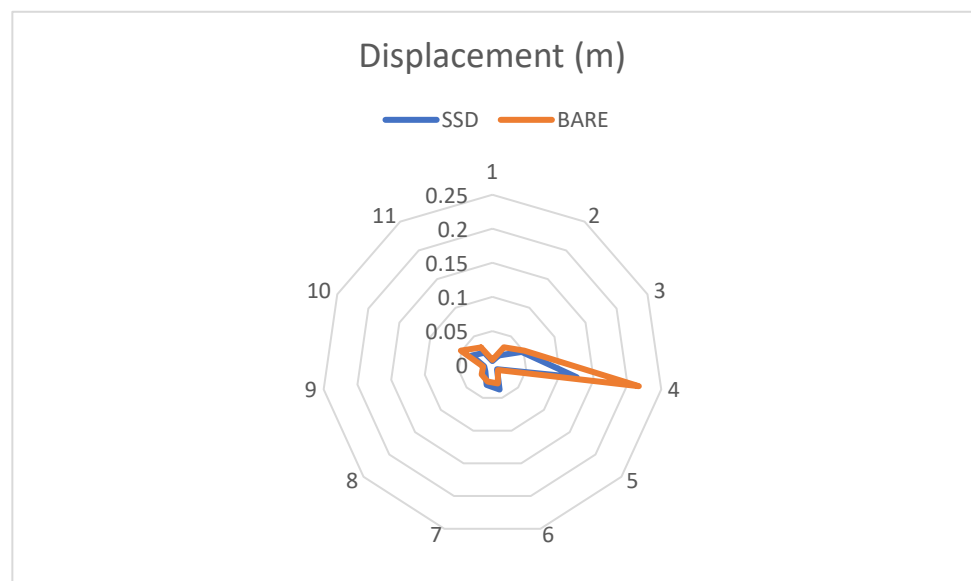


Figure 11. Top joint displacement of water tank (SSI).

Figure 12 compares the maximum base shear values across the 11 accelerograms for fixed-base conditions at a liquid height of 2.5 m. The outer ring of the radar chart corresponds to 1000 kN. The conventional tank (orange) reaches peak base shear values close to 1000 kN for record 4, whereas the SSD system (blue) consistently reduces the maximum base shear across all records, with the most pronounced reduction observed for the stronger ground motions.

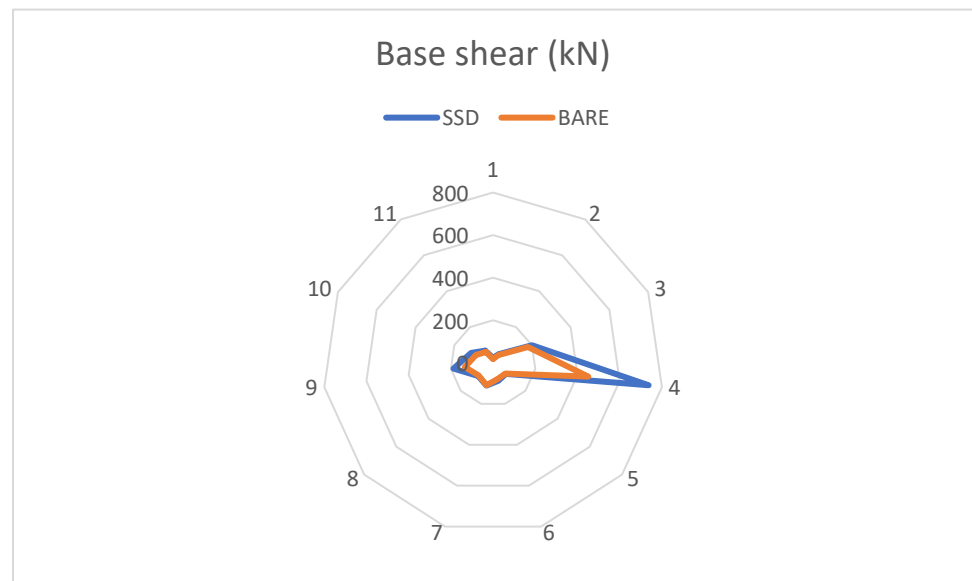


Figure 12. Base shear of water tank (fixed).

Figure 13 presents the base shear radar chart under SSI conditions at a liquid height of 2.5 m. The scale extends to 500 kN, and the conventional and SSD responses are very close for most records, with the conventional tank showing marginally higher values for records 3 and 4. The SSI effect reduces absolute base shear compared to the fixed-base case (Figure 12), consistent with the period elongation induced by the flexible soil, yet the SSD system still provides a measurable reduction.

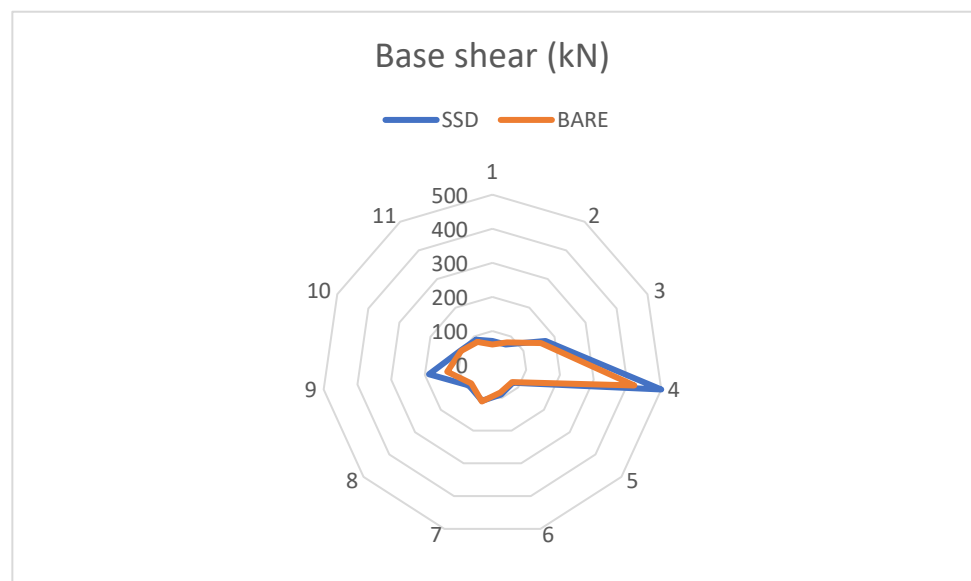


Figure 13. Base shear of water tank (SSI).

Figure 14 displays the maximum shell bending moments (in kNm/m) across the 11 records for the fixed-base tank at a liquid height of 2.5 m. The outer ring is at 20 kNm/m. The conventional tank (orange) produces noticeably higher shell moments than the SSD-equipped tank (blue) for several records, particularly records 3 and 4, where peak values approach 20 kNm/m. The lever-arm/SSD system reduces shell moments consistently across all input motions, reflecting the lower inertial demand transmitted to the tank wall.

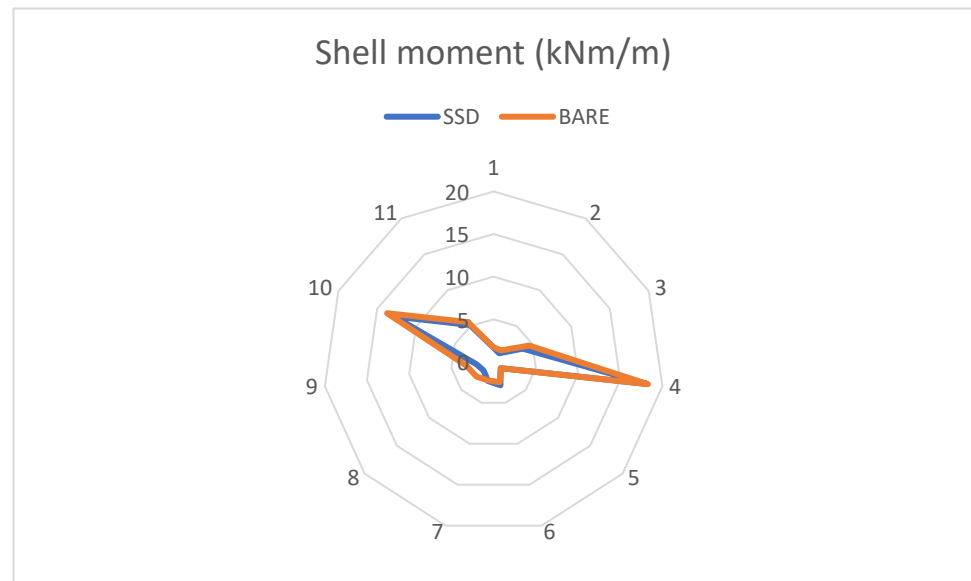


Figure 14. Shell moment of water tank (fixed).

Figure 15 shows the shell moment radar chart under SSI conditions at a liquid height of 2.5 m. The scale is again 20 kNm/m. Compared to Figure 14, the SSI condition produces a more concentrated response in records 3 and 4, with the conventional tank reaching values close to 20 kNm/m. The SSD configuration (blue) keeps shell moments markedly lower for all records, demonstrating consistent mitigation of hydrodynamic wall demands even when local soil amplification governs the seismic input.

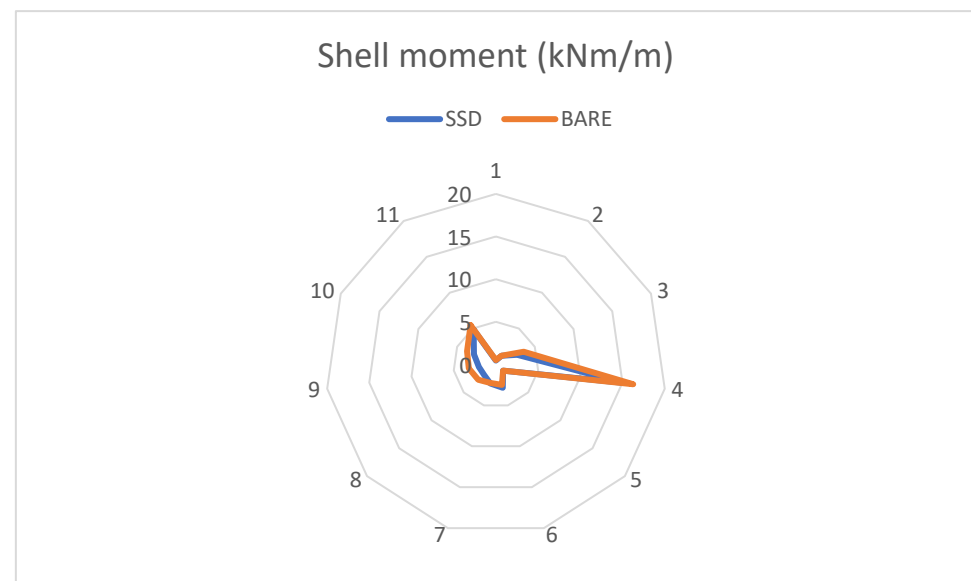


Figure 15. Shell moment of water tank (SSI).

4.2. Liquid Height 3.5 m

Figure 16 presents the plastic hinge distribution for the conventional tank with a fixed base at the higher fill level of 3.5 m. The increased liquid mass amplifies the seismic demand, and column-base hinges are again observed at the IO level, similar to the 2.5 m case. The heavier impulsive mass associated with the deeper fill generates larger inertial forces, as evidenced by the slightly wider deformed shape of the supporting frame.

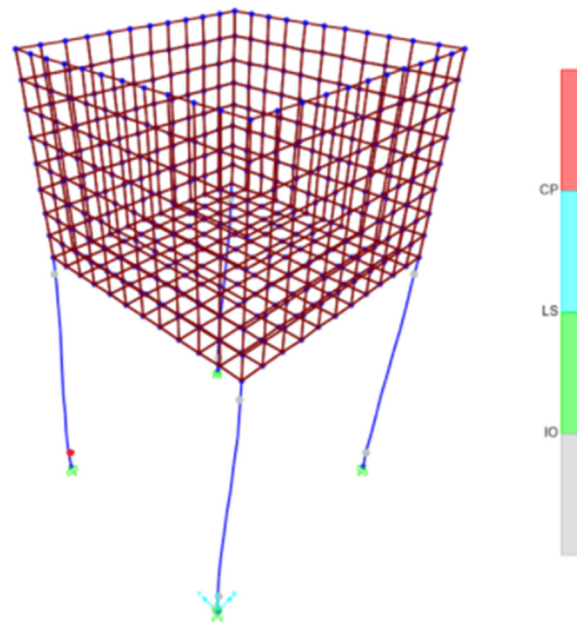


Figure 16. Formation of plastic hinges water tank.

Figure 17 illustrates the hinge pattern for the SSD-equipped tank under fixed-base conditions at a liquid height of 3.5 m. As in the 2.5 m case, the passive control system substantially limits plastic hinging. Despite the increased liquid mass, the lever-arm mechanism amplifies damper deformation and dissipates sufficient energy to maintain column demands within acceptable limits, with only minor IO-level hinges visible at the base.

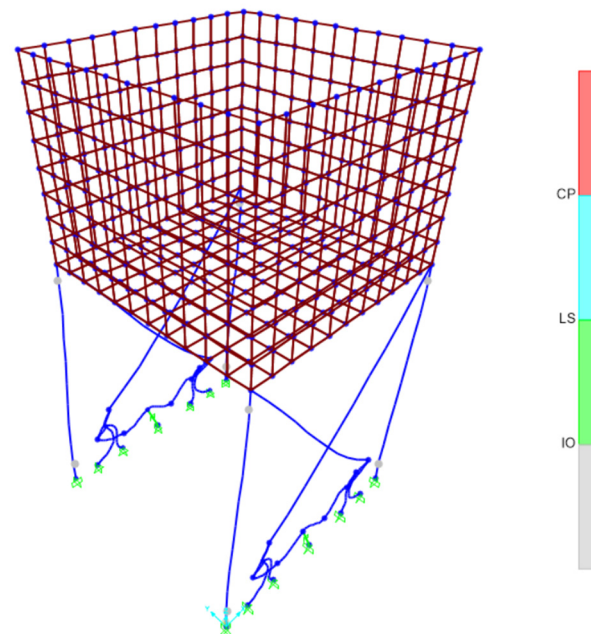


Figure 17. Formation of plastic hinges water tank with SSD.

Figure 18 shows the plastic hinge pattern for the conventional tank with SSI at a liquid height of 3.5 m. The combination of soft-soil amplification and higher liquid mass produces a more severe hinge pattern than the corresponding fixed-base case. Column-base hinges reach higher performance levels (approaching LS level [24]), and the deformed shape of the support frame indicates larger lateral displacements, consistent with the amplified displacements shown in the subsequent diagrams.

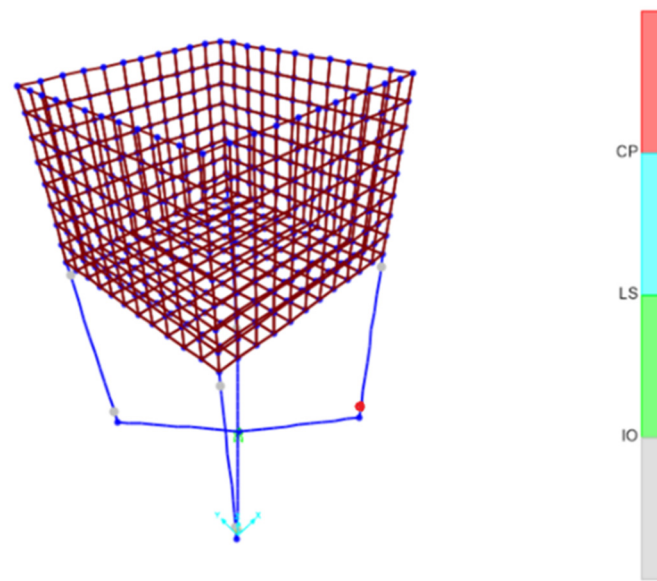


Figure 18. Formation of plastic hinges water tank (SSI).

Figure 19 presents the hinge distribution for the SSD-equipped tank under SSI conditions at a liquid height of 3.5 m. Despite the most demanding combination of soil flexibility and high fill, the lever-arm/SSD system effectively limits inelastic demand. Hinging is confined to a small number of column-base locations at the IO level [24], with no advancement to higher damage states, highlighting the robustness of the passive control strategy under the most challenging scenario examined.

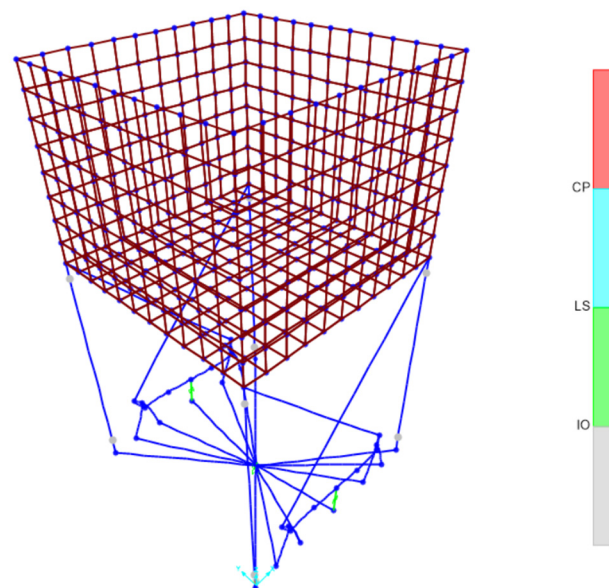


Figure 19. Formation of plastic hinges water tank with SSD (SSI).

Figure 20 presents the base shear–displacement diagrams for the fixed-base tank at a liquid height of 3.5 m. Displacements range between approximately -0.03 m and $+0.03$ m, while base shear values extend to roughly ± 580 kN, noticeably higher than the ± 250 kN observed at the 2.5 m fill level. The conventional tank (orange) exhibits a steeper slope (higher lateral stiffness), while the SSD system (blue) shows slightly reduced force demand for the same displacement, indicating early activation of the damper.

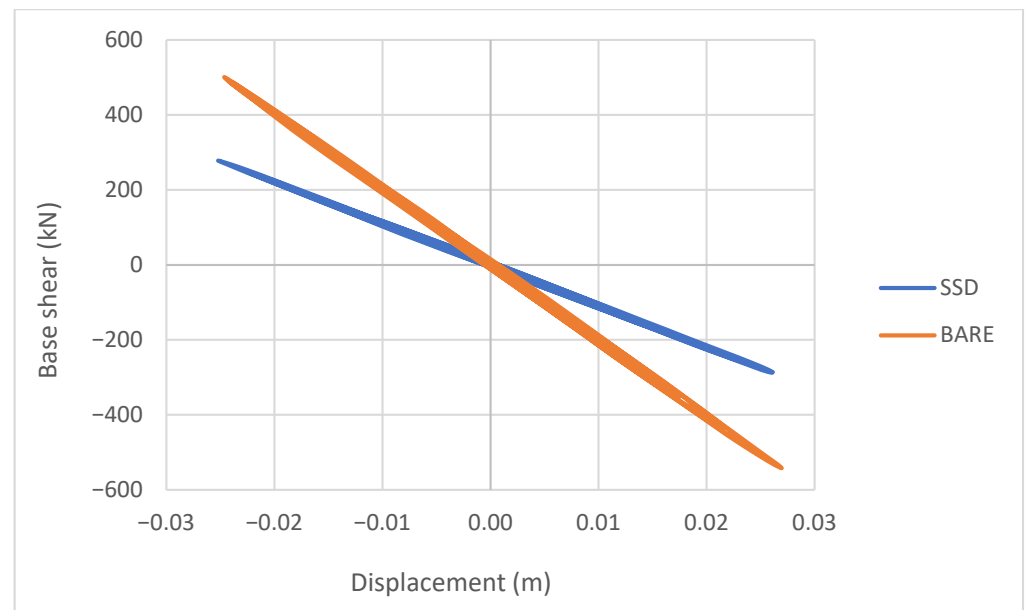


Figure 20. Base shear vs. displacement.

Figure 21 shows the hysteretic loops under SSI conditions at a liquid height of 3.5 m. Displacements reach up to approximately ± 0.3 m and base shear amplitudes approach ± 800 kN for the conventional tank (orange), representing the largest demands recorded across all four scenarios. The SSD-equipped tank (blue) produces significantly tighter hysteresis loops with peak displacements reduced to approximately ± 0.1 m, demonstrating the highest relative benefit of the lever-arm/SSD system in this most demanding configuration.

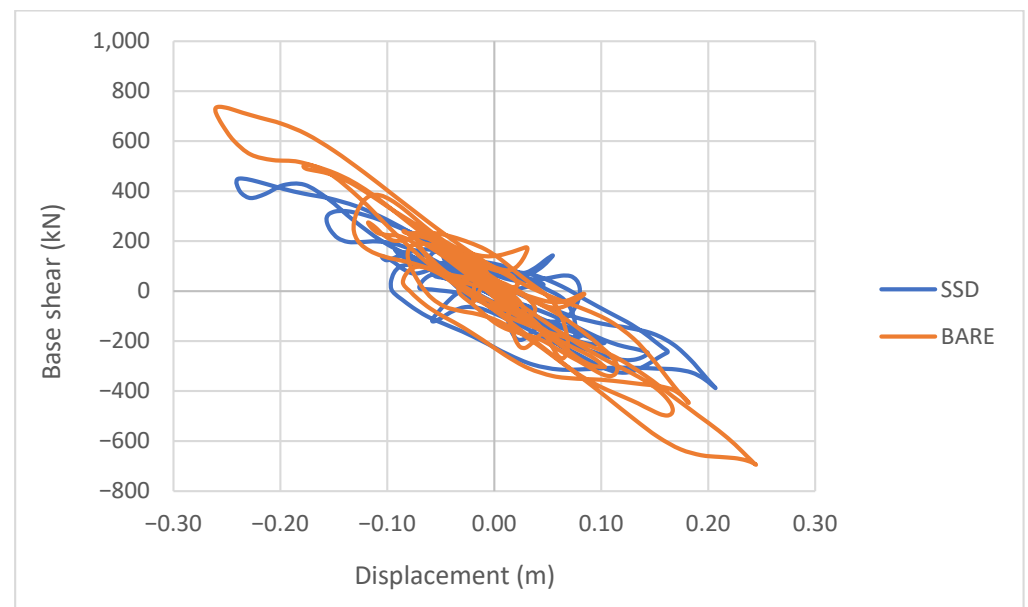


Figure 21. Base shear vs. displacement (SSI).

Figure 22 is the radar chart of top joint displacements across the 11 records for the fixed-base case at a liquid height of 3.5 m. The outer ring is set at 0.2 m. The response of both configurations is concentrated in records 3 and 4, with the conventional tank approaching 0.2 m, while the SSD system consistently reduces displacements across all records, with the most significant reduction evident for the governing records.

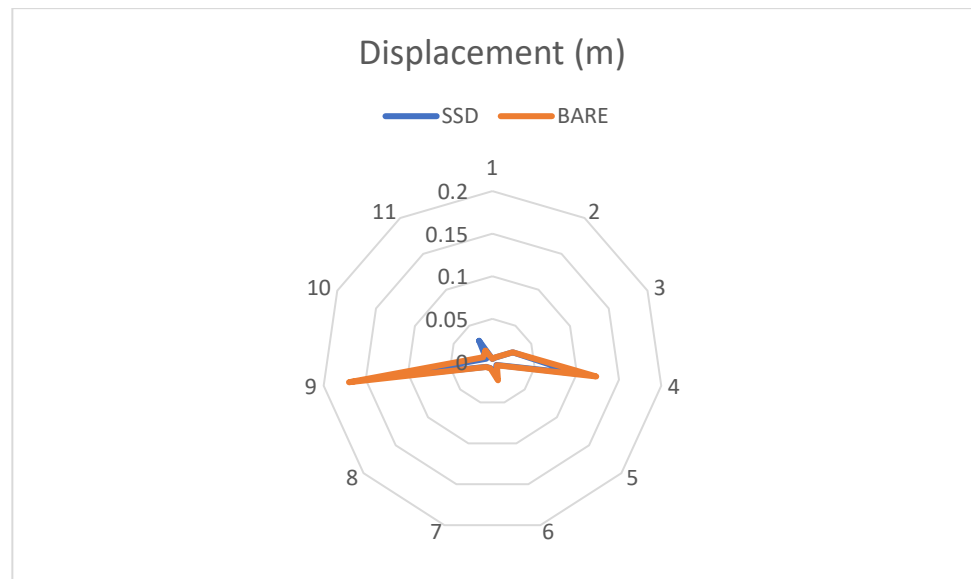


Figure 22. Top joint displacement of water tank (fixed).

Figure 23 presents the displacement radar chart under SSI conditions at a liquid height of 3.5 m, with the outer ring extending to 0.4 m. The conventional tank (orange) reaches peak values close to 0.4 m for record 4, while the SSD-equipped tank (blue) limits displacements to well below 0.2 m across all records. This substantial reduction underlines how effectively the passive control system counteracts the large deformation demands imposed by the combination of deep fill and soft soil.

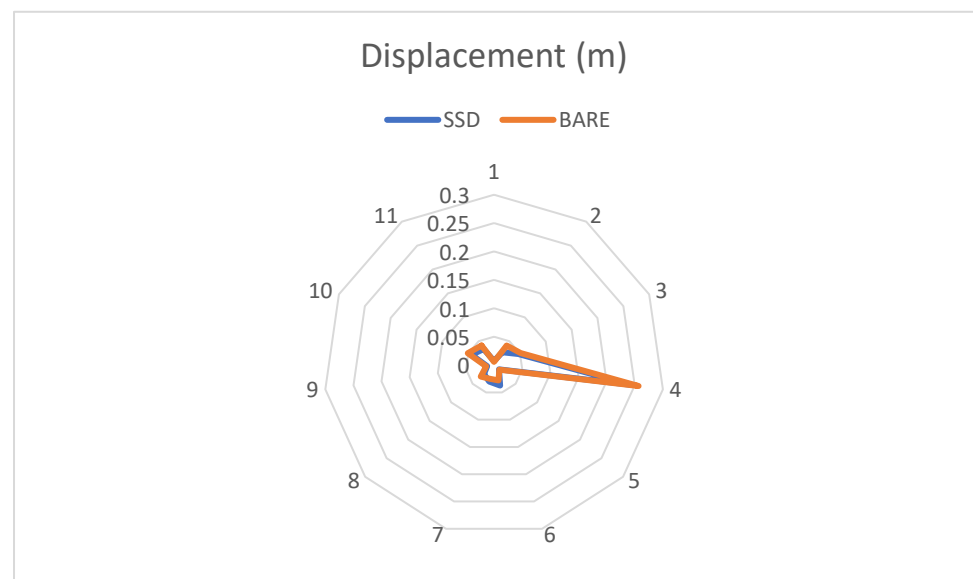


Figure 23. Top joint displacement of water tank (SSI).

Figure 24 compares maximum base shear values across the 11 records for fixed-base conditions at a liquid height of 3.5 m, with the radar scale reaching 1000 kN. The conventional tank (orange) approaches 1000 kN for record 4, while the SSD system (blue) reduces peak base shear by a considerable margin for all records. The dominance of records 3 and 4 is consistent across both fill levels, pointing to the frequency content of those ground motions being particularly detrimental for this tank configuration.

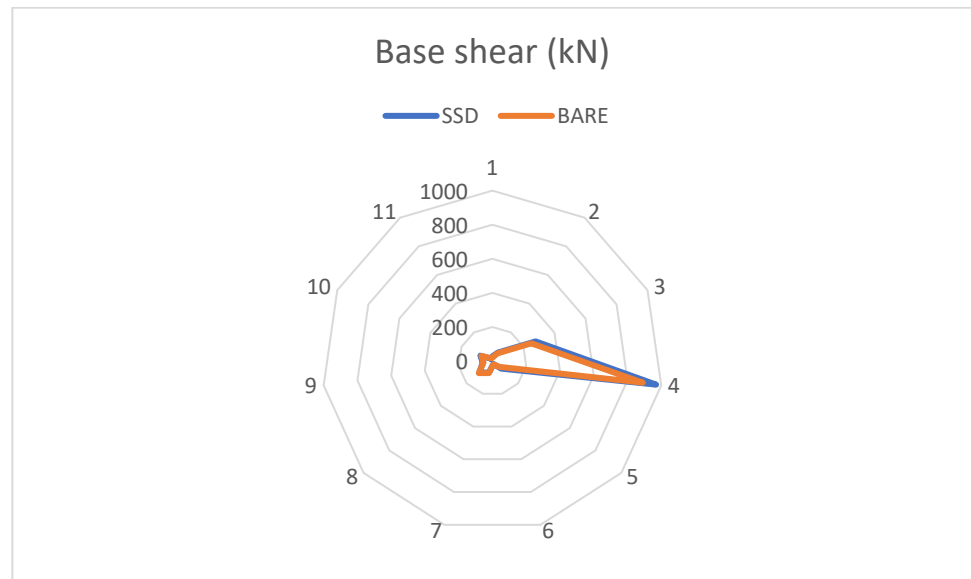


Figure 24. Base shear of water tank (fixed).

Figure 25 shows the base shear radar chart under SSI conditions at a liquid height of 3.5 m, with the scale again at 1000 kN. Compared to Figure 24, the SSI condition does not substantially reduce the peak base shear for the 3.5 m fill case, with the conventional tank still approaching 1000 kN for record 4. The SSD system (blue) produces a meaningful reduction in peak base shear, demonstrating its consistent mitigation performance regardless of boundary condition.

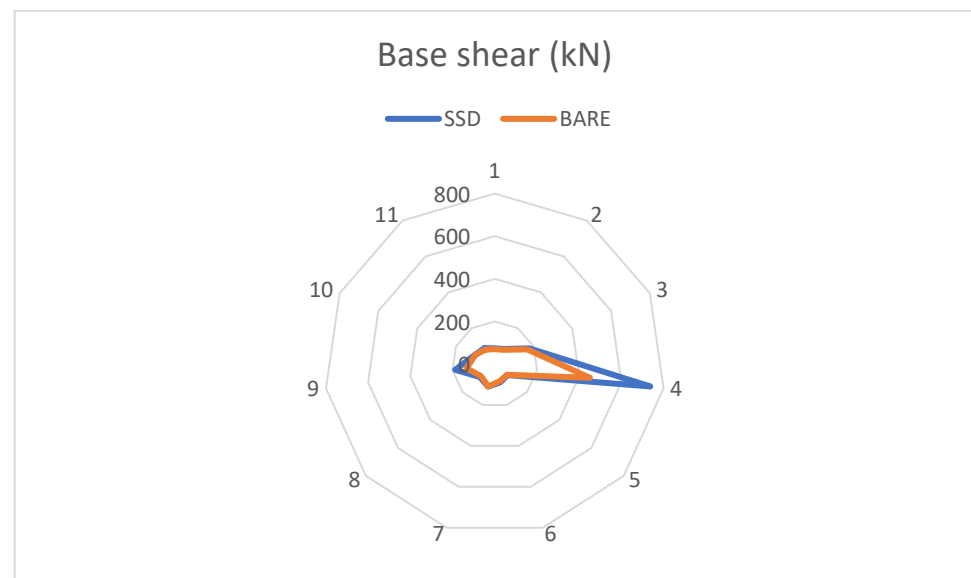


Figure 25. Base shear of water tank (SSI).

Figure 26 presents the maximum shell bending moments for the fixed-base tank at a liquid height of 3.5 m. The outer ring is set at 40 kNm/m, double that of the 2.5 m case, reflecting the substantially higher seismic-induced hydrodynamic pressures on tank walls associated with the deeper fill. The conventional tank (orange) reaches values close to 40 kNm/m for the critical records, while the SSD system (blue) consistently limits shell moments to well below this level, confirming the importance of the passive control system for tank wall integrity at higher fill depths.

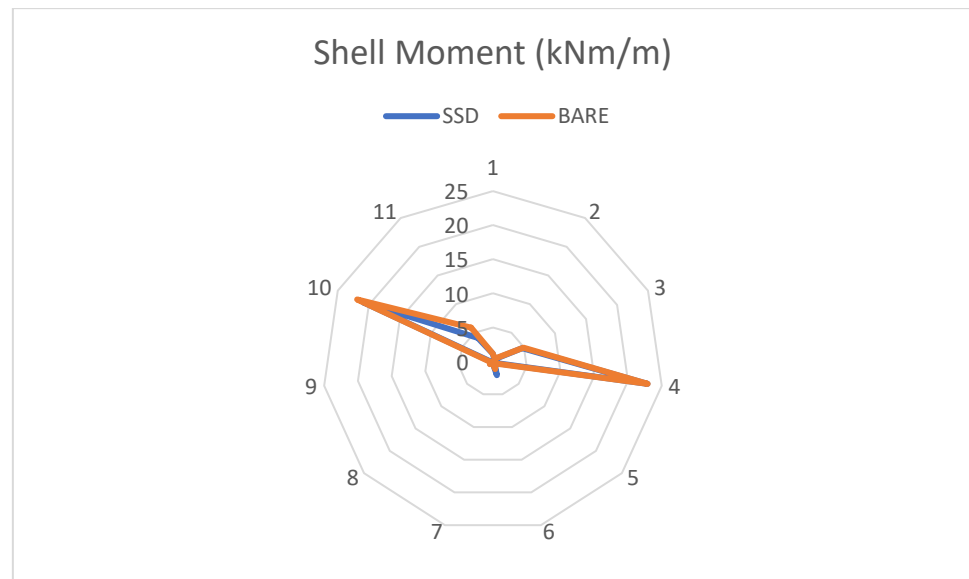


Figure 26. Shell moment of water tank (fixed).

Figure 27 presents the shell moment radar chart under SSI conditions at the 3.5 m fill level, with the outer ring at 20 kNm/m. The SSI condition moderates the absolute shell moment values compared to the fixed-base case of Figure 26, yet the conventional tank (orange) still exhibits noticeably higher moments than the SSD configuration (blue) for records 3 and 4. The SSD-equipped tank keeps shell moments consistently low across all input motions, demonstrating that the lever-arm/steel slit damper system effectively reduces hydrodynamic wall demands even when soft-soil amplification is the governing factor.

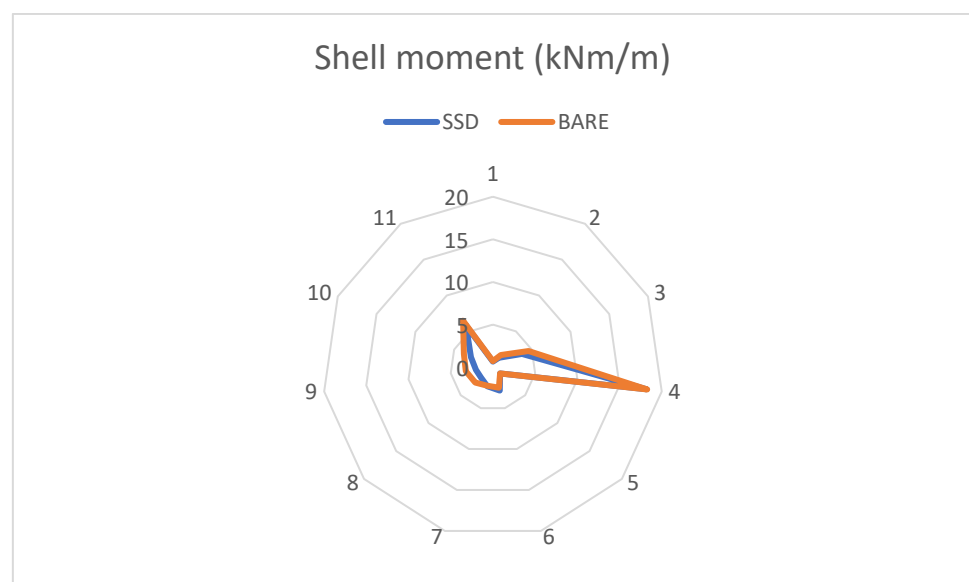


Figure 27. Shell moment of water tank (SSI).

4.3. Quantitative Evaluation of Fluid Sloshing Responses

To quantify the impact of the lever-arm/SSD system on the fluid dynamics inside the tank, the convective wave height d_{max} was calculated at each time step using the kinematic relationship derived from the Malhotra analogue:

$$d_{max} = 0.837 \cdot \left(\frac{R \cdot \omega_c^2}{g} \right) \cdot u_c(t)$$

where R is the equivalent tank radius, ω_c is the natural frequency of the convective fluid component, g is gravity, and $u_c(t)$ is the instantaneous relative displacement of the convective mass element m_c . Table 2 presents the peak sloshing wave heights recorded across the 11 ground motion scenarios. In the bare frame configuration, severe lateral structural drift amplifies the convective fluid oscillations, resulting in a maximum sloshing wave height of 0.709 m. Conversely, the installation of the lever-arm/SSD system restricts the global structural drift, which fundamentally disrupts the resonance build-up of the convective fluid mass. Across all 11 records, the damper system achieves a clear quantitative reduction in sloshing height, yielding an average mitigation rate of 26.6%. This confirms that controlling the structural frame's lateral flexibility directly stabilizes the fluid's convective behavior.

Table 2. Peak sloshing wave heights.

No	Record Name	Bare Frame d_max (m)	Frame with SSD d_max (m)	Reduction (%)
1	Whittier Narrows	0.504	0.332	34.1%
2	Whittier Narrows	0.522	0.375	28.2%
3	Chi-Chi, Taiwan	0.359	0.285	20.6%
4	Chi-Chi, Taiwan	0.344	0.232	32.6%
5	Whittier Narrows	0.599	0.419	30.1%
6	Whittier Narrows	0.374	0.245	34.5%
7	San Fernando	0.570	0.447	21.6%
8	San Fernando	0.422	0.333	21.1%
9	Northridge	0.709	0.518	26.9%
10	Northridge	0.475	0.366	22.9%
11	Chi-Chi, Taiwan	0.581	0.463	20.3%
Average	—	0.496	0.365	26.6%

5. Conclusions

The high stability and robust energy-dissipation capacity observed in the proposed lever-arm/SSD system strongly align with recent advancements in advanced metallic dampers, such as brace-type configurations with multi-phase energy dissipation capabilities. While the literature underscores the importance of precise hysteretic modeling to capture multi-stage yielding and limit structural degradation, this study demonstrates that incorporating a displacement-amplifying lever-arm successfully maximizes the efficiency of the steel slit damper. This layout ensures stable, reliable, and predictable hysteretic loops without structural pinching, making it an exceptionally effective solution for the seismic protection of liquid storage structures under multi-directional excitation [25].

This study investigated the seismic response of an elevated steel liquid storage tank retrofitted with a lever-arm/steel slit damper (SSD) passive control system, with explicit consideration of soil–structure interaction (SSI) and local site amplification effects. Non-linear time history analyses were performed for two fill levels (2.5 m and 3.5 m) and two foundation conditions (fixed-base and SSI on Class D soil), using a suite of 11 accelerograms applied as surface ground motions derived from site response analyses of representative geotechnical profiles. The following conclusions are drawn from the results.

The lever-arm/SSD system consistently reduces seismic demand across all performance indicators examined. For both fill levels and both foundation conditions, the retrofitted tank exhibits smaller top joint displacements, lower base shear forces, reduced shell bending moments, and more limited plastic hinging in the supporting columns com-

pared to the unprotected configuration. The displacement amplification provided by the lever-arm mechanism enables early and efficient activation of the slit damper, allowing a compact device to dissipate substantial seismic energy without adding significant lateral stiffness to the structure.

Soil–structure interaction on soft Class D ground ($V_s = 180$ m/s) significantly amplifies the seismic demands imposed on the unprotected tank. Under SSI conditions, top joint displacements increase by an order of magnitude relative to the fixed-base case, reaching approximately 0.3–0.4 m, and hysteretic base shear–displacement loops reveal substantially larger inelastic excursions. The period elongation induced by the flexible soil shifts the dominant response into a frequency range that is particularly unfavorable for this tank typology, making the bare structure especially vulnerable. These observations confirm that fixed-base assumptions are non-conservative for elevated tanks founded on soft deposits.

The lever-arm/SSD system demonstrates robust mitigation performance under soft-soil excitation, where its benefits are most pronounced. When SSI is included, the retrofitted tank exhibits markedly tighter hysteretic loops, with peak displacements reduced to approximately ± 0.1 m at the 3.5 m fill level compared to ± 0.3 m for the conventional tank. Plastic hinging is virtually eliminated from the column bases in the SSI–SSD configuration, even at the higher fill level, confirming that the passive control system maintains its effectiveness precisely in the conditions where the unprotected structure is most at risk.

Fill level has a significant influence on both the absolute magnitude of seismic demand and the relative benefit of the passive control system. Increasing the water height from 2.5 m to 3.5 m raises the impulsive mass, intensifies hydrodynamic wall pressures, and doubles the maximum shell bending moments in the fixed-base case. The SSD system effectively counteracts this additional demand: shell moments remain consistently lower than those of the bare tank for all records and both boundary conditions, with the outer ring of the radar charts at 40 kNm/m for the deeper fill versus 20 kNm/m at the shallower fill, indicating that the damper continues to operate efficiently even as structural demands escalate.

The record-to-record variability of the response, captured through the radar charts for the 11-accelerogram suite, reveals that ground motions 3 and 4 consistently govern the design across all configurations. This frequency-specific sensitivity underlines the importance of using a sufficiently diverse set of input motions in nonlinear time history analysis and supports the practice of site-specific ground motion selection when assessing elevated tanks at soft-soil sites. Incorporating the lever-arm/SSD framework significantly mitigates fluid–structure interaction effects, restricting peak free-surface sloshing wave heights by 26.6% reduction, thereby reducing the risk of fluid overtopping or tank roof damage.

Despite the high efficiency and clear structural advantages demonstrated by the proposed retrofit strategy, a comprehensive engineering evaluation necessitates the acknowledgement of certain practical limitations, long-term durability concerns, and financial implications. From a mechanical and structural health perspective, the integrated lever-arm/SSD system is subjected to cumulative cyclic loading during severe seismic events, which may induce fatigue and micro-structural degradation over time. Consequently, implementing a strict regime of periodic visual inspections and non-destructive testing is paramount to guarantee the long-term operational integrity of the pins, pivot joints, and the steel slit damper elements. This maintenance necessity becomes significantly more critical when these passive control systems are deployed in highly aggressive industrial complexes or coastal environments, where atmospheric corrosion can accelerate material degradation. Under such demanding conditions, the application of advanced specialized anti-corrosion treatments, including high-durability epoxy coatings or hot-dip galvanization, is deemed

mandatory to safeguard the mechanical functionality and reliability of the displacement amplification mechanism.

Furthermore, to effectively bridge the gap between numerical validation and real-world engineering implementation, a preliminary life-cycle cost-benefit analysis and operational maintenance breakdown have been synthesized and are analytically presented in Table 3. The initial capital expenditure required for manufacturing, assembling, and installing the entire structural retrofit configuration is estimated to range between approximately 15% and 20% of the tank's total structural replacement value. This investment, however, translates into a highly sustainable and economically viable payback period of 5 to 8 years, which is heavily justified by the substantial reduction in expected annual losses, seismic risk mitigation, and the prevention of catastrophic environmental or structural damage. To further optimize this protective framework, future research trajectories must encompass extensive parametric investigations focusing on the non-linear geometric and material interactions of both the lever-arm mechanism and the SSD plates. Refining these specific design components will minimize material volume while maximizing displacement amplification and total hysteretic energy dissipation capacity under near-fault seismic excitations.

Table 3. Preliminary cost-benefit, durability, and maintenance parameters of the proposed lever-arm/SSD system.

Parameter	Estimated Value/Description	Focus Area
Retrofit Cost Contribution	15–20% of the total tank replacement value	Economic Viability
Estimated Payback Period	Every 2–3 years (visual check of pins and connections)	Financial Sustainability
Routine Inspection Interval	Every 10 years or after a major seismic event	Structural Integrity
Corrosive Environment Care	Special anti-corrosion coating required (epoxy/galvanized)	Long-term Durability
Future Parametric Targets	Optimization of ratio β , plate thickness, and slit geometry	System Efficiency

Overall, the results establish the lever-arm/steel slit damper system as a viable and effective passive retrofit strategy for elevated steel liquid storage tanks. Its ability to deliver consistent reductions in base shear, column drift, sloshing height, and hydrodynamic wall pressures across varying fill levels, foundation conditions, and ground motion characteristics makes it a practical solution for the seismic upgrading of existing tanks at sites with diverse geotechnical profiles. Future work should address the optimization of the lever-arm amplification ratio for different tank geometries, the influence of near-fault and pulse-like ground motions, and the experimental validation of the proposed system at prototype scale.

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