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Comparative Study On Seismic Performance And Dynamic Analysis Of Elevated Water Tanks In Seismic Zone Iii Under Varying Soil Conditions Using Staad.Prov8i.

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Abstract
This paper presents a structural analysis and design optimization of an 80 KL Elevated Over Head Service Reservoir (OHSR) with a 14 m staging height, focusing on Soil-Structure Interaction (SSI) under seismic and hydrodynamic loads. Conventional designs assume a rigid, fixed-base condition, which often miscalculates dynamic responses. To address this, a comparative evaluation was conducted using STAAD.ProV8i to model both fixed-base and flexible-base (SSI-inclusive) parameters. Structural evaluations of hydrostatic and hydrodynamic forces were performed per IS 3370:2021 and IS 1893:2016. Serviceability limit states were strictly validated. The foundation, consisting of isolated pad footings, was optimized to a 450 mm thickness with ø 12 @ 140/150 mm c/c reinforcement. The calculated crack width (W_{cr}) was restricted to 0.02 mm, well within the 0.2 mm limit of IS 456:2000. Results show that fixed-base models underestimate structural displacement, whereas SSI-inclusive analysis provides a realistic, resilient response. The findings highlight that accounting for SSI and crack-width optimization is vital for the durability and integrity of water tanks in seismic zones.
Keywords Intze-type elevated water tank, Seismic analysis, Soil-Structure Interaction (SSI), Winkler Spring Model, STAAD.PROV8i, Seismic Zone III, Structural resilience

1. INTRODUCTION

1.1 Structural and Functional Dynamics of Elevated Service Reservoirs (ESRs)

Elevated Service Reservoirs (ESRs), colloquially designated as elevated water tanks, constitute an indispensable backbone of civic hydraulic infrastructure in both highly dense urban centers and developing rural sectors. The primary engineering objective of these structures is to store processed water at a strategically calculated potential head, thereby facilitating a fully gravity-driven distribution network. This methodology guarantees the maintenance of a consistent and reliable hydraulic gradient across extensive pipeline networks without continuous reliance on active mechanical pumping systems.

Beyond routine municipal water distribution, civil engineering frameworks categorize ESRs as vital "Lifeline Structures." This classification stems from their imperative operational necessity during catastrophic post-disaster scenarios. In the immediate aftermath of major tectonic events, major fires, or severe industrial accidents, municipal power grids frequently collapse, rendering automated pumping stations non-functional. Under these emergency conditions, elevated tanks must remain structurally sound and functionally operational to supply high-pressure water for firefighting operations, emergency medical facilities, and basic public health sanitation. Consequently, any structural failure of an ESR triggers a catastrophic compounding secondary crisis, severely compromising public safety and community resilience.

From a structural mechanics perspective, an ESR presents an inherently complex system that diverges radically from conventional multi-storey residential or commercial building frames. This structural uniqueness is characterized by a massive, highly rigid fluid containment vessel positioned atop a relatively slender, high-aspect-ratio supporting substructure known as the staging system. While conventional buildings distribute their dead and live loads relatively evenly across multiple floor diaphragms, an ESR concentrates up to 80% of its total operational mass at the absolute apex of the structure. Under static parameters, the configuration effectively resists pure vertical gravity loads and hydrostatic pressures. However, when subjected to dynamic lateral environmental forces-such as transient seismic ground motions or high-velocity aerodynamic gusts-this top-heavy mass distribution creates a massive inverted pendulum effect. This behaviour induces severe overturning moments, high base shear vulnerabilities, and extreme localized stress concentrations at the junction between the container base and the uppermost staging elements.

1.2 Geotechnical Variations and the Mechanics of Soil-Structure Interaction (SSI)

In conventional structural design paradigms, it is standard practice to simplify analytical models by assuming a perfectly rigid, fixed-base boundary condition at the foundation level. This assumption implies that the supporting geological strata possess infinite stiffness, implying that seismic ground motion is transmitted directly and uniformly from the bedrock to the superstructure without modification. While this simplified approach accelerates computational workflows and satisfies basic, conservative linear elastic code requirements, it fails to reflect realistic field-level soil mechanics, often leading to unsafe or highly uneconomical structural designs.

In reality, no geological formation is infinitely rigid. The physical interaction between the shifting foundation elements and the surrounding soil mass is a coupled, bidirectional phenomenon known as Soil-Structure Interaction (SSI). When a seismic wave propagates through the ground, it induces transient deformations in the soil matrix. The foundation of the structure, resting within or upon this moving soil, experiences translational and rotational displacements. Simultaneously, the dynamic inertial forces generated by the vibrating elevated mass of the reservoir feedback into the foundation, causing secondary deformations in the surrounding soil. This interdependent behaviour alters the overall dynamic characteristics of the system in two primary ways:

1. **Natural Period Lengthening:** The introduction of soil flexibility reduces the overall lateral stiffness of the combined soil-foundation-structure system. According to fundamental structural dynamics, a reduction in system stiffness directly results in an elongation of the fundamental natural time period (T).

2. **Radiation and Material Damping:** As the flexible soil deforms under structural feedback, it dissipates kinetic energy away from the structure through two mechanisms-material damping (internal friction within the soil grains) and radiation damping (seismic waves travelling back into the infinite soil domain).

The magnitude of SSI impacts varies dramatically based on the site's specific geotechnical profile, which is classified into Hard, Medium, and soft soil zones under Indian standard classifications:

- **Hard Soil / Rock Strata:** Possesses high shear wave velocity and structural stiffness. In these zones, the fixed-base assumption remains reasonably accurate, as soil deformations are negligible.
- **Medium Soil (Dense Sands / Stiff Clays):** Introduces noticeable compliance. The structure begins to experience minor rocking and translational motions at the base, shifting the natural period and altering the design acceleration response.
- **Soft Soil (Loose Sands / Marine Clays):** Exhibits extreme compliance and low shear modulus. In soft soil profiles, the fixed-base assumption completely breaks down. The flexible foundation permits significant lateral swaying and structural rocking. This can severely amplify the lateral drift at the 14-meter container level, causing unpredicted secondary (P-Delta) moments and risking catastrophic structural collapse if not explicitly modeled.

1.3 Fluid Dynamics: Fluid-Structure Interaction (FSI) and the Two-Mass Model

To accurately predict the seismic performance of an ESR, the designer cannot treat the stored liquid as a monolithic, dead rigid mass attached to the tank walls. During a seismic event, the lateral acceleration of the ground excites the fluid inside the containment vessel, triggering complex fluid dynamic responses known as Fluid-Structure Interaction (FSI). As the tank walls move back and forth, the fluid experiences complex hydrodynamic pressures that are mathematically split into two distinct components based on the classic Housner Two-Mass Model, as codified in IS 1893 (Part 2): 2014:

1.3.1 Impulsive Mass Component (m_i): This represents the lower portion of the stored water that moves in perfect synchronicity with the rigid structural container walls. Under lateral acceleration, this mass exerts a high-frequency, impulsive hydrodynamic pressure against the tank walls, directly contributing to the base shear and overturning moments experienced by the supporting staging columns.

1.3.2 Convective Mass Component (m_c) This represents the upper portion of the fluid that undergoes free-surface sloshing oscillations. The convective mass acts as a low-frequency fluid wave that sloshes back and forth with its own distinct natural period. It exerts a lower-frequency convective hydrodynamic pressure on the upper walls and top dome structure.

If a design ignores this distinction and treats the water as a 100% rigid mass, it will severely miscalculate the base shear and completely overlook the convective sloshing wave height. This can lead to water overtopping, damage to the top dome, or localized failure of the tank's upper cylindrical rim. Therefore, evaluating the combined effects of SSI and FSI is imperative for structural safety.

1.4 Geometry and Structural Superiority of Intze Type Tanks

For liquid containment structures with capacities ranging between 50 KL to 200 KL, the **Intze Type Tank** configuration is widely favored over conventional flat-bottom cylindrical tanks. The Intze design is a highly optimized engineering solution that utilizes structural geometry to achieve superior stress distribution.

The defining feature of an Intze tank is its unique bottom floor profile, which combines an outer conical gallery with a central spherical bottom dome. This configuration is structurally ingenious because it uses opposing structural actions to cancel out excessive stresses:

- The water resting on the outer conical wall exerts an outward lateral thrust.
- Simultaneously, the water resting on the central spherical dome exerts an inward or downward thrust.

By adjusting the inclinations and dimensions of the conical section and the spherical dome, the outward horizontal thrust from the conical wall can be balanced against the inward horizontal thrust from the dome at their common junction ring beam. This balancing act minimizes the heavy bending moments that would otherwise develop in a standard flat-bottom slab. As a result, the concrete is subjected primarily to pure hoop compression and direct tension forces, which it can resist much more efficiently. This structural efficiency allows engineers to design thinner concrete cross-sections, significantly reducing the dead weight of the container and achieving an economical, highly stable structural form.

1.5 Framework of Indian Standard (IS) Codes and Serviceability Paradigms

The structural safety, durability, and execution of the 80 KL Intze tank analyzed in this study are strictly bound by the latest revisions of the Bureau of Indian Standards (BIS) codes. Liquid-retaining structures demand a much higher level of design conservative checking than standard buildings because they must remain completely water-tight throughout their operational lifespan. Even minor structural cracks can lead to persistent fluid leakage, causing the internal steel reinforcement to corrode rapidly and ultimately leading to premature structural failure.

To address these requirements, this study implements a stringent **hybrid design philosophy**:

- **Working Stress Method (WSM) via IS 3370:2021:** Applied strictly to all water-retaining members (top dome, cylindrical wall, conical wall, and bottom dome). WSM uses conservative allowable stress limits for both concrete and steel to prevent the concrete from reaching its tensile cracking limit under daily hydrostatic pressures.
- **Limit State Method (LSM) via IS 456:2000 & IS 1893:2014:** Applied to the supporting staging columns, braces, and isolated foundations. This ensures the structural framework possesses adequate ultimate load-bearing capacity and ductile detailing to withstand extreme seismic combinations without experiencing sudden brittle failures.

A critical design validation handled in this research is the quantitative verification of the **Serviceability Limit State of Crack Width (W_{cr})**. While **IS 456:2000** permits a general crack

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width limit of 0.2 mm for standard reinforced concrete members exposed to normal environmental conditions, liquid-retaining environments are far more severe. Under severe dynamic loads, crack widths must be optimized to micro-levels to preserve structural durability. This study targets a highly refined, strict optimization threshold to ensure long-term durability in seismically active zones.

1.6 Research Objectives and Scope of the Study

The primary aim of this research is to bridge the gap between idealized code-based analytical assumptions and realistic field-level geotechnical complexities by performing a rigorous, SSI-inclusive dynamic analysis of an 80 KL Intze-type water tank under Seismic Zone III parameters. To achieve this overarching aim, the study is structured around the following specific research objectives:

1.6.1 Three-Dimensional Finite Element Modeling: To construct highly accurate 3D finite element models of the 80 KL Intze tank with a 14-meter staging height using **STAAD.PROV8i**, simulating all shell, plate, and beam elements with precise material properties.

1.6.2 Quantification of Equivalent Soil Springs: To compute the directional translational and rotational spring stiffness matrices (K_x, K_y, K_z, $K_{\theta x}$, $K_{\theta y}$, $K_{\theta z}$) for Hard, Medium, and soft soil conditions based on standard geotechnical subgrade modulus theories.

1.6.3 Comparative Response Evaluation: To conduct a comparative evaluation of the structure's seismic response under conventional fixed-base boundary conditions versus realistic flexible-base (SSI-inclusive) parameters.

1.6.4 Hydrodynamic Response Mapping: To extract and map the separate impacts of impulsive and convective hydrodynamic forces on the container walls, analyzing variations in total base shear, peak nodal displacements, and structural column drifts.

1.6.5 Foundation and Crack Width Optimization: To design and optimize an isolated pad footing system, refining its thickness and reinforcement spacing to restrict the calculated serviceability crack width (W_{cr}) well below the most stringent durability limits.

2. METHODOLOGY AND MODELING PARAMETERS

This section delineates the computational modeling, structural discretization, and geotechnical boundary conditions executed to evaluate the performance of the 80 KL Intze Elevated Overhead Service Reservoir (OHSR). The entire numerical simulation is computed within the Finite Element Analysis (FEA) platform **STAAD.PROV8i**.

2.1 Spatial Discretization and Element Characterization

To achieve an accurate mathematical idealization of the axisymmetric 80 KL Intze container resting on its 14-meter-high framework, structural components are categorized into specific line and plate finite elements:

- **Staging Column Sections:** The primary vertical load path is sustained by six circular concrete columns configured symmetrically over a cross-sectional center-to-center diameter of 4.51 m.

These columns are modeled with a uniform circular cross-sectional diameter (D) of 450 mm, maintaining a standard segment length of 3.35 m between consecutive brace levels. The bottom-most column segments anchored to the foundations are set to a length of 2.75 m to optimize base moment resistance.

- **Staging Bracing Tiers:** The horizontal braces tie the structural columns together across four distinct levels (including the ground level plinth) to drastically reduce the effective slenderness ratio of the staging framework. The bracing beams are modeled as rectangular cross-sections with a width of 300 mm and a depth of 500 mm, covering an average clear span length of 2.255 m between column junctions.
- **Container Shell Elements:** The liquid container components-comprising the top dome (100 mm thick at crown, 125 mm at springings), cylindrical side wall (185 mm thick, 1.85 m height), conical bottom slab (200 mm thick), and spherical bottom dome (200 mm thick)-are modeled using 4-noded thin shell plate elements with dense, fine meshing to capture precise local localized hoop and bending stress fields.
- **Ring Beams:** Heavy boundary ring beams are configured at critical slope transition joints to arrest horizontal thrust profiles. The top ring beam measures 185 x185mm^2, the middle ring beam stands at 300 x 300 mm^2, and the main bottom ring beam is optimized to a cross-section of 450 x 650mm^2.

Figure 2.1 Complete 3D Rendered View of Elevated Intze Tank with Staging System

2.2 Isotropic Material Modeling and Design Philosophies

To guarantee a realistic dynamic response and cross-sectional strength characterization, designated material limits are assigned in accordance with Indian Standards:

Table 2.1 Material Specification Matrix

Structural Component	Material Grade	Elastic Modulus (E)	Poisson's Ratio (ν)	Design Philosophy

Fluid Container	M30 Concrete	27,000 N/mm^2	0.17	Working Stress Method (IS 3370:2021)
Staging & Braces	M30 Concrete	21718.5 N/mm^2	0.17	Limit State Method (IS 456:2000)
Reinforcement	Fe 500 Steel	205,000 N/mm^2	0.3	Ultimate/ Serviceability Validation

2.3 Geotechnical Baseline Mapping and Winkler Spring Constants

To incorporate the critical influence of Soil-Structure Interaction (SSI), the conventional fixed-base assumption is replaced by site-specific equivalent elastic springs. The modulus of vertical subgrade reaction (K_{sy}), which represents the resistance of the soil matrix per unit settlement, is derived via the following governing equation:

$$K_s = \frac{\text{SBC} \times \text{F.O.S}}{\delta_{\text{permissible}}}$$

Where:

- SBC = Site-specific Safe Bearing Capacity (kN/m^2).
- F.O.S = Geotechnical Factor of Safety.
- delta permissible = Maximum allowable settlement threshold per IS 1904.

The horizontal subgrade stiffness values (K_{sx} and K_{sz}) are subsequently derived using elastic shear coupling relationships given below. Laboratory soil test data from three authentic locations are solved below:

$K_{sx} = K_{sz} = 0.6 \text{ x } K_{sy}$

1. Baseline Case A: Hard Soil Strata (***Baliamba Site, Odisha***)

- **Geotechnical parameters**

  SBC = 280 kN/m^2; F.O.S=3.0; delta permissible = 0.012 m.

- **Calculations**

$$K_{sy} = \frac{280.0 \times 3.0}{0.012} = 70,000 \text{ kN/m}^3$$

$$K_{sx} = K_{sz} = 0.6 \times 70,000 = 42,000 \text{ kN/m}^3$$

2. Baseline Case B: Medium Soil Strata (***Mandi Village Site, MP***)

- **Geotechnical parameters**

  SBC = 160 kN/m^2; F.O.S=2.5; delta permissible = 0.025 m.

- **Calculations:**

$$K_{sy} = \frac{160.0 \times 2.5}{0.025} = 16,000\ \mathrm{kN/m^3}$$

$$K_{sx} = K_{sz} = 0.6 \times 16,000 = 9,600\ \mathrm{kN/m^3}$$

3. Baseline Case C: Soft Soil Strata (***Palapatana Site, Odisha***)

- **Geotechnical parameters**

  SBC = 90 kN/m^2; F.O.S=2.5; delta permissible = 0.040 m.

- **Calculations:**

$$K_{sy} = \frac{90.0 \times 2.5}{0.040} = 5,625\ \mathrm{kN/m^3}$$

$$K_{sx} = K_{sz} = 0.6 \times 5,625 = 3,375\ \mathrm{kN/m^3}$$

3. PROBLEM IDENTIFICATION AND RESEARCH RATIONALE

The structural engineering domain frequently approaches the design of Elevated Overhead Service Reservoirs (OHSRs) by employing simplified mathematical abstractions. The most critical among these is the idealization of the foundation-soil boundary as an infinitely rigid, fixed-base support. While this assumption is computationally convenient for linear static calculations, it creates a significant mismatch with the actual physical behavior of elevated liquid-retaining structures during seismic events. Unlike standard multi-storey buildings that distribute their dead and live loads across multiple floor diaphragms, an Intze-type water tank exhibits a highly top-heavy mass distribution. It operates effectively as an inverted pendulum, concentrating up to 80% of its total operational and fluid mass at the absolute crown height of a slender, high-aspect-ratio 14-meter supporting staging system.

The core technical limitations identified in traditional design practices include:

- **Artificial Underestimation of Nodal Displacements:** By neglecting foundation compliance, standard models assume zero rotation and zero translation at the base. This miscalculation underpredicts the actual lateral sway and peak nodal drift experienced by the container elements at the 14-meter elevation under transient environmental actions.
- **Omission of Geotechnical Flexibility:** Real-world geological strata possess finite elastic stiffness. As the subgrade profile transitions from hard weathered rock to softer sand or marine clay matrices, the soil behaves as a flexible boundary spring. This compliance lowers the global lateral stiffness of the combined soil-foundation-structure system, causing natural period lengthening.
- **Redistribution of Base Forces and Secondary Instabilities:** Altering base flexibility modifies the boundary stiffness matrices within the solver. This redistribution introduces unpredicted force concentrations, base shear imbalances, and complex structural fatigue profiles in the lower tiers of the staging columns. It also enhances the vulnerability of the slender support columns to secondary geometric non-linearities, known as \(P-\Delta\) effects.

- **Serviceability and Durability Failure Risks:** The omission of coupled Soil-Structure Interaction (SSI) combined with complex hydrodynamic liquid pressures can lead to unpredicted micro-crack propagation within the concrete walls. For public utility structures holding drinking water, macro-scale cracking breaches watertightness, speeds up reinforcement corrosion, and drastically decreases the operational lifecycle of the lifeline infrastructure.

This study addresses these explicit engineering gaps by performing a rigorous comparative evaluation using STAAD.PROV8i. By substituting idealized rigid foundations with a site-specific Winkler Spring Model, this research maps the structural and dynamic behavioral transitions of an 80 KL Intze tank across distinct soil profiles under Zone III parameters to ensure long-term durability and structural resilience.

4.SEISMIC HAZARD CHARACTERIZATION AND QUANTITATIVE LOAD FORMULATIONS

This section presents the mathematical parameterization of design loads under combined static, hydrodynamic, and seismic actions. The model compares conventional fixed-base dynamics against flexible boundary parameters using site-specific Winkler springs for Hard (Case A), Medium (Case B), and Soft (Case C) soil zones under Seismic Zone III (Z = 0.16, I = 1.5).

4.1 Structural Load Configurations and Fluid Modeling

Static loads follow IS 875 and IS 456, while hydrodynamic actions are evaluated using Housner's two-mass analogy as per IS 1893 (Part 2): 2014. The tank's 80 KL operational volume is segregated into an impulsive mass component (m_i = 52.4 t) rigidly attached to the shell, and a convective mass component (m_c = 27.6 t) executing free-surface sloshing.

Linear soil-subgrade springs are modeled at the base elements. Soil flexibility alters the system stiffness matrix, elongating the fundamental natural period (T_i) and shifting responses across the design acceleration response spectra (S_a/g).

Table 4.1 Parametric Performance Matrix Under Seismic and Boundary Modifications

Structural Analysis Profile	**Vertical Spring Stiffness (Ksy, kN/m³)**	**Impulsive Time Period (Ti, s)**	**Spectral Coefficient (Sd/g)**	**Peak Lateral Node Displacement (mm)**	**Global Base Shear Force (Vb, kN)**
Fixed-Base Model	∞ (Rigid)	0.48	2.5	24.2	185.4
Flexible SSI: Case A (Hard)	70,000	0.52	2.5	26.85	181.2

Flexible SSI: Case B (Medium)	16,000	0.69	1.97	35.4	154.65
Flexible SSI: Case C (Soft)	5,625	0.94	1.45	48.72	128.3

4.2 Analytical Evaluation and Stress Distribution

- **Deformation Patterns:** Traditional fixed-base workflows underestimate peak lateral nodal displacements by 101.32% compared to soft soil baselines. The 48.72 mm lateral sway amplifies secondary (P-Delta) moments across the slender 14 m staging.
- **Base Reactions:** Decreasing subgrade stiffness drops the total base shear by 30.79% due to period lengthening.
- **Hydrodynamic Action:** Peak impulsive pressure reaches 14.80 kN/m² at the base of the cylindrical wall. The convective wave sloshing height peaks at 0.42 m, which dictates the required freeboard.
- **Structural Optimization:** Element stresses comply with IS 3370:2021 (Working Stress limits). The maximum hoop tension reaches 112.5 kN/m at the 185 mm thick cylindrical rim.
- **Foundation & Durability Validation:** The isolated pad footing layout was optimized to a thickness of 450 mm with ϕ12 @ 140/150 mm c/c. Using the IS 456 formula, the surface crack width (W_{cr}) was restricted to 0.02 mm, which is well below the strict durability code limit of 0.20 mm.

5. CONCLUSIONS

1. **Period Elongation:** Soil compliance elongates the fundamental impulsive time period by 95.8% (from 0.48 s to 0.94 s) as subgrade profiles transition from rigid rock to soft marine clay.
2. **Displacement Risks:** Conventional fixed-base frameworks severely underestimate structural drift. Soft-soil SSI models experience a 101.32% increase in lateral node displacements, which amplifies geometric vulnerabilities.
3. **Force Modification:** Geotechnical flexibility decreases global design base shear by 30.79%, but elevates structural hazards via secondary P-Δ moments in the slender 14 m staging framework.
4. **Fluid Wave Containment:** Housner's two-mass characterization yields a convective sloshing height of 0.42 m, establishing the minimum freeboard required to prevent sloshing water from impacting the top dome shell.
5. **Optimized Durability:** Transitioning to an SSI-inclusive paradigm allows safe sub-structural optimization down to a 450 mm thickness. This setup restricts service micro-cracks to a tight 0.02 mm, ensuring long-term watertightness and structural safety.

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