

Soil Structure Interaction of Liquid Storage Tanks in High Seismicity Regions

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ABSTRACT

Water reservoirs in New Zealand are typically designed as Importance Level (IL) 4 structures for post-disaster continuity based on Site Specific Hazard Assessment (SSHA), which for recent designs, are based on the New Zealand National Seismic Hazard Model (NSHM), 2022 (GNS, 2022).

The results of NSHM in Wellington region have shown that for short period structures such as the reservoirs, the spectral accelerations are more than twice as much as those of the (NZS 1170.5, 2004). This exceptionally high seismic force is not only posing significant challenge to structural design but also increases the complexity and cost of the service lines when the reservoir service connections are required to accommodate instantaneous and residual seismic sliding.

This paper demonstrates the beneficial effects of considering radiation damping of the tank-liquid-foundation system as recognized by NZSEE Seismic Design of Storage Tanks (NZSEE, 2009). It also provides the magnitude of sliding that can be expected in high seismic regions, such as Wellington.

In this paper, the tank-liquid-foundation system is replaced by an equivalent single-degree-of-freedom (SDOF) system following the method developed by (Veletsos and Tang, 1990). The SDOF system is characterized by lateral period and equivalent viscous damping values that were used in subsequent seismic analysis and design.

These equivalent SDOF system parameters are obtained by rigorous Frequency Domain Analysis (FDA).

Finally, the sliding potential of the tank and its extent is examined by Nonlinear Response History Analysis (NLRHA) using the SDOF parameters obtained from the FDA.

1 INTRODUCTION

Large diameter flat-bottom drinking water reservoirs are classified as seismically critical and must be designed as Importance Level 4 (IL4) structures per (NZS 1170.0, 2002) for the Ultimate Limit State (ULS) corresponding to a 2500-year return period earthquake and a 100-year design life. Concrete tanks in New Zealand follow (NZS 3106, 2009), incorporating limited damping at ULS to ensure controlled, repairable damage without significant service loss after major seismic events.

In short-period structures like concrete reservoirs, spectral accelerations from the NSHM (GNS, 2022) nearly double those from (NZS 1170.5, 2004) in the Wellington region, significantly increasing seismic design actions. This leads to hydrodynamic pressures far exceeding hydrostatic pressures, resulting in high prestressing demands and challenges in crack control due to restrained deformations.

Accurate damping estimation is critical in high spectral demand regions such as Wellington. Damping sources include inherent tank-liquid system damping, radiation damping from the tank-liquid-foundation interaction, and hysteretic damping from tank sliding (NZSEE, 2009, Veletsos and Tang 1990). This paper presents a simplified analytical method to model the tank-liquid-foundation system as an equivalent SDOF oscillator with lateral period and viscous damping suitable for design (Veletsos and Tang 1990). It also offers insights into composite damping levels and sliding potential for a recently designed Wellington tank.

2 TANK-LIQUID-FOUNDATION SYSTEM

Tanks with a liquid depth-to-radius ratio below 0.4 are classified as squat tanks, whose seismic response is dominated by the fundamental impulsive mode (Veletsos, 1984). This mode of the tank-liquid-foundation system can be modelled as a single-degree-of-freedom (SDOF) system, as shown in Figure 1, with parameters defined by Equations 1 to 6 (Veletsos and Tang, 1990).

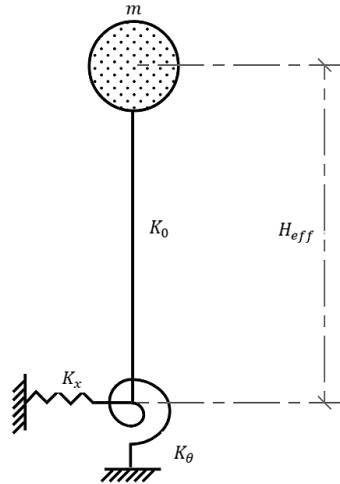


Figure 1. SDOF representation of tank accounting for foundation impedance

$$K_0 = \overline{K}_0 \left(1 + i2 \left(\xi_{vis} \frac{\overline{\omega}}{\omega_0} + \xi_{hys} \right) \right) \quad (1)$$

$$K_x = \overline{K}_x (\alpha_x + i2\beta_x) \frac{m}{m_{roof} + m_{wall} + m} \quad (2)$$

$$K_\theta = \overline{K}_\theta (\alpha_\theta + i2\beta_\theta) \frac{mh^2}{m_{roof}H^2 + m_{wall}\frac{H^2}{4} + mh^2} \quad (3)$$

$$\overline{K}_0 = \frac{4\pi^2}{T_0^2} m \quad (4)$$

$$\overline{K_x} = \frac{8G_s R}{2-\nu} \quad (5)$$

$$\overline{K_\theta} = \frac{8G_s R^3}{3(1-\nu)} \quad (6)$$

where K_θ is the complex stiffness of the tank-liquid system in the first impulsive mode; K_x is the complex translational foundation stiffness; K_θ is the complex rotational foundation stiffness; ξ_{vis} is the inherent viscous damping, and ξ_{hys} is the equivalent viscous damping corresponding to displacement ductility, $\mu = 1.25$ at ULS, of the tank-liquid system; $\overline{\omega}$ is the resonant frequency of the tank-liquid-foundation system; ω_0 is the frequency of the tank-liquid system only; α_x and α_θ are dimensionless factors that convert the static translational and rotational foundation stiffnesses, respectively, to dynamic stiffnesses; β_x and β_θ are translational and rotational complex damping coefficients, respectively, representing radiation damping (Gazetas, 1983). The complex stiffnesses are frequency-dependent and vary as a function of $a_0 = \overline{\omega} R V_s$ where V_s , G_s and ν are shear wave velocity, low strain shear modulus, and Poisson's ratio of the soil, respectively.

The coupled period of the tank-liquid-foundation system $\overline{T}$ can be determined as per equation 7 (Veletsos, 1984)

$$\overline{T}^2 = T_x^2 + T_\theta^2 + T_0^2 \quad (7)$$

The components of eq. (7) are defined in equations 8 to 14 (Veletsos, 1984).

$$T_0^2 = 4\pi^2 \frac{m_{roof}}{K_{wall}} + T_{liq}^2 \quad (8)$$

$$T_x^2 = \frac{4\pi^2}{\omega_x^2} \quad (9)$$

$$T_\theta^2 = \frac{4\pi^2}{\omega_\theta^2} \quad (10)$$

$$\frac{1}{K_{wall}} = \frac{1}{\frac{GA_{shear}}{H}} + \frac{1}{\frac{3EI}{H^3}} \quad (11)$$

$$T_{liq} = 2\pi \frac{H}{C} \sqrt{\frac{\rho_w}{E}} \sqrt{\frac{t_{0.01R}}{t_{wall}}} \quad (12)$$

$$\omega_x^2 = \frac{m_{roof} + m_{wall} + m}{\overline{K_x} \alpha_x} \quad (13)$$

$$\omega_\theta^2 = \frac{m_{roof} H^2 + m_{wall} \frac{H^2}{4} + m h^2}{\overline{K_\theta} \alpha_\theta} \quad (14)$$

where m_{roof} , m_{wall} and m are the mass of the roof, the effective mass of the wall, and the impulsive mass of the liquid, respectively; H is the liquid depth; h is the overturning modal height of the fundamental impulsive mode; C is the dimensionless factor for determining the fundamental impulsive mode as per the Table 1 (Veletsos and Shivakumar, 1997); ρ_w is the wall mass density; $t_{0.01R}$ is the assumed wall thickness of 0.01 times the tank radius R ; and t_{wall} is the uniform wall thickness.

Table 1. Dimensionless factor C for the fundamental impulsive mode adopted from [12]

H/R	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0
C	0.1222	0.1293	0.1366	0.1431	0.1488	0.1537	0.1578	0.1612

Stiffness of the wall K_{wall} can be determined as $A_{shear} = \pi R t_{wall}$ and $A_{shear} = \pi R t_{wall}$ where G and E are the shear and Young's modulus of the tank wall material, respectively.

3 FREQUENCY DOMAIN ANALYSIS

Frequency domain analysis of the mechanical model subjected to a unit lateral ground acceleration shown in Figure 2 was carried out in SAP 2000 (CSI, 2025) to determine the fundamental frequency and damping ratio of the equivalent SDOF oscillator.

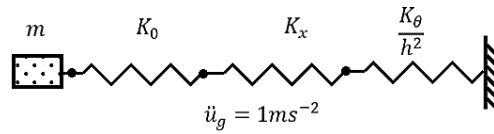


Figure 2: Mechanical model with impedances for the frequency domain analysis (Veletsos, 1984)

The damping ratio is calculated for a concrete tank with an 18.2 m radius, 6 m water height, and 250 mm shell thickness. The Vs is 930 m/s, the roof weight equals a 335 mm solid concrete slab, and the tank wall's Young's modulus is based on 40 MPa characteristic compressive strength concrete.

The frequency domain analysis results where the equivalent damping can be calculated as $1/2\rho$ where ρ is the amplification is shown in the Figure 3. It can be observed that the damping increases from 5.1% ($\rho = 9.76$) with the foundation radiation damping and the inherent damping of 2% of the tank-liquid system (Veletsos, 1984) to 11% ($\rho = 4.5$) with the foundation radiation damping, the inherent damping of 5%, and the hysteretic damping of 3.5% of the tank-liquid system at the ULS loading (NZSEE, 2009). The fundamental period of the tank-liquid-foundation system is determined to be 0.09 sec. In this way, the frequency domain analysis can be used to determine the period and the equivalent viscous damping of the replacement SDOF oscillator (Veletsos and Tang, 1990).

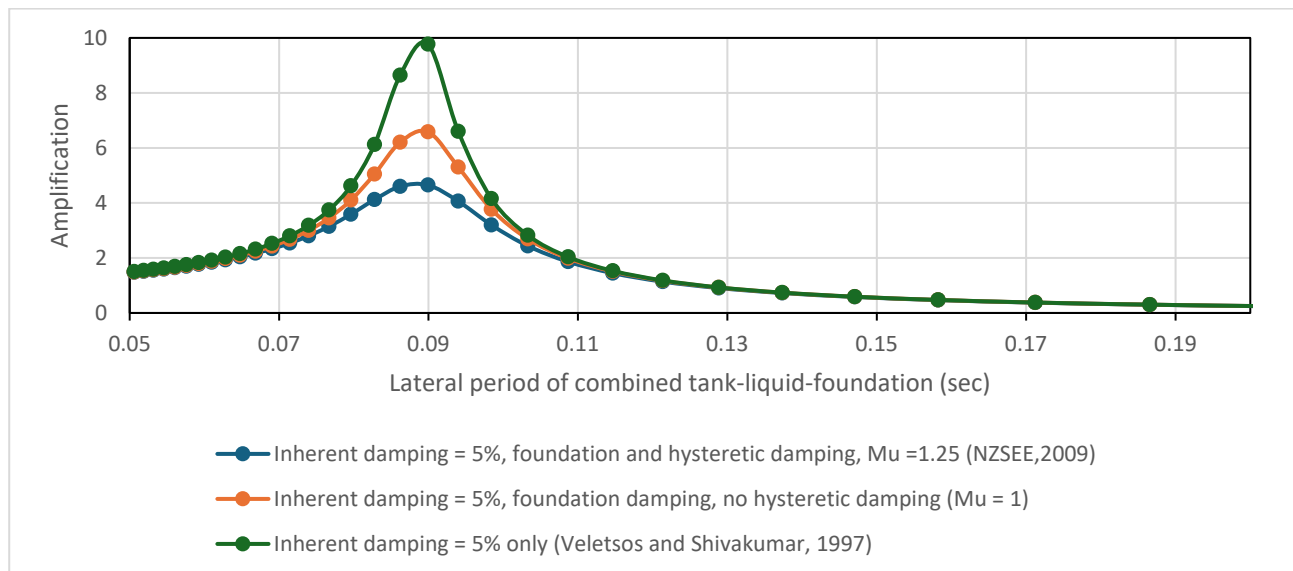


Figure 3: Frequency of the equivalent SDOF system for a squat tank without sliding

4 SLIDING POTENTIAL

Sliding of tank is accompanied by frictional damping and its magnitude is therefore bounded. To determine the magnitude of sliding, the analytical model shown in Figure 4 is developed where the equivalent stiffness, K_{eq} , and the equivalent damping, β_{eq} , can be determined either by using equations 15 to 17 or directly from the frequency domain analysis. The sliding is modelled by a rigid-plastic spring annotated as K_{slide} in the figure. The yield strength of spring is given by $F_s = \mu W$ where $\mu = 0.5$ is the coefficient of sliding friction between the tank bottom and the subgrade, and W is the weight of the tank and the liquid.

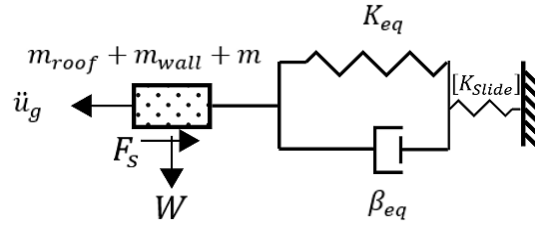


Figure 4: Mechanical model with optional sliding for response history analysis

$$\frac{1}{\bar{K}_{eq}} = \frac{1}{\bar{K}_x \alpha_x} + \frac{1}{\frac{\bar{K}_\theta \alpha_\theta}{h^2}} + \frac{1}{\bar{K}_0} \quad (15)$$

$$\beta_{eq} = \frac{\bar{K}_{eq} \beta_x}{\bar{K}_x \alpha_x^2} + \frac{\bar{K}_{eq} \beta_\theta}{\frac{\bar{K}_\theta \alpha_\theta^2}{h^2}} + \frac{\bar{K}_{eq} (\xi_{vis} \frac{\bar{\omega}}{\omega_0} + \xi_{hys})}{\bar{K}_0} \quad (16)$$

$$K_{eq} = \frac{m_{roof} + m_{wall} + m}{m} \bar{K}_{eq} \quad (17)$$

4.1 Resonant Response Analysis

Resonant response analyses of the equivalent SDOF oscillator with the period of 0.09 sec and the equivalent viscous damping of 11% (see Figure 3) were carried out with and without sliding as nonlinear response history analysis using SAP2000 (Chopra 2006, CSI, 2025). As can be seen in Figure 5, with the tank sliding, the equivalent viscous damping increased to 25% corresponding to $\rho = 2$.

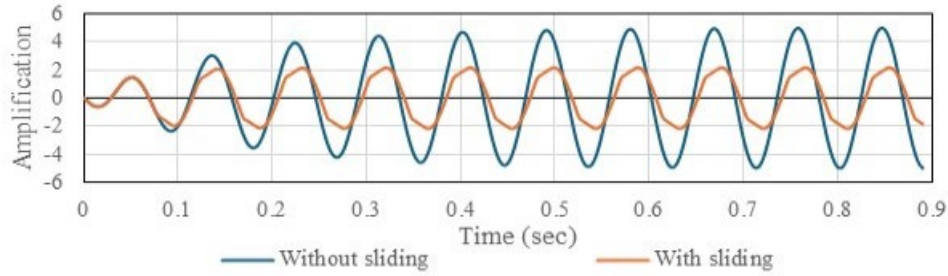


Figure 5: Dynamic Amplification comparison of oscillators with and without sliding

The yield displacement of the slider, $\delta_y = F_s/K_{eq}$ is estimated to be 3.5 mm. As shown in

Figure 6, the maximum sliding is 2.5 times of $\delta_y = 9$ mm. Reducing the system stiffness by 50% as a sensitivity check, will still result in small sliding magnitude of any practical consideration.

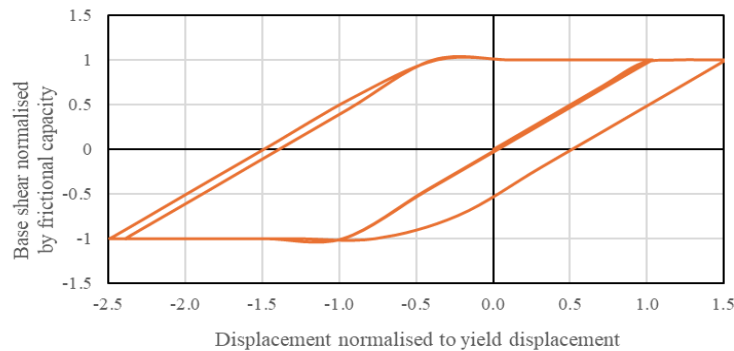


Figure 6: Response of K_{slide} in resonant response analysis

4.2 Response History Analysis

The sliding magnitude from resonant response analysis was verified by response history analysis using five shallow crustal and three subduction earthquakes (Table 2). Ground motions with magnitudes MW 6.5 and 8.5, V_{s30} between 600 m/s and 800 m/s, and site distances under 15 km were selected for shallow crustal records. Amplitude scaling was performed for periods between 0.02 and 0.2 sec, with results shown in Figure 7 and Figure 8. Due to high ULS spectral ordinates from NSHM (GNS, 2022) and the short period range, finding suitable ground motions per (ASCE, 2022) scaling was challenging. Both scaled ground motion components were applied concurrently in the SAP2000 model (CSI, 2025).

Table 2: Selected shallow crustal and subduction ground motions

(SUB)RSN	143	1507	1633	4876	8165	4016856	6001155	6001802
Record Name	Tabas, Iran	Chi Chi, Taiwan	Manjil, Iran	Chuetsu, Japan	Duzce, Turkey	Fukushima, Japan	Chile 2005	Chile 2010
Mechanism	Reverse	Reverse Oblique	Strike Slip	Reverse	Strike Slip	Subduction	Subduction	Subduction

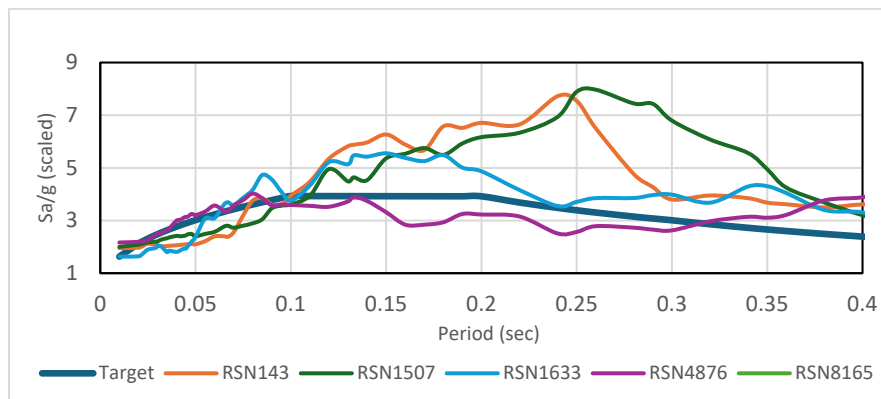


Figure 7: Scaled shallow crustal earthquakes vs NSHM (GNS, 2022)

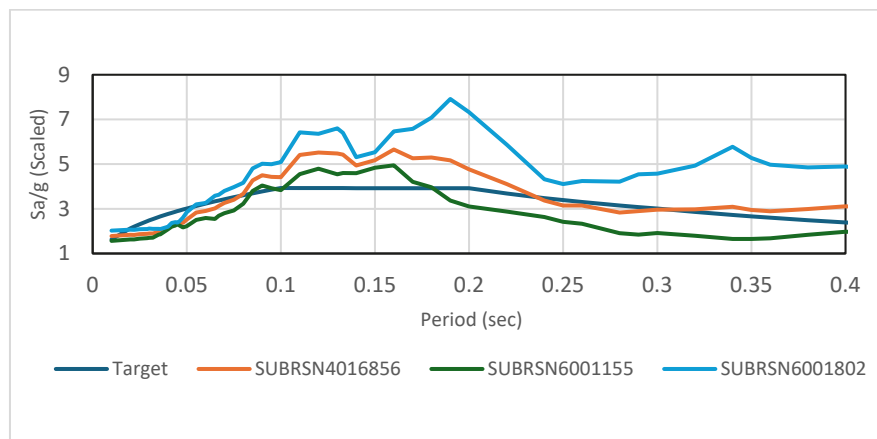


Figure 8: Scaled subduction earthquakes vs NSHM (GNS, 2022)

Figure 9 and Figure 10 show the sliding response under the bidirectional components of the ground motions applied concurrently. The displacements shown are the magnitude of the displacement in two orthogonal directions. It can be observed that the displacement response is negligible for practical design considerations.

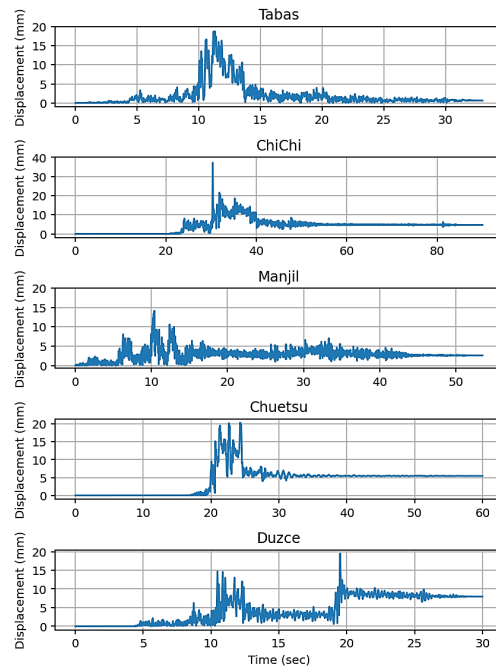


Figure 9. SDOF system sliding displacement subjected to scaled shallow crustal earthquakes

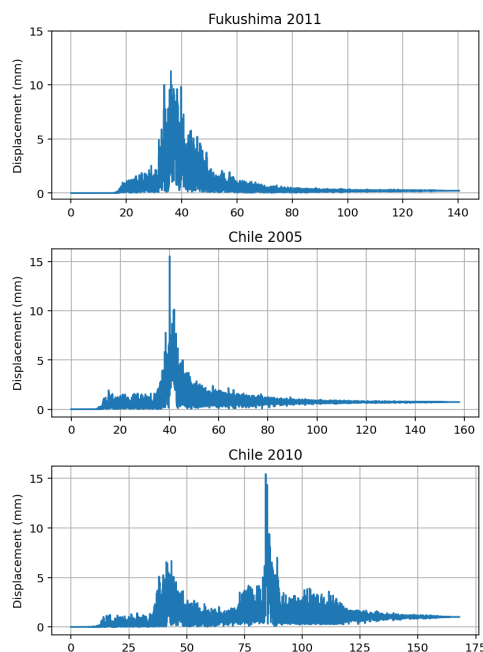


Figure 10. SDOF system sliding displacement subjected to subduction earthquakes

5 CONCLUSION

The conclusions are as follows.

- The paper presents a simplified analytical method to replace the tank-liquid-foundation system with an equivalent SDOF model based on (Veletsos and Tang, 1990), achieving 11% overall damping as in a recent design without sliding consideration.
- Incorporating Tank sliding according to (Veletsos and Tang, 1990) recommended procedure, show that sliding is accompanied by significant hysteretic damping which results in limiting the sliding.

The results of analyses confirm that for concrete squat tanks, sliding remains under 75 mm with up to 25% overall damping, even with a safety factor of 2.

- Regardless of sliding magnitude, piping attachment design should allow minimum flexibility; 100 mm displacement flexibility from standard attachments is sufficient.
- To include sliding damping, time history analyses are recommended with limiting overall damping to 20% (ASCE, 2022).

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