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Highlights

- Rigid-water models overstate how much filling raises wind uplift at tower bases
- Convective water decouples; the tower frequency falls far less than assumed
- With sloshing modelled, a filled tank governed uplift in 11 of 300 towers, not 74
- Foundation compliance raised empty-tank uplift by 15% at spring ratio $r = 1$
- Unequal support stiffness of $\pm 30\%$ raised 95th-percentile corner uplift by 5–18%

Abstract

Uplift anchorage for elevated steel tank towers is normally designed for the drained tank, the condition with the least stabilising load. However, when dynamic amplification is included through the EN 1991-1-4 structural factor, filling lowers the natural frequency and can increase the resonant response faster than it increases the stabilising load. Previous single-degree-of-freedom assessments treated the stored water as a rigid mass and found that a partly or fully filled tank governed uplift in a quarter of towers at a basic wind velocity of 40 m/s. This paper replaces the rigid-water idealization with a two-mass model with impulsive and convective water masses from Housner's formulation, evaluates the structural factor by a spectral generalisation of the EN 1991-1-4 Annex B procedure, and adds foundation compliance and support-stiffness scatter through a three-dimensional frame model. The spectral procedure reproduced Annex B within 0.5% in the single-degree-of-freedom limit. For a 15 m reference tower with a 100 m³ tank and a structural logarithmic decrement of 0.02, the rigid-water model predicted a 19.6% rise in uplift from empty to full, whereas the two-mass model predicted 0.5%, because a third of the water sloshes at about 0.4 Hz and acts as a tuned absorber rather than as tower mass. In 300 random towers, the fill level governed

in 11 cases (at most 7.3%) instead of 74 (at most 34.6%). Foundation compliance was more influential: vertical support springs equal to a stanchion's axial stiffness raised empty-tank uplift by 15%, and a $\pm 30\%$ scatter in support stiffness raised the 95th-percentile corner uplift by 5% to 18%.

Keywords: elevated tank; along-wind response; sloshing; structural factor; foundation compliance; uplift; EN 1991-1-4

1. Introduction

An elevated water tank supported on a steel lattice tower carries its largest overturning demand relative to its stabilising load when the tank is drained. Steel and glass-reinforced plastic panel tanks weigh a small fraction of the water they store, so the drained condition is conventionally taken as governing the tension at the windward stanchion bases, which must be anchored into concrete pedestals. The drained assumption is quasi-static. When dynamic amplification is included, filling the tank lowers the fundamental frequency and increases the resonant part of the along-wind response. If the stored water is treated as a mass rigidly attached to the tower, the resonant growth can outpace the growth of the stabilising load in lightly damped towers, so that a partly or fully filled tank governs uplift.

That conclusion depends on how the water is modelled. Liquid in a tank does not move as a rigid body: part of it moves with the walls (the impulsive mass), and part sloshes at a much lower frequency (the convective mass) [1]. Seismic assessments of elevated tanks routinely distinguish empty, half-full and full conditions and treat the fluid-structure interaction explicitly [2,3], but wind assessments have tended to lump the water with the tower. A second quantity that the usual assessment omits is the stiffness of the foundation. Where stanchions stand on separate pad footings or piles, the vertical compliance of the supports lowers the lateral stiffness of the tower, and unequal compliance redistributes the overturning among the stanchions.

This paper quantifies both effects on the design uplift of elevated steel tank towers. It develops a two-mass (impulsive and convective) along-wind model, evaluates the EN 1991-1-4 structural factor [4] by a spectral generalisation of its Annex B procedure, and verifies the procedure against Annex B in the single-degree-of-freedom limit. It then examines a reference tower, a random sample of 300 towers at a basic wind velocity of 40 m/s, the effect of uniform foundation compliance, and the effect of unequal support stiffness computed with a three-dimensional frame model.

2. Background

Wind-tunnel and aeroelastic studies of lattice towers have shown that the dynamic part of the response is not negligible. Azzi et al. [5] found a resonant contribution of up to 18% of the peak response of a self-supported lattice tower, and Jeddi et al. [6] measured gust response factors up to 13% above recommended values. Skew-wind load factors and drag coefficients of lattice towers have also been found to exceed code values [7,8], and directional extreme wind speeds can differ from estimates that ignore direction [9]. For tanks, Chiang and Guzey [10] found no resonance in empty ground-supported steel tanks, whose shell frequencies are high, and Hu et al. [11] showed that the empty condition governs bottom-plate uplift of ground-supported tanks. Elevated tanks differ in that the supporting tower has a low fundamental frequency, typically a few hertz, which falls further as the tank fills.

The dynamic behaviour of liquid in rigid tanks is commonly represented by Housner's mechanical analogue [1], in which an impulsive mass moves rigidly with the tank and one or more convective masses are attached by springs whose frequencies correspond to the sloshing modes. For elevated tanks under earthquakes, the analogue and its refinements underpin fragility assessments [2,3] and response-mitigation studies that also consider wind [12]. Its use in along-wind response calculations for uplift has not been reported in the literature reviewed.

Foundation behaviour has been examined mainly for wind turbine towers, where the anchorage zone and soil act as a system [13]; cracking and retrofit of foundations have been documented [14–16]; and construction and inspection experience has been reviewed [17]. For lattice tank towers with separate stanchion pedestals, the influence of foundation stiffness on the magnitude and distribution of uplift has not been quantified.

3. Method

3.1 Reference tower and wind actions

The reference tower has a height of 15 m to the tank floor, a 3.4 m square base with four stanchions, and a 100 m³ steel panel tank, 5.0 by 5.0 by 4.0 m, whose self-weight is 10% of the stored water weight. The tower self-weight is 4 kN/m and the grillage 25 kN, giving a drained stabilising load of 183.1 kN and a lumped drained mass $M_0 = 18.7$ t. The lateral stiffness at the tank level, obtained from a three-dimensional frame model of the braced tower on pinned rigid supports, is $k = 6863$ kN/m. Wind actions follow EN 1991-1-4 [4] with a basic wind velocity $v_b = 40$ m/s in terrain category II, a tank force coefficient of 1.5 and lattice force coefficients from EN 1993-3-1 [18]. The design uplift at the windward stanchion under skew wind is computed from the rigid-group distribution of the factored overturning moment, with $\gamma_Q = 1.5$ on wind and $\gamma_{G,inf} = 1.0$ on the stabilising load, the wind forces being multiplied by the structural factor $c_s c_d$. The stabilising load includes the full weight of the stored water at each fill level; only the dynamic model distinguishes impulsive and convective water.

3.2 Impulsive and convective masses

For a rectangular tank of half-length l in the wind direction containing water of mass m_w to a depth h_w , Housner's analogue [1] gives the impulsive mass, the first convective mass and its circular frequency as

$$m_i = m_w \frac{\tanh(\sqrt{3} l/h_w)}{\sqrt{3} l/h_w} \quad (1)$$

$$m_c = 0.527 m_w \frac{l}{h_w} \tanh\left(1.58 \frac{h_w}{l}\right) \quad (2)$$

$$\omega_c^2 = \frac{1.58 g}{l} \tanh\left(1.58 \frac{h_w}{l}\right) \quad (3)$$

The tank walls are treated as rigid, which is appropriate for the stiff, compartmented panel tanks considered; heights of the impulsive and convective masses above the tank floor are neglected relative to the tower height. At full depth in the reference tank, Eqs. (1) to (3) give $m_i = 73.4$ t, $m_c = 32.5$ t and $f_c = \omega_c/2\pi = 0.39$ Hz. As is well known for the analogue, the two masses sum to slightly more than the water mass (by 6% here).

3.3 Two-mass model

Mass 1 comprises the tower, tank, grillage and impulsive water, $M_1 = M_0 + m_i$, and is connected to the ground by the lateral stiffness k . The convective mass m_c is attached to mass 1 by a spring $k_c = m_c \omega_c^2$ and a dashpot $c_c = 2\zeta_c m_c \omega_c$. The wind force $F(t)$ acts on mass 1 because the tank walls shield the water. The equations of motion in the frequency domain are

$$\begin{bmatrix} k + k_c - \omega^2 M_1 + i\omega(c_s + c_a + c_c) & -k_c - i\omega c_c \\ -k_c - i\omega c_c & k_c - \omega^2 m_c + i\omega c_c \end{bmatrix} \begin{Bmatrix} X_1 \\ X_c \end{Bmatrix} = \begin{Bmatrix} F \\ 0 \end{Bmatrix} \quad (4)$$

where $c_s = (\delta_s/\pi)\sqrt{(kM_1)}$ represents a structural logarithmic decrement δ_s and c_a is the quasi-steady aerodynamic damping of the tank,

$$c_a = c_f \rho v_m(z_s) A_{ref} \quad (5)$$

with $\rho = 1.25$ kg/m³, c_f the tank force coefficient, A_{ref} its projected area and $v_m(z_s)$ the mean wind velocity at the reference height $z_s = 0.6h$ of EN 1991-1-4 Annex B. For an SDOF system, Eq. (5) reproduces the aerodynamic decrement $\delta_a = c_f \rho v_m A_{ref}/(2n_1 M)$ used with Annex B. The convective damping ratio ζ_c was taken as 0.5% and varied from 0.1% to 2% in Section 4.3. The response quantity of interest is the restoring force of the tower, kX_1 , which carries the overturning moment to the base; its transfer function is $H(n) = kX_1/F$. Fig. 1 shows the model.

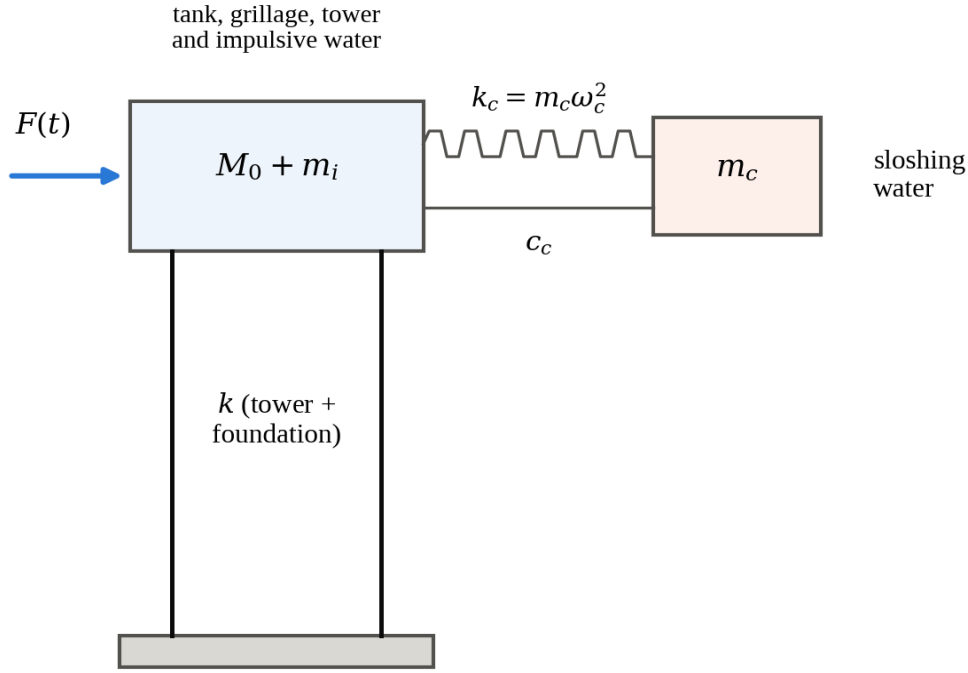


Fig. 1. Two-mass model: tower, tank, grillage and impulsive water ($M_0 + m_i$) on the lateral stiffness k of the tower and foundation, with the convective water m_c attached by k_c and c_c . The wind force acts on the tower mass.

3.4 Spectral structural factor

EN 1991-1-4 Annex B writes the structural factor as

$$c_s c_d = \frac{1 + 2k_p I_v(z_s) \sqrt{B^2 + R^2}}{1 + 7I_v(z_s)} \quad (6)$$

with I_v the turbulence intensity, B^2 the background factor and R^2 the resonance response factor [4]. For an SDOF system $R^2 = (\pi^2/2\delta) S_L(z_s, n_1) R_h R_b$, where S_L is the non-dimensional power spectral density and R_h , R_b are aerodynamic admittance functions:

$$S_L(n) = \frac{6.8 f_L}{(1 + 10.2 f_L)^{5/3}}, \quad f_L = \frac{n L(z_s)}{v_m(z_s)} \quad (7)$$

$$R_\ell = \frac{1}{\eta_\ell} - \frac{1}{2\eta_\ell^2} (1 - e^{-2\eta_\ell}), \quad \eta_\ell = \frac{4.6 \ell f_L}{L(z_s)}, \quad \ell = h, b \quad (8)$$

138 The SDOF expression for R^2 is the narrow-band approximation of the integral of the dynamic
 139 magnification over the wind spectrum. For the two-mass system, the same quantity was
 140 evaluated directly:

$$141 \quad R^2 = \int_0^\infty (|H(n)|^2 - 1) S_L(n) R_h(n) R_b(n) \frac{dn}{n} \quad (9)$$

142 B^2 was taken from Annex B unchanged, so that Eq. (9) adds only the dynamic part. The up-
 143 crossing frequency was $\nu = n_s \sqrt{(R^2/(B^2 + R^2))}$, where n_s is the frequency of the tower-
 144 dominated mode, and the peak factor k_p followed Annex B with an averaging time of 600 s.
 145 Equation (9) was integrated on 60,000 logarithmically spaced points from 0.003 to 30 Hz.

146 **3.5 Foundation compliance and stiffness scatter**

147 To represent footing compliance, the pinned supports of the frame model were replaced by
 148 vertical springs of stiffness $k_v = r EA/H$ at each stanchion, where $EA/H = 74,270$ kN/m is the
 149 axial stiffness of one stanchion over the tower height and r is the spring ratio. Uniform
 150 springs with $r = 5, 2, 1, 0.5$ and 0.2 were used to obtain the lateral stiffness $k(r)$ for the
 151 dynamic model. Uniform springs do not alter the linear distribution of base reactions, so their
 152 effect on uplift is purely dynamic. Unequal support stiffness was studied separately by
 153 multiplying each spring by an independent factor uniformly distributed in $1 \pm s$, with $s = 0.1,$
 154 $0.2, 0.3$ and 0.5 , and 30 realisations for each combination of layout (four-leg and nine-leg
 155 towers), spring ratio ($0.2, 1$ and 5) and scatter. For each realization, the corner uplift under
 156 the factored skew-wind loads was compared with the rigid-group value through the demand
 157 amplification

$$158 \quad \beta = \frac{N_{Ed,frame}}{N_{Ed,rigid}} \quad (10)$$

159 **3.6 Random sample**

160 The random sample of an earlier single-degree-of-freedom study was regenerated with the
 161 same seed: 300 four-leg towers at $v_b = 40$ m/s with the empty tank ratio uniform in 0.05 to

1.0, δ_s in 0.015 to 0.10, a stiffness multiplier on k in 0.2 to 3.0, capacity in 30 to 250 m³ and height in 8 to 25 m, with square tanks 4 m deep. For each tower, the design uplift was computed at 21 fill levels from 0 to 1 with both the rigid-water SDOF model and the two-mass model, and the fill level giving the largest uplift was recorded.

4. Results

4.1 Verification in the SDOF limit

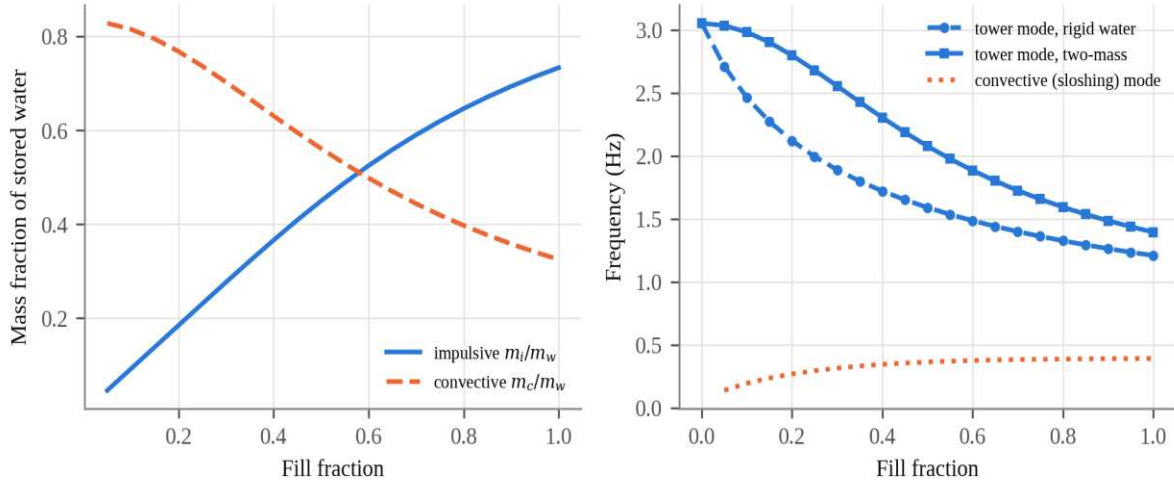
With $m_c = 0$, Eq. (9) should reproduce the Annex B structural factor. Table 1 compares the two for the reference tower with three masses and three decrements. The spectral R^2 exceeds the narrow-band Annex B value by 0.7% to 7.2%, because the integral captures the full resonant peak and the variation of the spectrum across it, but the resulting $c_s c_d$ agrees within 0.5% in all nine cases. The spectral procedure is therefore consistent with Annex B and differs from it only where the dynamics differ.

Table 1. Structural factor from Annex B and from the spectral integral, SDOF limit ($v_b = 40$ m/s).

δ_s	Mass (t)	n_i (Hz)	$c_s c_d$, Annex B	$c_s c_d$, spectral
0.02	18.7	3.05	1.095	1.099
0.02	68.7	1.59	1.435	1.439
0.02	118.7	1.21	1.661	1.664
0.05	18.7	3.05	1.025	1.029
0.05	118.7	1.21	1.338	1.343
0.08	18.7	3.05	0.996	1.000
0.08	118.7	1.21	1.216	1.221

4.2 Mass partition and frequencies

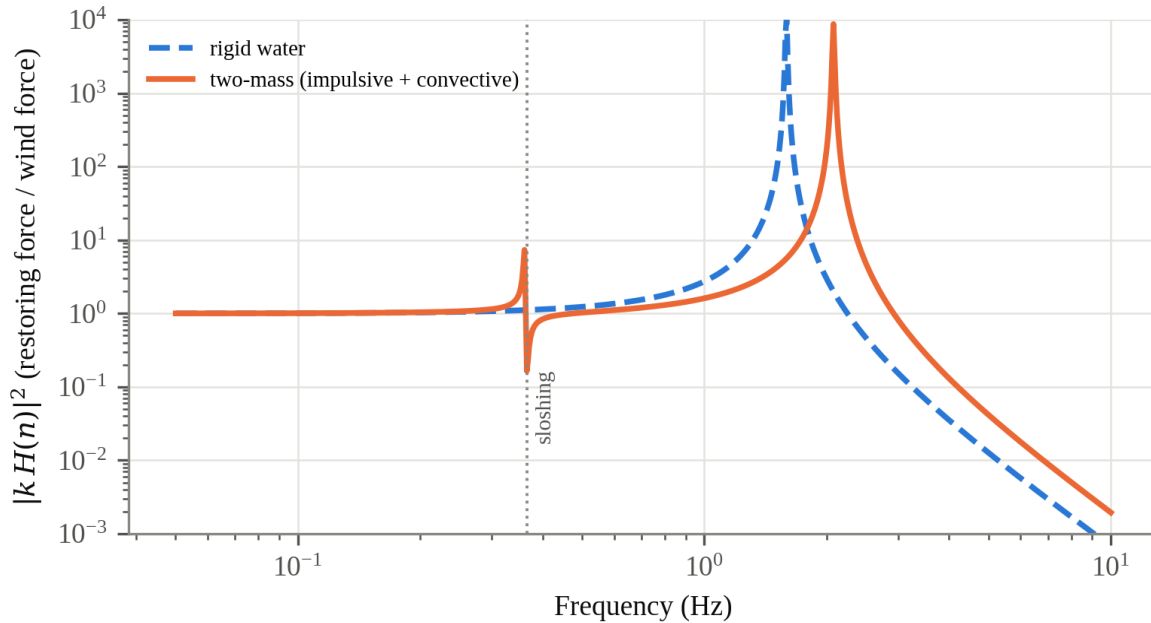
Fig. 2 shows how the stored water divides between impulsive and convective parts as the tank fills, and the resulting frequencies. At low fill, most of the water sloshes: at 20% fill the convective mass is 15.4 t against an impulsive mass of 3.7 t. Even at full depth, a third of the water is convective. Consequently, the tower-dominated frequency falls far less than the rigid-water model predicts: at half fill it is 2.08 Hz rather than 1.59 Hz, and at full fill 1.40 Hz rather than 1.21 Hz. The sloshing frequency rises from 0.27 Hz at 20% fill to 0.39 Hz at full depth.



184

185 Fig. 2. (left) Impulsive and convective fractions of the stored water against fill fraction for the 5.0 by
 186 5.0 by 4.0 m tank; (right) frequency of the tower-dominated mode with rigid water and with the two-
 187 mass model, and the sloshing frequency.

188 Fig. 3 compares the magnitude of the transfer function at half fill. The two-mass system
 189 shows a sharp anti-resonance at the sloshing frequency, where the convective mass acts as a
 190 tuned absorber, and a tower resonance shifted upward to about 2.1 Hz, where the wind
 191 spectrum carries less energy than at 1.6 Hz. Both features reduce the resonant response
 192 relative to the rigid-water model.



193

194 Fig. 3. Squared magnitude of the transfer function from wind force to tower restoring force at half fill
 195 ($\delta_s = 0.02$): rigid water and two-mass model. The dotted line marks the sloshing frequency.

196

4.3 Uplift of the reference tower

Fig. 4 and Table 2 give the design uplift against fill level. With $\delta_s = 0.02$, the rigid-water model predicts a monotonic rise from 691.7 kN empty to 827.5 kN full (+19.6%), so a filled tank would govern. The two-mass model predicts 694.2 kN empty, a minimum of 652.3 kN at 40% fill and 697.8 kN full (+0.5%): the drained and full conditions are practically equal, and neither is exceeded by an intermediate level. With $\delta_s = 0.05$ and 0.08, the two-mass uplift decreases monotonically with filling, so the drained tank governs.

Table 2. Design uplift (kN) of the reference tower against fill fraction, $v_b = 40$ m/s.

Fill	Rigid, $\delta_s = 0.02$	Two-mass, $\delta_s = 0.02$	Rigid, $\delta_s = 0.05$	Two-mass, $\delta_s = 0.05$	Rigid, $\delta_s = 0.08$	Two-mass, $\delta_s = 0.08$
0	691.7	694.2	644.4	647.2	624.7	627.9
0.2	747.2	657.6	651.6	606.2	614.4	584.7
0.4	784.5	652.3	649.9	583.3	598.4	554.5
0.6	808.2	666.6	641.5	571.2	578.1	532.0
0.8	821.7	684.5	627.8	561.0	554.4	510.8
1.0	827.5	697.8	610.0	548.2	527.9	487.8

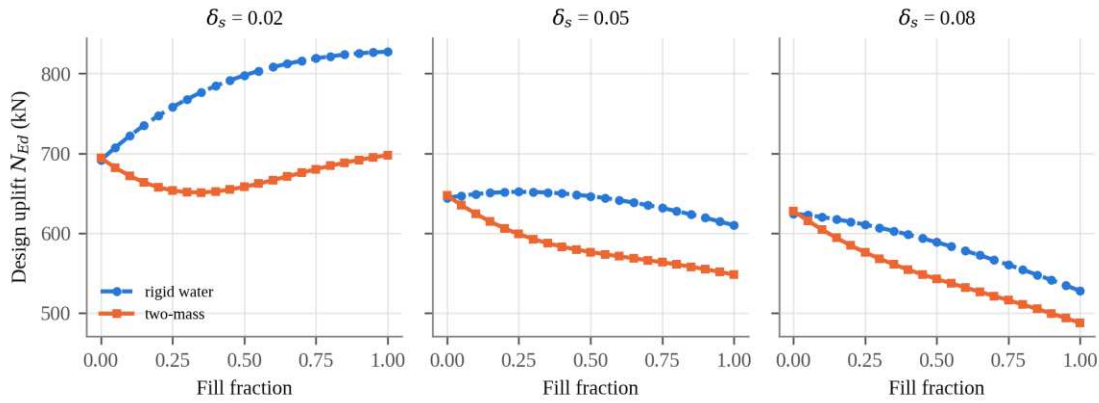


Fig. 4. Design uplift of the reference tower against fill fraction for three structural decrements: rigid-water SDOF model and two-mass model.

Two modelling choices were tested (Table 3). The convective damping ratio matters: with $\zeta_c = 0.1\%$ the full-tank uplift rises to 733.5 kN, 5.7% above the drained value, because a lightly damped absorber leaves stronger side peaks; with $\zeta_c = 2\%$ it falls to 619.2 kN. Reducing the tower contribution to the lumped mass from the full self-weight to a quarter lowered all uplifts but left the comparison unchanged: the rigid-water model still predicted a 22.6% rise from empty to full, and the two-mass model 2.6%.

Table 3. Sensitivity of the two-mass uplift (kN) to the convective damping ratio and to the lumped tower mass; $\delta_s = 0.02$.

Case	Empty	25%	50%	75%	Full	Full vs empty
$\zeta_c = 0.1\%$	694.2	661.9	679.6	710.8	733.5	+5.7%
$\zeta_c = 0.5\%$ (reference)	694.2	653.5	658.1	680.3	697.8	+0.5%
$\zeta_c = 2\%$	694.2	632.1	612.9	615.7	619.2	-10.8%
Quarter tower mass, two-mass	667.4	629.6	638.6	664.6	685.0	+2.6%
Quarter tower mass, rigid water	665.2	736.9	780.9	805.0	815.5	+22.6%

4.4 Random sample

Table 4 summarizes the 300 random towers. With rigid water, a partly or fully filled tank governed in 74 towers (24.7%), by more than 5% in 34 and by up to 34.6%. With the two-mass model, a filled tank governed in 11 towers (3.7%), by more than 5% in 5 and by at most 7.3%. The reduction was largest where the rigid-water model had flagged most towers: among the 66 towers with δ_s below 0.03, the count fell from 45 to 7. Fig. 5 plots the excess of the governing fill level over the drained condition against δ_s for both models.

Table 4. Towers in which a filled tank governs uplift, 300 towers at $v_b = 40$ m/s.

δ_s	Towers	Rigid water: governs	Rigid water: excess > 5%	Two-mass: governs	Two-mass: excess > 5%
0.015–0.03	66	45	24	7	3
0.03–0.05	63	14	7	3	2
0.05–0.07	64	8	2	1	0
0.07–0.10	107	7	1	0	0
All	300	74	34	11	5

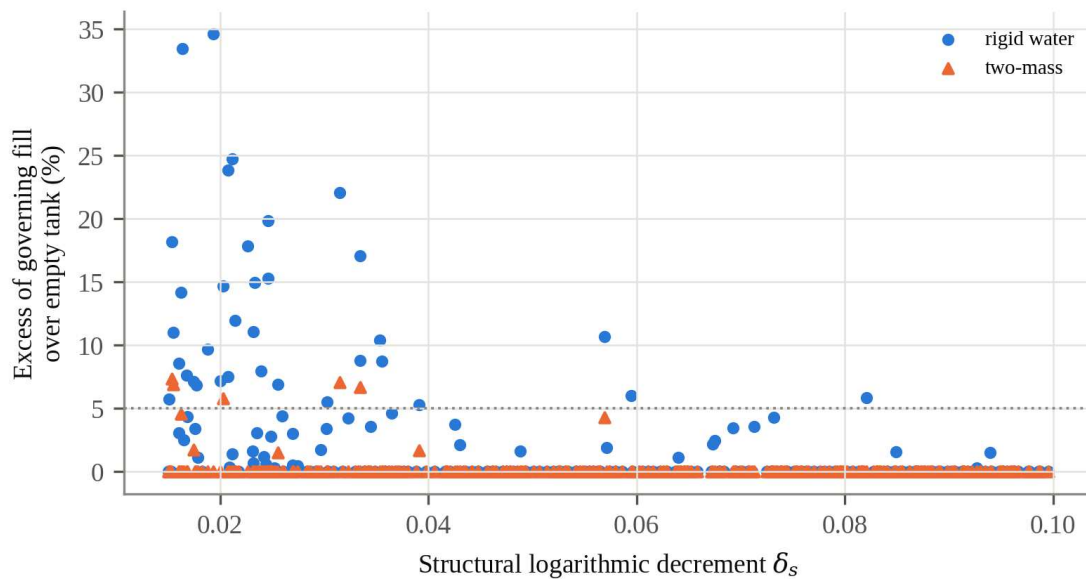


Fig. 5. Excess of the governing fill level over the drained tank, in per cent of the drained uplift, against the structural decrement for 300 random towers: rigid-water and two-mass models.

4.5 Foundation compliance

Uniform vertical springs reduced the lateral stiffness at the tank level from 6863 kN/m on rigid supports to 4694, 3185, 2073, 1221 and 547 kN/m for $r = 5, 2, 1, 0.5$ and 0.2 . Fig. 6 and Table 5 show the effect on the design uplift of the reference tower with the two-mass model and $\delta_s = 0.02$. The drained frequency fell from 3.05 Hz to 1.68 Hz at $r = 1$, and the drained uplift rose by 15.0%, from 694.2 to 798.2 kN. At $r = 0.2$, the drained uplift rose by 33.0%, and the full-tank uplift exceeded the drained value by 21%, because the tower frequency approached the sloshing frequency and the two modes coalesced. That case corresponds to a quasi-static lateral deflection of about 0.23 m at the tank under the characteristic wind force, which would normally be excluded on serviceability grounds.

Table 5. Effect of uniform foundation compliance on the two-mass design uplift (kN), $\delta_s = 0.02$.

Spring ratio r	k (kN/m)	n_s empty (Hz)	Empty	Half	Full	Empty vs rigid supports
Rigid	6863	3.05	694.2	658.1	697.8	—
5	4694	2.52	723.0	698.1	750.9	+4.1%
2	3185	2.08	757.0	738.1	793.7	+9.0%
1	2073	1.68	798.2	777.4	820.7	+15.0%
0.5	1221	1.29	851.0	817.7	853.3	+22.6%
0.2	547	0.86	923.4	940.7	1118.4	+33.0%

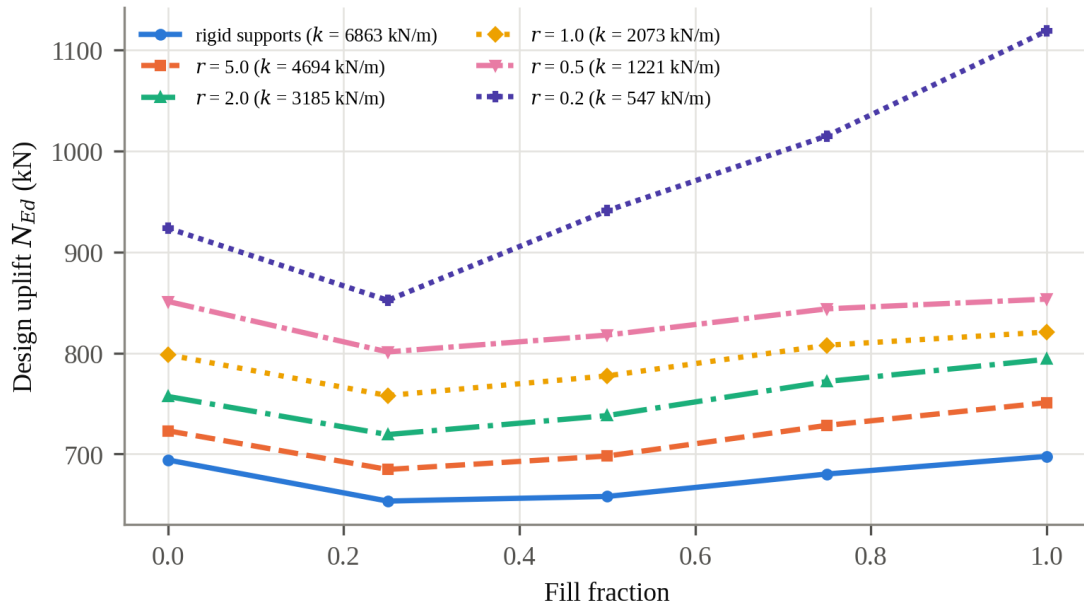


Fig. 6. Design uplift of the reference tower against fill fraction for uniform vertical support springs $k_v = r EA/H$ (two-mass model, $\delta_s = 0.02$). The lateral stiffness at the tank level is given for each case.

4.6 Unequal support stiffness

Fig. 7 gives the 95th percentile of the demand amplification β over 30 realisations. For a four-leg tower on springs of $r = 1$, a $\pm 30\%$ scatter of support stiffness gave $\beta_{95} = 1.11$ and a $\pm 50\%$ scatter gave 1.20; on stiffer springs ($r = 5$) the corresponding values were 1.05 and 1.10. Nine-leg towers were more sensitive: with $r = 1$, $\beta_{95} = 1.14$ at $\pm 30\%$ and 1.25 at $\pm 50\%$, and with soft springs ($r = 0.2$), 1.18 and 1.30. The mean amplification stayed between 1.00 and 1.06, so the effect is one of scatter rather than bias: a stiffer support attracts more of the overturning, and the corner that happens to be stiffest carries the extra uplift.

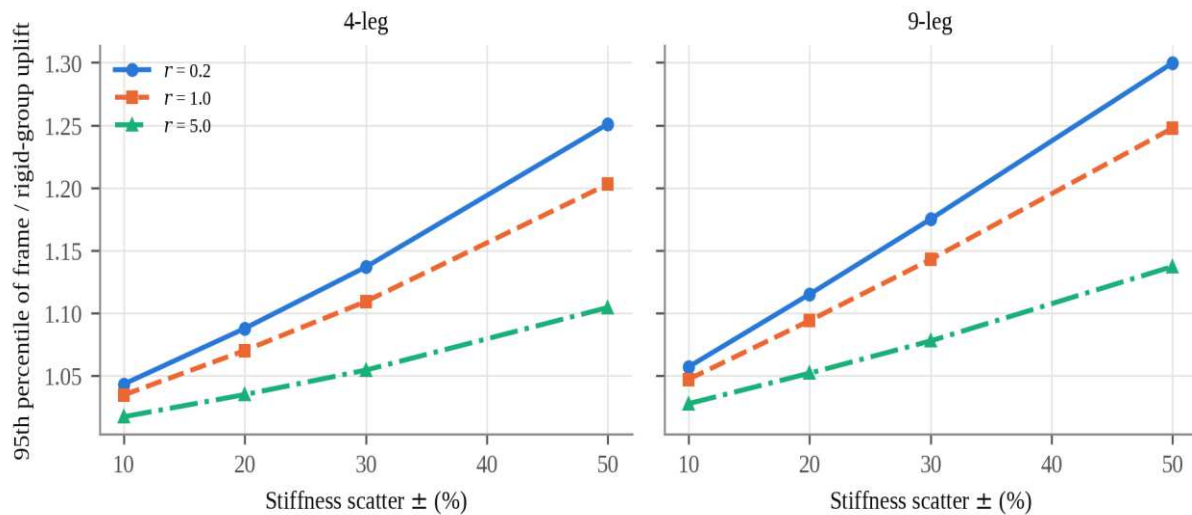


Fig. 7. 95th percentile of the ratio of frame to rigid-group corner uplift against the scatter of support stiffness, for four-leg and nine-leg towers and three spring ratios.

5. Discussion

5.1 Why the rigid-water model overstates the fill effect

The rigid-water model attaches all the stored water to the tower mode. At low fill, however, most of the water is convective (Fig. 2), and at full depth a third still is, so the tower mode sees only part of the added mass and its frequency falls less. The convective mass also acts as a detuned absorber: its frequency of about 0.4 Hz lies well below the tower frequency, so it removes little energy from the tower resonance but introduces an anti-resonance that suppresses response near 0.4 Hz. For the reference tower with low structural damping, the net effect is that the resonant growth from filling is roughly matched by the growth of the

stabilising load. The result that a filled tank governs uplift in a quarter of lightly damped towers at 40 m/s, obtained with rigid water, is therefore largely an artefact of the idealisation; with the water modelled by Housner's analogue, the drained tank governs in about 96% of towers, and the exceptions exceed it by at most 7.3%.

The sensitivity to ζ_c (Table 3) shows that this conclusion is not unconditional. Convective damping in real tanks depends on baffles, internal bracing and the free-surface condition, and a very low value raises the full-tank uplift to about 6% above the drained value. A fill-level check therefore remains prudent for towers with δ_s below about 0.05, but a margin of about 5% to 10% covers the cases found here, rather than the 20% to 35% that the rigid-water model suggests.

5.2 Foundation compliance as the larger effect

The foundation had a larger effect on uplift than the fill level. Uniform compliance lowers lateral stiffness and, therefore, the frequency of every fill state, and because the drained tower is the stiffest and lightest configuration, its resonant response is most sensitive to that loss: a spring ratio of 1 raised the drained uplift by 15%. For towers on a common raft, the effect is small, but for separate footings on compressible ground, the lateral stiffness used in the dynamic assessment should include the foundation. Unequal compliance acts through a different mechanism, redistributing the static overturning among the stanchions (Fig. 7). Its magnitude depends on the scatter, which is rarely known at design stage; values of β_{95} between 1.05 and 1.30 for scatter of $\pm 30\%$ to $\pm 50\%$ indicate the size of the margin required where supports may differ.

The two foundation effects add to each other. A tower on soft, variable footings may see a dynamic increase of 15% or more in its uplift and a further 10% to 25% at the stiffest corner, which together exceed the fill-level effect by a wide margin. This ordering, foundation before fill, reverses the emphasis of assessments that treat the supports as rigid and study the tank contents in detail.

5.3 Design implications

Three practical recommendations follow for the uplift design of elevated steel tank towers in high-wind regions. First, the natural frequency used in the Annex B structural factor should be computed with the impulsive water mass only, from Eq. (1), rather than with the full water mass; this removes most of the spurious fill-level governance at no extra cost. Second, the lateral stiffness should include the vertical compliance of separate footings, since omitting it underestimates the drained uplift by about 15% when the footing stiffness equals the axial stiffness of a stanchion. Third, where supports may differ in stiffness, increase the rigid-group corner uplift by a factor of about 1.1 to 1.25, or compute the reactions with the foundation modelled. The drained condition should still be checked against the full condition for lightly damped towers, with a margin of about 5% to 10%.

5.4 Limitations

The analysis is linear and considers only along-wind buffeting; across-wind excitation, vortex shedding from the tank, and aeroelastic effects of the lattice were not modelled. The tower was represented by its fundamental mode, the tank walls as rigid, and only the first sloshing mode was included, with the masses located at the tank floor. The wind spectrum, admittance functions, background factor, and peak factor follow EN 1991-1-4 Annex B, and the aerodynamic damping is quasi-steady. The sloshing wave amplitude and freeboard, which may limit the validity of the linear analogue at high response, were not checked. Foundation compliance was represented by vertical springs only; rotational and lateral soil springs and soil damping were omitted. Measured damping values for bolted and welded tank towers, and full-scale or wind-tunnel data on filled elevated tanks, would allow the conclusions to be tested directly.

6. Conclusions

1. A spectral generalization of the EN 1991-1-4 Annex B procedure reproduced the Annex B structural factor within 0.5% in the single-degree-of-freedom limit and extends it consistently to a tower carrying impulsive and convective water.

2. Treating the stored water as rigid overstates the effect of filling on wind uplift. For the reference tower with $\delta_s = 0.02$, the rise from drained to full was 19.6% with rigid water and 0.5% with the two-mass model; among 300 random towers at 40 m/s, a filled tank governed in 74 cases with rigid water and in 11 cases with the two-mass model, by at most 7.3%.
3. Foundation compliance had a larger effect than fill level: vertical support springs equal to the axial stiffness of a stanchion raised the drained uplift by 15%, and softer supports by up to 33%.
4. Unequal support stiffness of $\pm 30\%$ raised the 95th-percentile corner uplift by 5% to 18% depending on layout and foundation stiffness, and $\pm 50\%$ by up to 30%.
5. Uplift design should use the impulsive water mass in the frequency calculation, include foundation compliance in the lateral stiffness, and allow for unequal support stiffness where stanchions stand on separate footings.

Declaration of generative AI and AI-assisted technologies in the writing process

During the preparation of this work, the author used Claude (Anthropic) to assist with drafting, restructuring the thesis material into article form and implementing the analysis scripts. After using this tool, the author reviewed and edited the content as needed and takes full responsibility for the publication's content.

CRedit authorship contribution statement

Conceptualisation, methodology, software, validation, formal analysis, writing – original draft, review and editing

Declaration of competing interests

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

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Data availability

The analysis scripts and generated data (Python; seed 20260925 for the random sample) are available.

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