

Intze Tank Design — Complete Guide & Worked Example

Every governing force in an Intze tank, component by component, followed by one complete worked example for a 50 kL tank on a 20 m staging — every formula shown as formula → substitution → result, nothing skipped.

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1. Part 1 — Concepts and checks discussed, component by component
2. Part 2 — Complete worked example: 50 kL Intze tank, top dome to foundation, including staging, wind and seismic

Part 1 — Concepts and Checks Discussed

A recap of the reasoning behind each component. The actual numbers for every formula here are worked through in full in Part 2.

1. Top dome

What governs its design?

Self-weight and live load (maintenance access). These create two internal forces: meridional thrust (always compression, running along the slope) and hoop thrust (compression near the crown, only turning to tension if the dome's own half-angle exceeds about 51.8° — most Intze top domes are kept shallow enough to avoid this).

What checks are needed?

Minimum thickness (rarely under 100 mm), meridional and hoop stress, minimum steel both ways, a local bending check right at the ring beam B1 junction (membrane theory alone misses this), and crack width. In practice, minimum thickness and minimum steel govern — the shape is naturally so efficient that calculated stress is rarely the limit.

2. Top ring beam, B1

What forces act on it?

Two: the dome's outward push (hoop tension, $T_1 = H_1 \times r$ — the number that actually needs steel), and the dome's own weight pressing straight down. The vertical part needs no special design at B1 — the wall sits right below, so it simply flows through as ordinary compression.

3. Cylindrical wall

What governs it?

Hydrostatic pressure, increasing with depth, causing hoop tension (maximum at the base) — and the wall's own weight, carried straight down with no complication, same reasoning as B1.

Minimum steel — which code, and how much?

IS 3370 (Part 2):2009, Cl 8.1 — a *surface-zone* rule, not a percentage of the whole thickness: 0.24% of a zone ($t/2$ or 250 mm, whichever is smaller) per face, for a tank under 15 m in every dimension; 0.35% for a bigger one.

4. Ring beam, B2 (wall–cone junction)

What loads onto it?

Two things pushing the same way: the wall's own hoop tension at its worst (full depth), plus the outward thrust the conical slab delivers at this junction (H_c). They simply add.

Where does H_c come from?

From the cone's meridional force N_ϕ , split into vertical (carries the weight above) and horizontal (H_c) parts — exactly the same decomposition used for every sloped member in this tank.

5. Conical slab

What's different about the cone versus the wall?

Pressure still increases going down, but the radius *shrinks* at the same time — so unlike the wall, the worst hoop tension doesn't sit at either end. It has to be found by treating hoop tension as $\text{pressure} \times \text{radius}$ and finding where that product peaks along the slope.

Does the meridional (compression) force need calculated steel?

No — it's compression, and concrete carries that comfortably on its own. Only a stress check plus minimum steel is needed there, the same pattern as the dome.

6. Bottom dome

Which way does it curve, and why does that matter?

Upward, into the tank (RDSO/Intze arrangement) — not sagging down like an inverted bowl. This is why it can be analysed with the exact same compression formulas as the top dome, why its water-load calculation *subtracts* a displaced volume rather than adding one, and why its edge thrust pushes outward, the same direction as the top dome's — which is what lets it oppose the cone's thrust at B3.

7. Ring beam, B3 (circular girder)

What forces does it carry?

Three, and this is the busiest member in the tank: (1) a net ring force — cone pushing inward vs. dome pushing outward, whichever wins decides compression or tension; (2) a vertical load, from the same two meridional forces' vertical components, handed to the columns; (3) bending, because unlike every ring beam above it, B3 doesn't sit on a continuous wall — it spans between individual columns, like an ordinary beam.

8. Columns and staging

What makes column design different from everything above?

Every other member had one governing force, checked on its own. A column always carries axial load *and* bending moment together, and they must be designed for as a combined pair (P–M interaction) — never as two separate checks. Tall, slender columns also pick up an extra moment purely from their own geometry, on top of whatever wind or seismic contributes.

9. Foundation

What's checked?

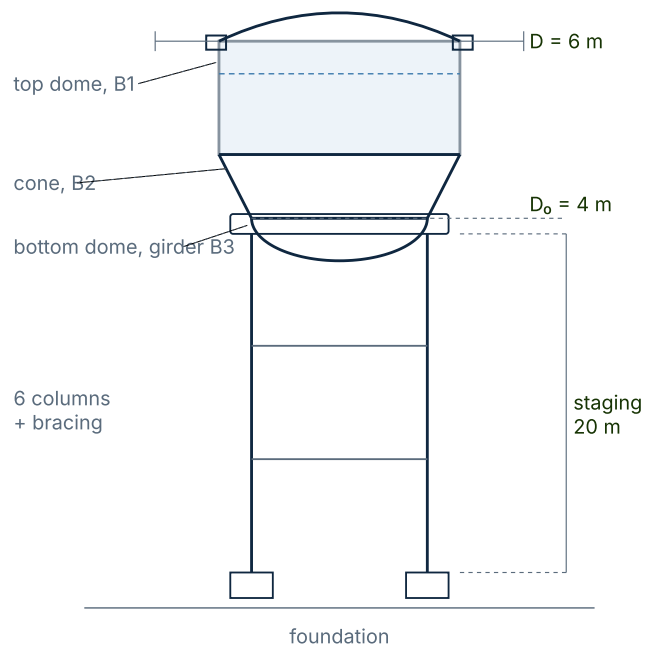
Bearing pressure under the worst column ($q = P/A \pm M/Z$) against the safe bearing capacity — and separately, whether any column goes into net uplift under wind or seismic, since a plain footing can't take tension from the soil.

Part 2 — Complete Worked Example

A 50 kL Intze tank: 6 m diameter, on 6 columns, 20 m staging. Every quantity below is shown as formula → substitution → result.

Geometry adopted

D = 6 m · water depth $h_3 = 1.5$ m · freeboard = 0.3 m (full wall height $h = 1.8$ m) · top dome rise $h_1 = 1$ m, thickness 120 mm · cone height $h_0 = 1$ m, bottom diameter $D_0 = 4$ m, thickness 250 mm · bottom dome rise $h_2 = 0.8$ m, thickness 200 mm · wall thickness 200 mm · staging height 20 m, 6 columns, 450×450 mm, 5 bracing panels.



Elevation of the tank used in this worked example (schematic, not to scale). Two of the six columns are shown, as usual in this kind of drawing.

Capacity check

$$V_{\text{cyl}} = \pi/4 \cdot D^2 \cdot h_3 = \pi/4 \cdot 6^2 \cdot 1.5 = \mathbf{42.41 \text{ m}^3}$$

$$V_{\text{cone}} = \pi h_0 / 12 \cdot (D^2 + D D_0 + D_0^2) = \pi / 12 \times (36 + 24 + 16) = \mathbf{19.90 \text{ m}^3}$$

$$V_{\text{ded}} = \pi h_2^2 / 3 \cdot (3R_2 - h_2) = \mathbf{2.19 \text{ m}^3} \text{ (} R_2 \text{ found below)}$$

Capacity = 42.41 + 19.90 - 2.19 ≈ 57.0 m³ (~57 kL)

1. Top dome

$$R_1 = (D^2+4h_1^2)/(8h_1) = (36+4)/8 = 5.000 \text{ m}$$

$$\Phi_1 = \sin^{-1}(D/2R_1) = \sin^{-1}(3/5) = 36.87^\circ$$

$$w_1 = \gamma_c \cdot t + LL = 25 \times 0.12 + 1.5 = 4.50 \text{ kN/m}^2$$

$$N_\phi(\Phi_1) = w_1 R_1 / (1 + \cos \Phi_1) = 4.5 \times 5 / (1.8) = 12.50 \text{ kN/m}$$

$$N_\theta(\Phi_1) = w_1 R_1 [\cos \Phi_1 - 1 / (1 + \cos \Phi_1)] = 5.50 \text{ kN/m (compression – under } 51.8^\circ)$$

$$H_1 = N_\phi \cdot \cos \Phi_1 = 12.5 \times 0.8 = 10.00 \text{ kN/m}$$

$$T_1 = H_1 \times D / 2 = 10 \times 3 = 30.0 \text{ kN}$$

$$A_s \text{ (B1 ring, } \sigma_{st}=150) = T_1 / \sigma_{st} = 30000 / 150 = 200 \text{ mm}^2$$

Both N_ϕ and N_θ are compression — minimum steel governs the dome slab itself; only the ring beam B1 needs a calculated tension check.

2. Cylindrical wall

$$p = \gamma_w \cdot h_3 = 9.81 \times 1.5 = 14.72 \text{ kN/m}^2$$

$$T_h = p \times D / 2 = 14.72 \times 3 = 44.14 \text{ kN/m}$$

$$A_s = T_h / \sigma_{st} = 44140 / 150 = 294 \text{ mm}^2/\text{m}$$

$$A_{s,min} \text{ (IS 3370 Cl 8.1, zone}=\min(t/2,250)=100\text{mm, } 0.24\%) = 240 \text{ mm}^2/\text{m per face}$$

Calculated (294) > minimum (240) → calculated demand governs

3. Conical slab

$$\Phi_0 = \tan^{-1}[(D-D_0)/2h_0] = \tan^{-1}(1/1) = 45.0^\circ$$

$$L = \sqrt{h_0^2 + ((D-D_0)/2)^2} = \sqrt{2} = 1.414 \text{ m}$$

$$\text{Water on cone} = \gamma_w [\pi/4 (D^2 - D_0^2) h + \pi h_0 / 12 (D^2 + D_0^2 + D D_0) - \pi / 4 D_0^2 h_0] = 349.3 \text{ kN}$$

$$\text{Self-wt} = \gamma_c \cdot \pi \cdot [(D+D_0)/2] \cdot L \cdot t = 25 \times \pi \times 5 \times 1.414 \times 0.25 = 138.8 \text{ kN}$$

Peak hoop tension: $N_\theta(z) = p(z)r(z)/\sin\Phi_0$, maximised at $z^* = r_{\text{top}}/(2\tan\Phi_0) - h/2$ (h = full wall height, same "water to the brim" basis as the water-load term above):

$$z^* = 3/2 - 1.8/2 = 0.60 \text{ m down the slope}$$

$$N_{\theta, \text{peak}} = [9.81 \times (1.8 + 0.6)] \times (3 - 0.6) / \sin 45^\circ = 79.9 \text{ kN/m}$$

$$A_s (\text{peak hoop}) = 79900/150 = 533 \text{ mm}^2/\text{m}$$

Line load reaching B2 (top-dome + B1 + wall + B2 self-weights, added): 16.48 kN/m → total force at D: **310.6 kN**

$$N_\phi(\text{top}) = V_{\text{top}} / (2\pi r_{\text{top}} \cos\Phi_0) = 310.6 / (2\pi \times 3 \times 0.707) = 23.30 \text{ kN/m}$$

$$H_c = N_\phi \times \sin\Phi_0 = 23.3 \times 0.707 = 16.48 \text{ kN/m}$$

4. Ring beam, B2

$$T_{B2} = T_h(\text{wall base}) + H_c = 44.14 + 16.48 = 60.62 \text{ kN/m}$$

$$A_s = 60620/150 = 404 \text{ mm}^2/\text{m}$$

Carrying forward: total load reaching the bottom of the cone (adding water + self-weight of the cone) = 310.6 + 349.3 + 138.8 = **798.7 kN**

$$N_\phi(\text{bottom of cone}) = 798.7 / (2\pi \times 2 \times 0.707) = 89.9 \text{ kN/m}$$

$$H_{\text{cone}} (\text{into B3}) = 89.9 \times \sin 45^\circ = 63.56 \text{ kN/m}$$

5. Bottom dome

$$R_2 = (D_0^2 + 4h_2^2) / (8h_2) = (16 + 2.56) / 6.4 = 2.900 \text{ m}$$

$$\Phi_2 = \sin^{-1}(D_0/2R_2) = \sin^{-1}(2/2.9) = 43.60^\circ$$

$$\text{Water} = \gamma_w [\pi/4 D_0^2 (h+h_0) - \pi h_2^2 / 3 (3R_2 - h_2)] = 256.3 \text{ kN}$$

$$\text{Self-wt} = 2\pi R_2 h_2 t \gamma_c = 2\pi \times 2.9 \times 0.8 \times 0.2 \times 25 = 72.9 \text{ kN}$$

$$w' = (256.3 + 72.9) / (2\pi R_2 h_2) = 329.1 / 16.24 = 22.58 \text{ kN/m}^2$$

$$N_{\phi}(\Phi_2) = w'R_2/(1+\cos\Phi_2) = \mathbf{37.98 \text{ kN/m}}$$

$$N_{\theta}(\Phi_2) = w'R_2[\cos\Phi_2-1/(1+\cos\Phi_2)] = \mathbf{9.44 \text{ kN/m (compression)}}$$

$$H_{\text{dome}} (\text{into B3}) = N_{\phi}\times\cos\Phi_2 = 37.98\times0.724 = \mathbf{27.50 \text{ kN/m}}$$

Stress check: $37.98/200 \approx 0.19 \text{ N/mm}^2$ (vs ~ 6 allowed) $\rightarrow$ minimum steel governs both directions.

6. Ring beam, B3

$$\text{Net} = H_{\text{cone}}-H_{\text{dome}} = 63.56-27.50 = 36.06 \text{ kN/m} \times r(=2\text{m}) = \mathbf{72.1 \text{ kN, compression}}$$

$$\text{Total vertical at B3} = 798.7 \text{ (cone)} + 329.1 \text{ (dome)} = \mathbf{1127.8 \text{ kN}}$$

$$\text{Girder self-wt} = b\times D\times\gamma_c = 0.45\times0.6\times25 = \mathbf{6.75 \text{ kN/m}}$$

$$\text{UDL, } w = 1127.8/(\pi\times4) + 6.75 = 89.75+6.75 = \mathbf{96.50 \text{ kN/m}}$$

$$L_{\text{span}} = \pi D_{\theta}/n = 12.57/6 = \mathbf{2.094 \text{ m}}$$

$$M = wL^2/8 = 96.5\times2.094^2/8 = \mathbf{52.9 \text{ kN}\cdot\text{m}}, V = wL/2 = \mathbf{101.1 \text{ kN}}$$

$$A_{\text{st}} (\text{exact IS 456, } b=450, d=550) = 0.5(f_{ck}/f_y)bd[1-\sqrt{(1-4.6M/f_{ck}bd^2)}] = \mathbf{225 \text{ mm}^2}$$

$$P_{\text{column}} (\text{gravity}) = (1127.8+6.75\times\pi\times4)/6 = \mathbf{202.1 \text{ kN}}$$

7. Wind (IS 875 Part 3:2015)

$$V_z = V_b k_1 k_2 k_3 k_4 = 47\times1\times1.08\times1\times1 = \mathbf{50.76 \text{ m/s}}$$

$$p_z = 0.6V_z^2 = \mathbf{1.546 \text{ kN/m}^2}$$

$$F_w(\text{container}) = p_z\times A\times C_f = 1.546\times22.8\times0.7 = \mathbf{24.67 \text{ kN}}$$

$$F_w(\text{staging}) = p_z\times A_{\text{gross}}\times\phi_{\text{solidity}}\times C_{f,\text{frame}} = 1.546\times80\times0.20\times1.9 = \mathbf{47.00 \text{ kN}}$$

Total $F_w = 71.7 \text{ kN}$, at $\bar{h} \approx 14.1 \text{ m} \rightarrow M_w = 71.7\times14.1 \approx 1010 \text{ kN}\cdot\text{m}$

The staging solidity ratio and frame C_f are assumed representative values, not taken from a specific IS 875 table lookup for this exact member layout — a real design should confirm these against the actual column/bracing geometry.

8. Seismic (IS 1893 Part 2:2014)

$$h_{eq} = V_{cap}/(\pi/4 \cdot D^2) = 57.0/28.27 = \mathbf{2.016\ m} \ (h/D=0.336)$$

$$m_i/m = \tanh(0.866D/h)/(0.866D/h) = \mathbf{0.384} \rightarrow m_i=\mathbf{21.9\ t}$$

$$m_c/m = 0.23(D/h)\tanh(3.68h/D) = \mathbf{0.578} \rightarrow m_c=\mathbf{33.0\ t}$$

$$A_{h,i} = (Z/2)(I/R)(S_a/g)_i = (0.16/2)(1.5/2.5)(2.5) = \mathbf{0.120}$$

$$A_{h,c} = (0.16/2)(1.5/2.5)(1.75) = \mathbf{0.084}$$

$$V_i = A_{h,i}(m_i+m_s)g = \mathbf{47.6\ kN}, \ V_c = A_{h,c}m_cg = \mathbf{27.2\ kN}$$

$$V\ (SRSS) = \sqrt{(V_i^2+V_c^2)} = \mathbf{54.8\ kN}$$

Overturning moment (impulsive+convective, about the base) $\approx$ 1567 kN·m

h_i, h_c used here are indicative fractions of h_{eq} , not the exact IS 1893 Part 2 table values for this h/D — check the code's own h_i/h_c formulas for a real design. The participating staging mass m_s is a rough one-third estimate.

9. Column design

Case	V (kN)	M (kN·m)	Governs
Wind	71.7	1010	base shear
Seismic	54.8	1567	overturning moment governs column design

Per the usual rule (and the RDSO notes' own instruction): wind and seismic are checked as two separate, complete cases — never combined. Seismic's moment governs here despite its lower shear.

$$\Delta P = M \cdot y_{max}/\Sigma y^2 = 1567 \times 2/24 = \mathbf{261.1\ kN}$$

$$P_u(\text{leeward}) = 1.5(202.1+261.1) = \mathbf{694.8\ kN}$$

$$P_u(\text{windward}) = 1.5(202.1-261.1) = \mathbf{-88.5\ kN - net\ UPLIFT}$$

The windward column goes into net tension under this seismic case — the foundation there needs an uplift check, not just a bearing check (same flag raised for the foundation earlier in Part 1).

$$L_e/D = (\text{staging height}/5 \text{ panels})/0.45 = 4/0.45 = \mathbf{8.9 - short\ column} \text{ (no added slenderness moment)}$$

$$P_{uz} \text{ (1\% steel, } 450 \times 450) = 0.45f_{ck}A_g(1-p)+0.75f_yA_gp = \mathbf{\approx 3015\ kN}$$

$P_u/P_{uz} \approx 0.23 \rightarrow$ a moderate axial ratio; final steel % is read from an SP-16 (or strain-compatibility) P–M interaction chart for this P_u, M_u pair — not a closed-form formula, per the interaction discussion in Part 1.

Part 2 uses standard, checked formulas for every shell and ring-beam quantity (sections 1–6). Sections 7–9 (wind, seismic, column) use correct governing formulas with clearly flagged simplified inputs where a real design would need the specific IS 875/1893 table lookups for the exact geometry in hand.

For study and quick recall only. Always verify against the current edition of IS 3370, IS 456, IS 875 and IS 1893 before actual design use, and have the design checked by a qualified structural engineer. © Nirmaanpro