

A COMPLETE DESIGN MANUAL

Intze Elevated Water Tank

Analysis and design procedure,
with a fully worked example

IS 456 : 2000 · IS 3370 (Parts 1, 2 and 4) : 2009
IS 875 (Parts 1, 2 and 3) : 2015 · IS 1893 (Part 1) : 2016
IS 1893 (Part 2) : 2014 · IS 11682 : 1985 · IS 1904 : 1986

Scope and status of this document

This manual explains how an Intze-type elevated reinforced concrete water tank is analysed and designed, and works one tank completely through from the storage requirement to the foundation steel. It is written as a reference for an engineer or student who wants to understand the procedure and be able to reproduce it by hand.

It is not a substitute for the Indian Standards themselves, and it is not a design certificate. An elevated water tank is a life-safety structure: the geometry, the reinforcement, the staging and the foundation of a real tank must be designed, checked and signed by a licensed structural engineer against the current editions of the codes, a site-specific geotechnical report, and the requirements of the sanctioning authority.

Clause and table numbers are quoted against the editions listed on the title page. Codes are revised and amended; verify every reference against your own copy before relying on it.

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How to read this manual

The book is in six parts, and they are meant to be read in order the first time.

Part I — Chapters 1 to 3 is the theory you need before any arithmetic makes sense: what the Intze shape is for, how the load actually travels out of the water and into the ground, which code governs which member and why liquid-retaining concrete is still designed by working stress in 2026, and the membrane theory of shells of revolution from which every force in the container is derived. Chapter 3 is the mathematical heart of the book. Everything in Part III is an application of its two equations.

Part II — Chapters 4 to 7 turns the storage requirement into a geometry, then puts loads on it: dead, imposed and water loads and their centres of gravity; the full wind procedure of IS 875 (Part 3) : 2015; and the two-mass spring-mass seismic model of IS 1893 (Part 2) : 2014, derived rather than merely quoted.

Part III — Chapters 8 to 12 designs the container, element by element, in the order the load travels: top dome, top ring beam, cylindrical wall, ring beam B₂, conical slab, circular girder, bottom dome and bottom ring beam.

Part IV — Chapters 13 to 15 takes the reactions from the container into the staging and the ground: load combinations and how the overturning moment is shared between the columns, column design on a real interaction curve, bracing beams, and the foundation.

Part V — Chapter 16 is a complete worked example. A 450 kilolitre tank on 20 metres of staging in Zone III is carried through every step of Chapters 4 to 15 with real numbers. Every result is also matched against the corresponding line of the *Intze Tank Designer PRO* software, so the two can be used to check each other.

Part VI — Chapter 17 is detailing: what the reinforcement actually looks like at each junction, how the laps and anchorages are arranged, where the construction joints and water bars go, and the sequence in which the thing is built and tested.

The appendices hold the notation, the property and permissible-stress tables, a clause-by-clause index of every code reference used in the book, and the bibliography.

Conventions

Forces are in kilonewtons and moments in kilonewton-metres unless stated. Lengths are in metres in analysis and in millimetres in detailing — the point at which the change happens is always stated. Stresses are in newtons per square millimetre, which is the same thing as a megapascal.

A membrane force per unit length in kilonewtons per metre is numerically equal to the same force in newtons per millimetre. That coincidence is used constantly: a hoop tension of 157.7 kN/m divided by a permissible steel stress of 150 N/mm² gives the steel area directly in mm² per metre, with no conversion factor. It is worth fixing in mind now.

Tension is positive throughout. Where a membrane force comes out negative the member is in compression, and the manual says so rather than silently taking a modulus.

A clause reference in the margin of a formula, like [IS 1893-2 Cl. 4.6.2], means that the expression is taken from that clause. Where the manual derives something the code only tabulates, it says so.

A note on the arithmetic

Every number in Chapter 16 has been computed twice — once by hand following the method in the text, and once by the software — and the two agree. Intermediate values are carried at full precision and only rounded for display, so if you reproduce a step on a calculator using the rounded figures printed here you may see a difference in the last digit. That is rounding, not an error, and the manual says where it matters.

The Intze Form and Its Load Path

1.1 What problem the shape solves

An elevated service reservoir has to hold a large volume of water high in the air and hand the weight down to a small number of columns. The obvious container is a cylinder with a flat floor. It is also the wrong one, and the reason is worth stating carefully because the whole Intze geometry follows from it.

A flat circular floor spanning the full tank diameter carries the water by *bending*. Bending is an inefficient way to carry load: the stress varies linearly through the depth, so only the extreme fibres work hard, and the moment grows with the square of the span. For a tank of 11 metres diameter holding 3 metres of water, a flat slab floor works out at 400 to 500 mm thick with heavy steel in two directions — and every kilogram of that floor is dead load that the columns and the foundation must also carry, for the whole life of the structure.

A *shell* carries load by direct stress instead. If the surface is curved and the load is reasonably smooth, the internal forces are almost entirely in the plane of the shell — tension or compression along the surface — and the material is uniformly stressed through the thickness. A shell can therefore be thin. Replacing the flat floor with a conical shell and a spherical dome turns a bending problem into a membrane problem, and the floor thickness drops from 450 mm to 200 mm.

There is a second saving, and it is the one people usually miss. If the floor slopes inward and downward to a circle appreciably smaller than the tank, the columns can be put on *that* smaller circle instead of on the tank diameter. The staging becomes narrower, the bracing beams shorter and stiffer, the raft smaller, and the whole frame more compact — while the container above stays as wide as the capacity requires.

But a curved floor introduces a difficulty. A cone or a dome, unlike a flat plate, pushes sideways at its edges. Those horizontal thrusts have to go somewhere, and if they are not dealt with they appear as very large hoop forces in the ring beams, which then need to be very large themselves — giving back in ring beam concrete what was saved in floor concrete.

The Intze principle

The Intze tank resolves this by using *two* curved surfaces at the bottom whose horizontal thrusts act in *opposite* directions. A conical shell slopes inward and downward from the cylindrical wall to a smaller circle; a spherical dome, *arching upward*, then closes that circle from below. At the circular girder where the two meet — which is also where the columns are — the cone pushes inward and the arched dome pushes outward. By choosing the rise of the bottom dome correctly the two can be made to very nearly cancel, and the girder is left carrying almost no hoop force at all, free to do its real job of spanning between the columns.

That is the whole idea. Everything else — the proportioning ratios, the way the columns are placed, the sequence of the design — is a consequence.

Why it is called an Intze tank

After Otto Intze (1843–1904), professor at Aachen, who developed the form for elevated steel water towers in Germany in the 1880s. The same geometry was carried over into reinforced concrete and is now the standard elevated service reservoir across India, used by municipal bodies, the Public Health Engineering departments, and Indian Railways in capacities from about 50 to 2000 kilolitres.

1.2 Where the columns go

This is the point on which most textbook treatments are vague and on which it is easiest to go wrong, so it is worth settling before anything else.

The columns are *not* at the tank diameter

In the standard Intze arrangement — and in every issued drawing the author has checked, including RDSO standard drawing B-1235 — the circular girder and the staging columns sit on a circle of diameter $2r$ at the **bottom of the conical slab**. The entire container, cone included, stands *above* that level. Nothing hangs below the girder except the bottom dome, and that arches upward into the tank.

Put the columns at the tank diameter instead and two things go wrong at once. Geometrically, the staging is as wide as the tank, which throws away the compactness the cone was bought for. Structurally — and this is the serious one — the cone and the bottom dome now hang from the girder in tension, every horizontal thrust in the bottom half of the tank reverses, and the Intze balance stops working: the two thrusts at the ring add instead of cancelling.

You can check the arrangement on any real drawing without reading the notes. Find the column circle. If its diameter is close to the tank diameter, the container hangs; if it is around 0.6 to 0.7 of the tank diameter — the same as r/R — the container stands on the cone bottom. It is always the second.

Reading it off the drawing's own numbers

RDSO B-1235 gives a tank diameter of 11.48 m and, in its seismic load row, a leeward column load of 157 340 kg against a windward 81 340 kg, with an overturning moment of 576 810 kg m on eight columns. Each column deviates from the mean by $\Delta P = 2M/(nR_c)$, so $\Delta P = (P_{\text{lee}} - P_{\text{wind}})/2 = 372.8 \text{ kN}$ and $R_c = 2M/(n \Delta P) = 2(5658.5)/(8 \times 372.8) = 3.795 \text{ m}$ — a column circle of 7.59 m, not 11.48 m. A second and quite independent check agrees: eight times the drawing's mean column load, less its tabulated dead load, leaves 4409 kN against $450 \times 9.81 = 4415 \text{ kN}$ of water. The columns are at the cone bottom, and they are carrying the whole tank.

1.3 The parts of the tank

Figure 1.1 names the elements. Learn these names now; the rest of the book uses them without explanation. Note in particular the numbering of the three rings: B_1 at the top of the wall, B_2 at the wall-cone junction, and the circular girder B_3 at the cone bottom, on the columns.

Top dome

A shallow spherical cap closing the tank. It keeps out rain, dust, birds and sunlight, and it carries only its own weight plus an access live load. Thickness 100 to 120 mm, nominal steel both ways.

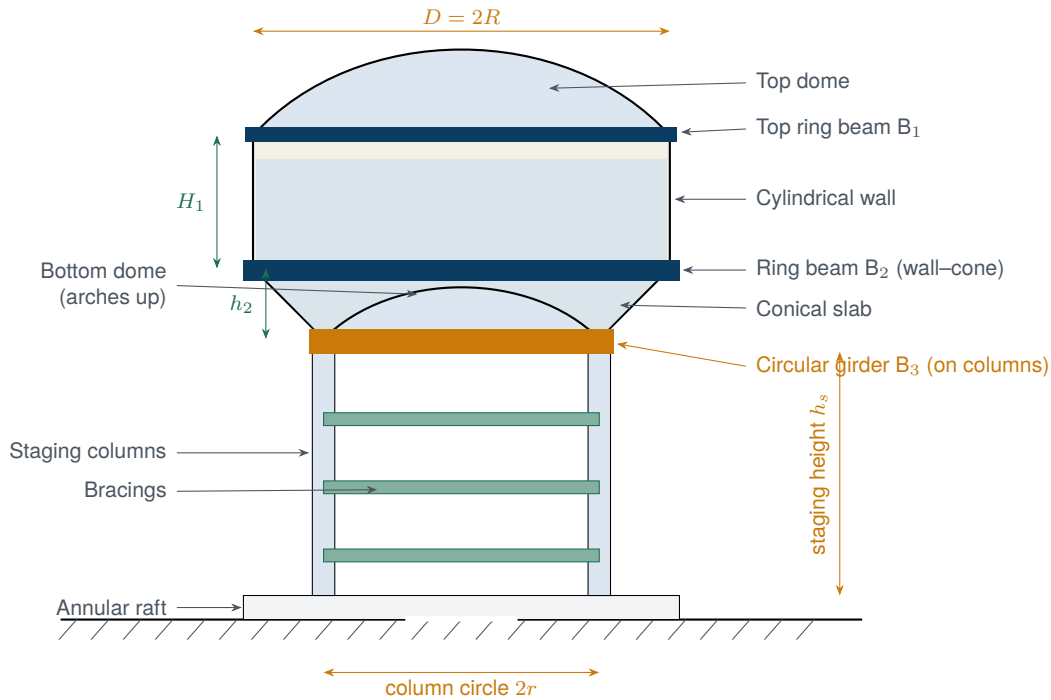


Figure 1.1: The elements of an Intze tank, in section. The circular girder B_3 sits on the columns at the *bottom* of the conical slab, on a circle of diameter $2r$. The whole container stands above it; only the bottom dome closes the space below, and it arches upward into the tank.

Top ring beam B_1

The ring at the springing of the top dome. It takes the outward horizontal thrust of the dome and converts it into hoop tension in itself, so the cylindrical wall below does not have to.

Cylindrical wall

The main storage barrel. It carries hoop tension from the water pressure, and it carries the weight of everything above it straight down into the cone. Thickness governed by the crack check, typically 150 to 300 mm.

Ring beam B_2

The ring at the wall–cone junction. The cone, being in compression, pushes it *outward*, and the water pushes outward too; B_2 is therefore in **hoop tension** and is usually the most heavily tensioned ring in the tank. Typically 600×350 mm.

Conical slab

The sloping shell from the girder up to the wall. It carries the wall, the top dome and the bulk of the water down to the girder in **meridional compression**, with hoop tension across it. Thickness 200 to 500 mm depending on the tank.

Circular girder B_3

The most heavily worked member in the tank. It sits on the columns at the bottom of the cone, collects the entire weight of the container and its contents, and spans between the columns in bending, shear and torsion. It also settles the Intze balance between the cone’s inward thrust and the bottom dome’s outward thrust. Typically 550×720 mm or larger.

Bottom dome

A shallow spherical cap closing the bottom of the container, *arching upward* from the girder. It carries the water above it to the girder in compression, and its bulge displaces water — so it reduces the stored volume rather than adding to it.

Staging

Columns on the circle of diameter $2r$, tied at intervals by bracing beams, carrying the container to the ground and resisting all the lateral load.

Foundation

An annular raft under the ring of columns, or isolated footings tied with plinth beams.

1.4 The load path, and why it is the key to everything

The single most common mistake in Intze design

Assuming the container hangs from the girder — that the conical slab and the bottom dome are in meridional tension. They are not. In the standard arrangement the girder is at the *bottom* of the cone, the container stands on it, and both lower shells are in **compression**. Getting this backwards reverses every horizontal thrust in the bottom half of the tank, turns the Intze balance into an Intze addition, and produces ring beams that are wrong by a factor of two or more.

Trace the load. The columns support the tank at the circular girder B_3 , on a circle of diameter $2r$ at the bottom of the conical slab. Everything *above* that level — the cone, the wall, both ring beams above, the top dome and all the water — stands on the girder and travels down to it in compression, exactly as a wall stands on a footing. The only thing at or below the girder level is the bottom dome, and it arches upward into the tank, so it too is a compression member. Nothing in the container hangs.

Statics of the conical slab

Cut the cone on a horizontal plane at any level and take the free body *above* the cut: the remaining cone, the ring beam B_2 , the wall, B_1 , the top dome, and all the water resting on the cone and standing in the cylinder above it. That free body has weight, and the only thing holding it up is the membrane force in the cone across the cut. That force must therefore act *upward* on the free body, which means the cone is in **meridional compression**:

$$N_\phi = \frac{W_{\text{above}}}{2\pi r_0 \cos \alpha} \quad (\text{compression}),$$

with r_0 the radius of the cut and α the slope of the cone from the horizontal.

The same argument applied to the bottom dome gives the same answer. The dome arches up; the water sits on top of it; it behaves as a shallow masonry arch turned into a circle, and it is in compression throughout, both meridionally and in hoop. This is not a subtlety and it is not optional; it is the vertical equilibrium of a free body, and there is nothing to argue with.

Why does this matter so much? Because the direction of a membrane force determines the direction in which it pushes on the ring beams at its ends, and that in turn determines whether a ring beam is in hoop tension or hoop compression. Figure 1.2 shows it.

A compressive membrane force pushes the ring away from the body of the shell. At the *top* of the cone (ring B_2) the shell lies below and inboard, so the cone pushes B_2 outward: B_2 gets hoop *tension*, and the water pressure at that level adds to it. At the *bottom* of the cone (girder B_3) the shell lies above and outboard, so the cone pushes B_3 inward: hoop *compression*. The arched bottom dome springs from B_3 going upward and inboard, so it pushes B_3 outward: hoop tension. The two effects at B_3 oppose, and the design aims to make them cancel.

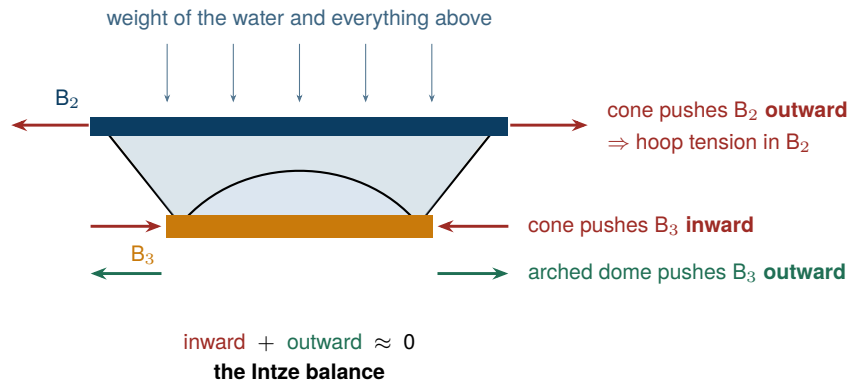


Figure 1.2: Because both lower shells stand rather than hang, their membrane forces are compressive, and a compressive force pushes its supporting ring *away* from the shell. At B_2 the cone pushes outward (hoop tension, added to by the water). At the girder B_3 the cone pushes inward and the arched dome pushes outward — the two oppose, and that is the Intze balance.

Summary of the load path

1. Rain, dust and access loads, plus its own weight $\rightarrow$ top dome $\rightarrow$ outward thrust $\rightarrow$ hoop tension in B_1 ; vertical component down the cylindrical wall.
2. Water pressure on the wall $\rightarrow$ hoop tension in the wall, carried entirely by the wall itself. A vertical wall carries no vertical water load.
3. Weight of the water $\rightarrow$ onto the conical slab and the bottom dome, the only surfaces with a horizontal projection. The bottom dome's upward bulge *displaces* water, so it reduces the stored volume.
4. Wall, top dome and the water above $\rightarrow$ ring beam B_2 (hoop tension) $\rightarrow$ conical slab in meridional compression $\rightarrow$ circular girder B_3 .
5. Bottom dome, in compression, also $\rightarrow$ girder B_3 , pushing outward against the cone's inward push. This is the Intze balance.
6. Girder $B_3 \rightarrow$ columns, in bending, shear and torsion between them.
7. Columns and bracings $\rightarrow$ foundation $\rightarrow$ soil.
8. Wind and earthquake act horizontally on the whole assembly and are resisted by the staging frame, appearing at the foundation as a shear and an overturning moment.

A one-line check you can apply to any Intze design

Add up the total weight of container plus water. Divide by $2\pi r$. That is roughly the meridional force per metre arriving at the girder, and it must be *compressive*. If your calculation has it tensile, you have put the columns in the wrong place.

1.5 The order in which the design is done

The design is not done in a single pass, because the force in the cone depends on the weight of everything above it — which includes the cone's own weight, which depends on its thickness, which depends on the force. There is a circularity, and it is resolved by iteration.

The practical order is:

1. Fix the geometry from the capacity (Chapter 4), remembering that the bottom dome's bulge *subtracts* from the stored volume. Assume trial thicknesses.

2. Compute all dead loads and the water load with those thicknesses (Chapter 5).
3. Compute wind (Chapter 6) and earthquake (Chapter 7). These do not affect the container design, only the staging and foundation, so they can be done in parallel.
4. Design the container downwards from the top: top dome, B_1 , wall (Chapters 8 and 9). Nothing here depends on what is below.
5. Design the conical slab and the ring beam B_2 (Chapter 11). The cone needs the weight of everything standing on it.
6. Design the bottom dome (Chapter 12) and check the Intze balance at the girder.
7. If any thickness has changed materially, go back to step 2. In practice one iteration is enough; two is plenty.
8. Design the circular girder B_3 (Chapter 10) for the residual hoop force together with the bending, shear and torsion from spanning between the columns.
9. Combine the loads, distribute to the columns, and design the staging (Chapters 13, 14).
10. Design the foundation (Chapter 15).

Why the software iterates

The *Intze Tank Designer PRO* calculator runs this loop up to four times when the thicknesses are set to **Auto**. Each pass recomputes the dead loads from the previous pass's thicknesses, re-derives the required thicknesses from the crack check, and stops when nothing increases. This is exactly the hand procedure above, done quickly.

1.6 Typical proportions

Before any arithmetic it helps to know what a reasonable Intze tank looks like, so that a result can be sanity-checked. Table 1.1 gives the ranges used in Indian practice. Chapter 4 explains where each comes from.

Table 1.1: Customary proportions for Intze tanks

Quantity	Usual range	Comment
H_1/D	0.30 to 0.50	cylindrical height to diameter
r/R	0.60 to 0.70	cone bottom radius to tank radius; also fixes the column circle
Column circle	$2r$	the girder and columns are at the cone bottom
Cone slope α	30° to 45° from horizontal	steeper drains better, flatter is cheaper
Top dome rise	$D/8$ to $D/6$	shallow cap
Bottom dome rise	$0.22r$ to $0.30r$	chosen to balance the thrusts at the girder
Freeboard	0.30 to 0.60 m	must exceed the sloshing height
Top dome thickness	100 to 120 mm	minimum steel governs
Wall thickness	150 to 300 mm	crack check governs
Cone thickness	200 to 500 mm	hoop crack check governs; meridian is in compression
Bottom dome thickness	150 to 250 mm	minimum steel usually governs
Number of columns	6 to 12	8 for 200 to 600 kL
Panel height between bracings	3 to 5 m	4 m is usual
Column size	$h_s/40$ to $h_s/30$	say 450 to 750 mm square
Staging height	12 to 25 m	set by the required residual head

On the bottom-dome rise

Older texts often quote $0.30r$ to $0.45r$ for the bottom dome rise. That range belongs to the *hanging* model and is too deep for the standing one. With the container standing on the girder the balancing rise works out at about $0.24r$ to $0.28r$ for normal proportions — and RDSO B-1235, an issued drawing, uses $0.245r$. Chapter 12 derives the exact value.

Codes, Design Philosophies and Permissible Stresses

2.1 Which code governs which member

An Intze tank is not designed to one code. Different parts of it are governed by different standards, and using the wrong one for a member is a common source of confusion.

Table 2.1: Codes and what each governs

Code	Governs
IS 3370 (Part 1) : 2009	General requirements for liquid-retaining concrete: minimum grade, cover, joints, water-tightness testing.
IS 3370 (Part 2) : 2009	Design of reinforced concrete liquid-retaining members by working stress, with the crack-control checks. Governs the top dome, wall, conical slab, bottom dome and the ring beams.
IS 3370 (Part 4) : 1967	Design tables and coefficients for walls and floors of cylindrical and rectangular tanks, including the moments caused by base fixity.
IS 456 : 2000	Everything not in contact with the liquid: the circular girder, the staging columns, the bracings and the foundation, by limit state.
IS 875 (Part 1)	Unit weights of materials — dead load.
IS 875 (Part 2)	Imposed loads — the access load on the top dome.
IS 875 (Part 3) : 2015	Wind loads.
IS 1893 (Part 1) : 2016	Seismic zones, zone factor Z , the design response spectrum S_a/g , and soil types.
IS 1893 (Part 2) : 2014	Seismic design of liquid-retaining tanks: the spring–mass model, importance factor, response reduction factor, hydrodynamic pressure, sloshing.
IS 11682 : 1985	Criteria for the design of the RCC staging of overhead water tanks.
IS 1904 : 1986, IS 6403 : 1981	Foundation design, settlement and bearing capacity.
IS 13920 : 2016	Ductile detailing of the staging in Zones III, IV and V.
SP 16 : 1980	Design aids — interaction charts for columns.

2.2 Two design philosophies in one structure

2.2.1 Why liquid-retaining members still use working stress

Limit state design replaced working stress for ordinary concrete in India in 1978, and IS 456 : 2000 is a limit state code. Yet IS 3370 (Part 2) : 2009 still designs the water-retaining parts of the tank by *working stress*, and it does so deliberately.

The reason is that a water tank has a serviceability requirement that an ordinary beam does not: it must not leak. A limit state design is a strength design — it asks whether the member will collapse, and it accepts a certain amount of cracking at working load as normal and harmless. For a water tank it is not harmless. A crack of 0.3 mm in a floor beam is invisible and irrelevant; the same crack through the wall of a tank is a leak, and a leak on the face of a 20 metre high structure is both a nuisance and the beginning of corrosion.

The design objective for liquid-retaining members

Not strength but **crack control**. The section is sized so that at full working load the tensile stress in the concrete, calculated on a transformed (uncracked) section, stays below a small permissible value — typically 1.3 to 1.7 N/mm². If the concrete is not permitted to reach its tensile capacity, it does not crack, and if it does not crack it does not leak.

This is why the thickness of an Intze wall is not governed by strength at all. A 150 mm wall would carry the hoop tension in the steel alone many times over. It is 150 mm because that is what keeps the concrete stress below 1.5 N/mm².

Where this goes wrong

Do not design the wall or the cone by limit state and then wonder why the answer is so much thinner than the textbooks. Limit state gives the collapse thickness, which is irrelevant here. Equally, do not design the columns or the girder by working stress — they are not in contact with the liquid and IS 456 limit state is correct for them.

2.2.2 The transformed section and the modular ratio

The crack check needs the stress in the concrete of a section that contains steel and is assumed uncracked. Both materials strain equally, so the steel carries m times the stress of the concrete at the same level, where

$$m = \frac{E_s}{E_c} = \frac{280}{3 \sigma_{cbc}} \quad (2.1)$$

[IS 3370-2] — the second form being the value the code prescribes rather than a measured ratio, σ_{cbc} being the permissible compressive stress in bending.

A steel area A_{st} is therefore replaced by an *equivalent* concrete area of $m A_{st}$, of which A_{st} is already counted as part of the gross concrete area, so the net addition is $(m - 1)A_{st}$.

Direct tension on a cracked-section check

For a member of thickness t per metre width carrying a direct tensile force F :

$$\sigma_{ct} = \frac{F}{1000 t + (m - 1)A_{st}} \leq \sigma_{ct,perm} \quad (2.2)$$

with F in newtons, t in millimetres, A_{st} in mm² per metre.

The steel area is fixed independently by strength, $A_{st} = F/\sigma_{st}$, so equation (2.2) can be inverted to give the thickness directly. Working per millimetre of width and remembering that a force in kN/m is numerically a force in N/mm:

Required thickness of a member in direct tension

$$t_{req} = F \left(\frac{1}{\sigma_{ct}} - \frac{m - 1}{\sigma_{st}} \right) \quad (2.3)$$

with F in kN/m and the stresses in N/mm², giving t in millimetres.

This one equation sizes the cylindrical wall, the conical slab and the bottom dome. It is worth memorising. For M30 concrete with Fe 500 steel at $\sigma_{st} = 150$, the bracket evaluates to $1/1.5 - 8.333/150 = 0.6667 - 0.0556 = 0.6111$, so the required thickness in millimetres is about 0.61 times the membrane

tension in kN/m — a useful rule of thumb for a first guess.

2.3 Permissible stresses

2.3.1 Concrete

IS 3370 (Part 2) : 2009 Table 1 gives the permissible stresses used in the resistance-to-cracking calculation. IS 3370 (Part 1) requires a minimum grade of M30 for liquid-retaining members, which is a change from the 1965 edition and is the reason older textbook examples in M20 look under-designed by today's rules.

Table 2.2: Permissible concrete stresses, IS 3370 (Part 2) : 2009, Table 1, with the modular ratio

Grade	Direct tension N/mm ²	Bending tension N/mm ²	σ_{cbc} N/mm ²	$m = 280/3\sigma_{cbc}$
M25	1.3	1.8	8.5	10.98
M30	1.5	2.0	10.0	9.33
M35	1.6	2.2	11.5	8.12
M40	1.7	2.4	13.0	7.18
M45	2.0	2.6	14.5	6.44
M50	2.1	2.7	16.0	5.83

2.3.2 Steel

Table 2.3: Permissible stresses in steel reinforcement, IS 3370 (Part 2) : 2009, Table 4

Type of stress	Mild steel N/mm ²	HYSD N/mm ²
Tensile stress in members under direct tension	115	150
Tensile stress in bending, on the liquid-retaining face	115	150
Tensile stress in bending, on the face away from the liquid, for members less than 225 mm thick	115	150
Tensile stress in bending, on the face away from the liquid, for members 225 mm thick and over	125	190
Tensile stress in shear reinforcement, members under 225 mm	115	150
Tensile stress in shear reinforcement, members 225 mm and over	125	175
Compressive stress in columns under direct load	125	175

Why the permissible steel stress is so low

Fe 500 steel yields at 500 N/mm² and a limit state design would work it at $0.87 \times 500 = 435$. Here it is held to 150. That is not timidity about the steel — it is crack control again. Crack width in a reinforced member is roughly proportional to the steel strain, so limiting the steel stress limits the crack width. Many designers adopt 130 N/mm² rather than 150 for the liquid face where tightness is critical; the software lets you set the value, and the difference feeds straight through equation (2.3) into the required thickness.

2.4 Minimum reinforcement — the surface zone rule

This changed substantially in the 2009 revision and it is easy to get wrong, because most textbook examples still use the 1965 rule.

IS 3370 (Part 2) : 2009, Clause 8.1

The minimum reinforcement in walls, floors and roofs, in each of two directions at right angles, shall not be less than **0.35 percent of the surface zone** for high yield strength deformed bars, or 0.64 percent for mild steel. For a tank in which *every* dimension is not more than 15 metres, these may be reduced to **0.24 percent** for deformed bars and 0.40 percent for plain round bars.

The phrase that matters is *surface zone*. The minimum is not a percentage of the gross section; it is a percentage of a zone measured from each face:

- For a section **less than 200 mm** thick, the whole section counts as one surface zone, and the calculated steel may all be placed in one face.
- For a section **200 mm or thicker**, there are two surface zones, each of thickness $t/2$, and the minimum applies to each — so steel is required near both faces.
- For a section thicker than **500 mm**, the surface zone is capped at 250 mm per face. Beyond that thickness the minimum stops growing, which is the point of the rule: the core of a thick member does not need crack-control steel, only the skin does.

Minimum steel per face

$$A_{st,min} = \frac{\rho_{min}}{100} z \times 1000 \text{ mm}^2 \text{ per metre, per face,} \quad z = \begin{cases} t & t < 200 \text{ mm} \\ \min(t/2, 250) & t \geq 200 \text{ mm} \end{cases} \quad (2.4)$$

with $\rho_{min} = 0.35$ (or 0.24 for a tank with all dimensions ≤ 15 m) for HYSD bars.

Worked comparison

A 300 mm conical slab in a tank 11 m across. Every dimension is under 15 m, so $\rho_{min} = 0.24$ percent. The section is over 200 mm, so there are two zones of $300/2 = 150$ mm:

$$A_{st,min} = \frac{0.24}{100} \times 150 \times 1000 = 360 \text{ mm}^2/\text{m per face,}$$

720 mm²/m in total, each way. Under the superseded 1965 rule the same member would have taken about 0.20 percent of the gross 300 mm, or 600 mm²/m in total — 17 percent less. For a large tank where the 0.35 percent applies, the 2009 rule gives 1050 mm²/m, nearly double.

2.5 Cover and durability

IS 3370 (Part 1) requires a clear cover of not less than **45 mm** on the liquid face, and not less than the IS 456 exposure requirement elsewhere — which for a structure of this kind is severe exposure, 45 mm. A larger cover is worse, not better, for crack control: crack width at the surface grows with cover, so 45 to 50 mm is the working range and 75 mm is not a conservative choice.

Minimum cement content and maximum water-cement ratio follow IS 456 Table 5 for severe exposure: 320 kg/m³ and 0.45 respectively, with M30 the minimum grade.

2.6 Limit state design of the non-liquid members

The girder, columns, bracings and foundation follow IS 456 : 2000 in the ordinary way. Two results are used repeatedly and are collected here.

Flexural steel, singly reinforced rectangular section

For a factored moment M_u on a section of width b and effective depth d ,

$$A_{st} = \frac{0.5f_{ck}}{f_y} \left[1 - \sqrt{1 - \frac{4.6M_u}{f_{ck}bd^2}} \right] bd \quad (2.5)$$

and the section is adequate as singly reinforced provided

$$M_u \leq M_{u,\text{lim}} = \begin{cases} 0.138 f_{ck}bd^2 & \text{Fe 415} \\ 0.133 f_{ck}bd^2 & \text{Fe 500} \end{cases} \quad (2.6)$$

Minimum and maximum flexural steel, IS 456 Cl. 26.5.1

$$A_{st,\text{min}} = 0.85 bd/f_y \quad \text{and} \quad A_{st,\text{max}} = 0.04 bD.$$

On $M_{u,\text{lim}}$

Equation (2.5) is used in this book with the Fe 415 coefficient 0.138 for both steel grades, which is very slightly unconservative for Fe 500. The margin is about 4 percent in $M_{u,\text{lim}}$ only — it does not affect the steel area, which comes from the quadratic itself — and every girder in a tank of this kind is far below the limit anyway. Where a section approaches $M_{u,\text{lim}}$ the right answer is a deeper section, not a doubly reinforced one.

Membrane Theory of Shells of Revolution

Every force in the container of an Intze tank comes out of the two equations derived in this chapter. They are not difficult, and knowing where they come from makes the rest of the book mechanical rather than mysterious.

3.1 What a membrane is, and when a shell behaves like one

Membrane state

A shell is in a *membrane state* when the internal forces are entirely in the plane of the surface — a force per unit length along the meridian and a force per unit length around the parallel — with no bending moment, no twisting moment and no transverse shear.

A real shell is never purely in a membrane state, but it is close to one over most of its area provided three conditions hold:

1. The surface is smooth and the thickness does not change abruptly.
2. The load is smooth and continuous — self weight, or a hydrostatic pressure. A point load or a line load cannot be carried by membrane action alone.
3. The edges are supported in a way that is compatible with the membrane deformations. The shell must be free to move as membrane action requires it to.

Condition 3 is the one that is always violated in a real tank, because the edges are cast monolithically into ring beams that restrain them. Near an edge, therefore, bending appears. The important fact — established by shell theory and confirmed by every test — is that this *edge disturbance decays very rapidly*. Its characteristic length is of the order of $\sqrt{Rt}$: for a shell of radius 5 m and thickness 0.2 m, $\sqrt{5 \times 0.2} = 1.0$ m, and the bending has effectively died out within two or three times that.

How the codes deal with the edge disturbance

By not computing it, and covering it instead with detailing. The membrane solution is used for the whole shell, and the junctions are given haunches, extra reinforcement, and generous laps so that the local bending — which is real — is carried by steel that is there anyway. Chapter 17 shows those details. This is standard, accepted practice; the alternative is a finite-element analysis, which for a routine tank is not worth the effort.

3.2 Geometry of a surface of revolution

Take a curve — the *meridian* — and rotate it about a vertical axis. At any point on the resulting surface there are two principal radii of curvature.

r_1 , the meridional radius

the radius of curvature of the meridian curve itself, in its own plane. For a sphere $r_1 = R_d$; for a cone and for a cylinder the meridian is a straight line, so $r_1 = \infty$.

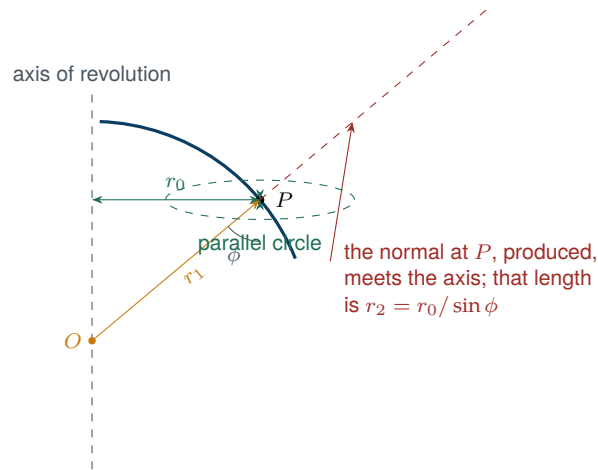


Figure 3.1: The two principal radii at a point on a surface of revolution. r_1 lies in the plane of the meridian; r_2 is measured along the normal to the axis, and equals $r_0 / \sin \phi$ where r_0 is the horizontal distance from the axis.

r_2 , the circumferential radius

the distance from the point to the axis *measured along the normal to the surface*. It is not the horizontal radius r_0 unless the surface is vertical there. In general $r_2 = r_0 / \sin \phi$, where ϕ is the angle between the normal and the axis of revolution.

For the three surfaces used in this book:

Surface	r_1	r_2	Note
Sphere of radius R_d	R_d	R_d	both equal
Cylinder of radius R	∞	R	meridian is a vertical straight line
Cone, generator at α to the vertical	∞	$r_0 / \cos \alpha$	r_0 is the horizontal radius

The cone's r_2 , which trips people up

For a cone whose generator makes an angle α with the vertical axis, the normal to the surface makes the angle α with the *horizontal*. The angle between that normal and the axis is therefore $\phi = 90^\circ - \alpha$, and

$$r_2 = \frac{r_0}{\sin \phi} = \frac{r_0}{\sin(90^\circ - \alpha)} = \frac{r_0}{\cos \alpha}.$$

For a 45° cone, $r_2 = 1.414 r_0$ — the effective radius for hoop action is 41 percent larger than the actual radius, which is why conical slabs carry so much more hoop tension than a cylinder of the same diameter and depth.

3.3 The two equilibrium equations

Isolate a small element of the shell bounded by two adjacent meridians and two adjacent parallels. Let N_ϕ be the membrane force per unit length along the meridian and N_θ the force per unit length around the parallel, both positive in tension. Let p_n be the load per unit area acting along the outward normal, positive outward.

3.3.1 Equilibrium normal to the surface

The meridional forces on the two curved edges of the element are inclined to each other by the small angle $d\phi_1 = ds_1/r_1$, so their resultant has a component along the inward normal of $N_\phi ds_2 ds_1/r_1$. Likewise the hoop forces on the other pair of edges give $N_\theta ds_1 ds_2/r_2$. The applied load on the element is $p_n ds_1 ds_2$ outward. Dividing through by $ds_1 ds_2$:

First membrane equation — the Laplace or “shell” equation

$$\frac{N_\phi}{r_1} + \frac{N_\theta}{r_2} = p_n \quad (3.1)$$

This is one equation in two unknowns, so a second is needed.

3.3.2 Vertical equilibrium of the cap above (or below) a parallel

Rather than differentiate again, it is far easier — and this is how it is always done in practice — to cut the shell on a horizontal plane at the parallel of radius r_0 and take the vertical equilibrium of the whole portion on one side of the cut.

Second membrane equation — global vertical equilibrium

$$N_\phi \times \sin \phi \times 2\pi r_0 = W \quad \Rightarrow \quad N_\phi = \frac{W}{2\pi r_0 \sin \phi} \quad (3.2)$$

where W is the total vertical load carried across the cut and ϕ is the angle between the normal to the shell and the axis — equivalently, $\sin \phi$ is the sine of the angle the meridian tangent makes with the horizontal.

Equation (3.2) does all the heavy lifting. Once N_ϕ is known, equation (3.1) gives N_θ immediately.

The sign of W decides tension or compression

Take W positive when the load is carried *downward past the cut* — that is, when the shell below the cut is being held up by the shell above. Then N_ϕ positive means tension. For a top dome, the cap above the cut pushes down on the shell below it, W is effectively negative in this convention, and N_ϕ comes out compressive; the usual practice is to write the top dome formula separately with compression positive, which is done in Chapter 8. The conical slab and the bottom dome of an Intze do *not* hang — they stand on the circular girder at the cone bottom — so for those two N_ϕ also comes out compressive, and Chapters 11 and 12 likewise write them with compression positive. The only tensile membrane force in an Intze container is the hoop force.

3.4 Application 1 — spherical dome under its own weight

Let the dome have radius R_d , a uniformly distributed load w per unit area of surface acting vertically downward, and let ϕ be measured from the crown.

The surface area of the cap from the crown to the parallel at ϕ is $2\pi R_d^2(1 - \cos \phi)$, so the load on it is $W = 2\pi R_d^2 w(1 - \cos \phi)$. The horizontal radius is $r_0 = R_d \sin \phi$, and the angle of the normal to the axis is also ϕ . Substituting into (3.2) and taking compression positive for this case:

$$N_\phi = \frac{2\pi R_d^2 w(1 - \cos \phi)}{2\pi R_d \sin \phi \cdot \sin \phi} = \frac{w R_d (1 - \cos \phi)}{\sin^2 \phi} = \frac{w R_d (1 - \cos \phi)}{(1 - \cos \phi)(1 + \cos \phi)}.$$

Spherical dome under self weight

$$N_\phi = \frac{wR_d}{1 + \cos \phi} \quad (\text{compression}) \quad (3.3)$$

$$N_\theta = wR_d \left(\cos \phi - \frac{1}{1 + \cos \phi} \right) \quad (3.4)$$

The second follows from (3.1) with $r_1 = r_2 = R_d$ and the normal component of the load $p_n = -w \cos \phi$.

Two things are worth noticing.

- N_ϕ is compressive everywhere and largest at the springing. At the crown ($\phi = 0$) it is $wR_d/2$; at a springing angle of 30° it is $wR_d/1.866 = 0.536 wR_d$.
- N_θ changes sign. It is compressive near the crown and becomes tensile where $\cos \phi = 1/(1 + \cos \phi)$, that is at $\phi = 51^\circ 49'$. A dome shallower than that — and all tank domes are — is in hoop compression throughout, which is why nominal steel suffices.

For a shallow cap of rise h_t over a base radius R , the sphere radius follows from the intersecting-chords theorem:

$$R_d = \frac{R^2 + h_t^2}{2h_t}, \quad \sin \phi_0 = \frac{R}{R_d} \quad (3.5)$$

where ϕ_0 is the springing angle.

3.5 Application 2 — spherical shell under normal pressure

If instead the load is a pressure p acting along the normal (as water pressure does), then with $r_1 = r_2 = R_d$ equation (3.1) gives $N_\phi + N_\theta = pR_d$; and by symmetry about the crown for a complete sphere, $N_\phi = N_\theta = pR_d/2$, the classical thin-pressure-vessel result.

For the bottom dome of a tank neither symmetry nor uniform pressure holds exactly, so the pair is used as follows: N_ϕ comes from vertical equilibrium (3.2) with the actual weight carried across the cut — for the upward-arching bottom dome of an Intze that is the water standing on it plus its own weight, and N_ϕ is compressive — and then

$$N_\theta = p_n R_d - N_\phi \quad (3.6)$$

which is simply (3.1) rearranged. This is how Chapter 12 works.

3.6 Application 3 — the cone

For a cone the meridian is straight, $r_1 = \infty$, and the first term of (3.1) vanishes. Therefore

$$N_\theta = p_n r_2 = \frac{p_n r_0}{\cos \alpha} \quad (3.7)$$

with α the angle of the generator to the vertical.

The normal pressure on the conical wall of a water tank has two parts: the water pressure p acting outward along the normal, and the component of the shell's own weight normal to the surface. A slab weight q per unit area of the inclined surface has a component $q \sin \alpha$ along the normal — directed inward for a cone that widens upward — and after resolving on the projected area the standard result is

Hoop tension in a conical shell of a tank

$$N_\theta = \left(\frac{p}{\cos \alpha} + q \tan \alpha \right) r_0 = \left(\frac{p}{\cos \alpha} + q \tan \alpha \right) \frac{d}{2} \quad (3.8)$$

where p is the water pressure at the level considered, q the self weight of the shell per unit area, α the angle of the generator with the vertical, and d the diameter at that level.

The meridional force comes from (3.2). For a cone the angle between the meridian tangent and the horizontal is $90^\circ - \alpha$, so $\sin \phi = \cos \alpha$, and

$$N_\phi = \frac{W}{2\pi r_0 \cos \alpha} \quad (3.9)$$

with W the weight transferred across the cut. For a cone that *hangs* from a ring above, W is the weight below the cut and N_ϕ is tensile; for the conical slab of an Intze, which *stands* on the girder at its lower edge, W is the weight of everything above the cut and N_ϕ is compressive. The formula is the same; only the free body changes.

Why the cone is the thickest member in the tank

Equation (3.8) contains $1/\cos \alpha$ and equation (3.9) contains $1/\cos \alpha$ as well. At $\alpha = 45^\circ$ both are multiplied by 1.414. The cone is also at the deepest water and carries the accumulated weight of the entire container above it. It is not unusual for the conical slab to be twice the thickness of the cylindrical wall above it — though what governs its thickness is the *hoop* tension, since the meridian is in compression.

3.7 Application 4 — the cylinder

For a vertical cylindrical wall the meridian is a vertical straight line, so $r_1 = \infty$ and $r_2 = R$. The water pressure at a depth y below the free surface is $p = \gamma_w y$ acting outward along the normal, so directly from (3.1):

Hoop tension in a cylindrical tank wall

$$N_\theta = \gamma_w y R \quad \text{per unit height, maximum at the base of the water} \quad (3.10)$$

And N_ϕ ? A vertical wall has $\sin \phi = 1$ and carries no vertical component of the water pressure at all — the pressure on a vertical surface is horizontal. The wall's meridional force is therefore just the weight of the top dome, the ring beam B_1 and the wall itself, transmitted straight down as compression. It never governs.

Summary of the membrane results used in this book

Element	N_ϕ	N_θ
Top dome	$wR_d/(1 + \cos \phi_0)$, compression	$wR_d[\cos \phi_0 - 1/(1 + \cos \phi_0)]$
Cylindrical wall	self weight above, compression	$\gamma_w y R$, tension
Conical dome	$W/(2\pi r_0 \cos \alpha)$, tension	$(p \sec \alpha + q \tan \alpha) r_0$, tension
Bottom dome	$W/(2\pi r_0 \sin \phi_0)$, tension	$p_n R_d - N_\phi$

3.8 From membrane force to ring force

One more small result closes the loop. A ring of radius R loaded by a uniform radial line load H per unit length of circumference carries a hoop force

Ring subjected to a uniform radial line load

$$P = H \times R \quad (3.11)$$

tension if H acts outward, compression if inward.

Cut the ring on a diameter and take the free body of one half. The radial load on an element subtending $d\theta$ is $HR d\theta$, whose component perpendicular to the cut is $HR \sin \theta d\theta$. Integrating from 0 to π gives $2HR$, resisted by the hoop force P at each of the two cut faces, so $P = HR$.

The radial line load H on a ring beam is the horizontal component of the membrane force of the shell that frames into it, plus any direct water pressure on the face of the beam itself. That is the whole of the ring beam calculation, and it is used four times in Chapters 8 to 12.

3.9 Summary — which force is which, in an Intze

Everything in this chapter reduces, for design purposes, to Table 3.1. It is worth committing to memory, because getting one sign wrong propagates through the whole design.

Table 3.1: Membrane forces in the shells of an Intze tank with the girder at the cone bottom

Element	Meridional N_ϕ	Hoop N_θ	Effect on the ring at its edge
Top dome	Compression $wR_d/(1 + \cos \phi)$	Compression (shallow cap)	Pushes B ₁ outward $\Rightarrow$ hoop tension in B ₁
Cylindrical wall	Compression (weight of dome + B ₁)	Tension $\gamma_w y R$	Passes its load down into the cone; no thrust of its own
Conical slab	Compression $W_{\text{above}}/(2\pi r_0 \cos \alpha)$	Tension $(p/\cos \alpha + q \tan \alpha)r_0$	Top edge pushes B ₂ outward (hoop tension); bottom edge pushes B ₃ inward
Bottom dome (arches up)	Compression $W/(2\pi r \sin \phi_0)$	Compression $p_n R_d - N_\phi$	Pushes B ₃ outward — opposing the cone. This is the Intze balance

The mnemonic

A shell in *tension* pulls its ring *towards* the shell. A shell in *compression* pushes its ring *away*. In an Intze every shell in the container is in meridional compression, so every ring is pushed away from the shell it carries. Decide which way “away” points and the sign follows without any algebra.

Proportioning the Container

4.1 What is given and what is chosen

The brief gives one number: the required storage, in kilolitres. Everything else is a choice, and the choices are constrained by custom, by drainage, by the bye-laws and by the balance condition of Chapter 12. This chapter turns the one number into a full set of dimensions.

Capacity

The stated capacity is the *net usable* volume of water, measured up to the top water level. It does not include the freeboard. A “450 kL tank” holds 450 m³ of water with the water surface at the top water level, and the structure is taller than that by the freeboard.

4.2 The volume equation

The water occupies three regions: a cylinder, a frustum of a cone, and a spherical segment bulging below the bottom ring.

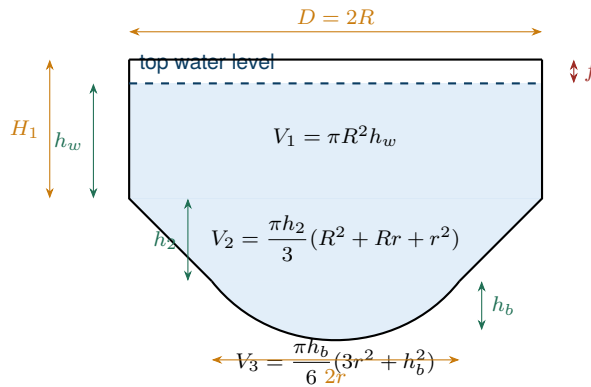


Figure 4.1: The three volumes of water in an Intze container.

Capacity of an Intze container

$$V = \underbrace{\pi R^2 h_w}_{\text{cylinder}} + \underbrace{\frac{\pi h_2}{3} (R^2 + Rr + r^2)}_{\text{conical frustum}} - \underbrace{\frac{\pi h_b}{6} (3r^2 + h_b^2)}_{\text{spherical segment, displaced}} \quad (4.1)$$

where $h_w = H_1 - f$ is the depth of water standing in the cylinder, f the freeboard, h_2 the height of the cone and h_b the rise of the bottom dome.

Where the three volume formulae come from

The frustum volume is the standard prismatoid result. The spherical segment of base radius r and height h_b has volume $\pi h_b (3r^2 + h_b^2)/6$. The bottom dome arches *upward* into the container, so

its bulge occupies space that would otherwise hold water: this volume is *subtracted*, not added. For a shallow dome the term is small — in the worked example it is 22 m³ out of 450 — but it is not negligible, and getting its *sign* wrong makes the tank 10 percent out, in the wrong direction. This is also the first place where the arrangement shows up in the arithmetic. If you find yourself adding the segment, stop and check where you have put the columns.

4.3 Choosing the ratios

Five ratios turn equation (4.1) into a single-variable problem.

4.3.1 H_1/D — how tall the cylinder is

Usual range 0.30 to 0.50. The trade-off is:

- A **taller, narrower** tank (larger H_1/D) has deeper water, so greater hoop tension in the wall and a thicker wall, but less surface area of top and bottom dome and a smaller circular girder because the girder moment goes with WR_c .
- A **squatter** tank has thin walls but large domes, a large girder, and — importantly for seismic — a larger diameter, which raises the sloshing wave height because $d_{\max} \propto D$.

0.30 is the common Indian default and is used in this book.

4.3.2 r/R — how far the cone slopes in

Usual range 0.60 to 0.70. This is the single most consequential choice in the whole geometry, because r is not only the bottom radius of the cone — it is also the radius of the circular girder and hence of the column circle. Changing r/R changes the staging, the bracing spans, the girder moment and the raft all at once.

- A **small** r/R (a deeply tapered cone) gives a long, steep cone carrying a very large meridional compression and a heavy hoop tension; it puts the girder at a much lower level, increasing the total height; and it shrinks the column circle until the staging becomes slender and the overturning stability suffers. Old textbooks that use $r/R = 0.3$ produce cones taller than the cylinder — geometrically valid, but not what is built.
- A small r/R also unbalances the tank: the narrower the girder circle, the larger the cone's inward thrust per metre of ring, and the deeper the bottom dome must be to match it.
- A **large** r/R approaching 1 degenerates towards a flat floor and loses the whole advantage of the shape.

0.66 is a good default and is used here; RDSO B-1235 uses 0.662, giving a column circle of 7.60 m under a tank of 11.48 m diameter.

4.3.3 Cone slope

30° to 45° from the horizontal. Steeper slopes drain and clean better and are usual where the tank is emptied regularly; 45° is the standard choice and makes $h_2 = R - r$ exactly, which is convenient. The slope fixes

$$h_2 = (R - r) \tan \theta_c, \quad L_s = \sqrt{(R - r)^2 + h_2^2} \quad (\text{slant length}) \quad (4.2)$$

with θ_c measured from the horizontal. The angle used in the membrane formulae is measured from the *vertical*, $\alpha = 90^\circ - \theta_c$.

4.3.4 Top dome rise

$D/8$ to $D/6$, that is $0.125D$ to $0.167D$. Shallower means less surface area and less concrete but a steeper rise in the meridional thrust, because $N_\phi = wR_d/(1 + \cos \phi_0)$ and R_d grows rapidly as the cap flattens. $0.15D$ is a sound middle choice.

4.3.5 Bottom dome rise

$0.22r$ to $0.30r$. Unlike the others, this one has a *correct* value rather than a customary range, and it is derived in Chapter 12: the rise should be whatever makes the cone's inward thrust and the arched dome's outward thrust cancel at the circular girder. Chapter 12 gives the closed form. In practice one starts at $0.25r$, completes the design, reads off the balancing value, and adjusts if it is far away.

Do not use the old $0.30r$ to $0.45r$ range

That range is quoted widely, and it belongs to the mistaken model in which the container hangs from a girder at the tank diameter. With the container standing on a girder at the cone bottom — the real arrangement — the balancing rise is shallower, typically $0.24r$ to $0.28r$. RDSO B-1235 uses $0.245r$. A dome built to $0.40r$ over-balances badly: it pushes the girder outward far harder than the cone pushes it in, and the girder ends up in substantial hoop tension for no reason.

4.3.6 The column circle

There is no separate ratio to choose: the columns stand under the circular girder, which is at the bottom of the cone, so the column circle is $2r$ and follows from k_r . Where a site constraint forces a different circle — an existing foundation, say — the girder is cantilevered or haunched out to it, and R_c then enters the girder moment $M \propto WR_c$ and the overturning arm independently of r . The software allows this as the Manual column circle option, and it should be used sparingly: it costs girder depth.

4.4 Solving for the diameter

With the ratios fixed, write everything in terms of R :

$$\begin{aligned} H_1 &= 2k_H R, & h_w &= 2k_H R - f, & r &= k_r R, \\ h_2 &= (1 - k_r)R \tan \theta_c, & h_b &= k_b k_r R, \end{aligned}$$

where $k_H = H_1/D$, $k_r = r/R$ and $k_b = h_b/r$. Substituting into (4.1) gives

$$V(R) = A R^3 - \pi f R^2 \quad (4.3)$$

with the constant

$$A = 2\pi k_H + \frac{\pi(1 - k_r) \tan \theta_c}{3} (1 + k_r + k_r^2) - \frac{\pi k_b k_r^3}{6} (3 + k_b^2). \quad (4.4)$$

Equation (4.3) is a cubic in R and could be solved in closed form, but nobody does. It is monotonically increasing for any sensible R , so a bisection converges in a dozen steps on a calculator and instantly in software.

The freeboard term matters

The $-\pi f R^2$ term in (4.3) is not a rounding detail. For the worked example $f = 0.5$ m and $\pi f R^2 = 49 \text{ m}^3$ — eleven percent of the capacity. Ignoring the freeboard when solving for R and

then adding it to H_1 afterwards gives a tank 11 percent too small.

Working the other way: diameter given

Where the plot, an existing foundation or a standard drawing fixes the diameter, the unknown becomes H_1 instead, and the solution is explicit — no iteration is needed:

$$h_w = \frac{V - V_2 + V_3}{\pi R^2}, \quad H_1 = h_w + f$$

with V_3 the displaced segment, which comes back with a plus sign here because it was subtracted in (4.1). The software offers this as the Manual geometry mode.

4.5 Trial thicknesses

The loads cannot be computed without thicknesses, and the thicknesses cannot be computed without the loads. Start with these and iterate once.

Table 4.1: Starting thicknesses

Element	First trial	Governed finally by
Top dome	100 mm	minimum steel; compressive stress never governs
Cylindrical wall	$\max(150, 30h_w + 50)$ mm	crack check, eq. (2.3)
Conical slab	200 to 300 mm	crack check on the <i>hoop</i> tension; the meridian is compressed
Bottom dome	150 to 200 mm	minimum steel, usually
Ring beam B_1	250×300	hoop tension from the top dome
Ring beam B_2	600×350	hoop tension from the cone and the water
Circular girder B_3	550×720	bending, shear and torsion between the columns, plus the residual unbalanced thrust

4.6 Surface areas, for the dead load

These are needed in Chapter 5 and are collected here because they are pure geometry.

Surface areas

$$\text{Top dome (spherical zone)} \quad A_{td} = 2\pi R_{d,t} h_t \quad (4.5)$$

$$\text{Cylindrical wall} \quad A_w = 2\pi R H_1 \quad (4.6)$$

$$\text{Conical dome (frustum)} \quad A_c = \pi(R + r)L_s \quad (4.7)$$

$$\text{Bottom dome (spherical zone)} \quad A_{bd} = 2\pi R_{d,b} h_b \quad (4.8)$$

with $R_{d,t} = (R^2 + h_t^2)/2h_t$ and $R_{d,b} = (r^2 + h_b^2)/2h_b$ from (3.5).

Archimedes' result, worth knowing

The area of a zone of a sphere is $2\pi R_d h$, depending only on the height of the zone and not at all on where on the sphere it is taken. That is why the dome area formula is so simple, and it is

also why the centroid of a spherical zone's *surface* lies exactly at its mid-height — a fact used in the next chapter for the centres of gravity.

4.7 Ring beam volumes

Each ring beam is a torus of small section; for the accuracy needed here the volume is simply the circumference times the section:

$$V_{B_1} = 2\pi R b_1 d_1, \quad V_{B_2} = 2\pi R b_2 d_2, \quad V_{B_3} = 2\pi r b_3 d_3, \quad (4.9)$$

B_1 and B_2 being at the tank radius R and the circular girder B_3 at the cone-bottom radius r .

4.8 Checking the result

Before going on, substitute the solved dimensions back into (4.1) and confirm the capacity. It is a two-line check that catches every arithmetic slip in this chapter, and the software prints it for the same reason.

Cross-check with the calculator

The calculator's Section 2 prints the solved R and D , then the line "Check: $V_1 + V_2 + V_3$ against the required capacity". If those two do not agree to within a few tenths of a cubic metre, something is wrong with the ratios — most often a freeboard larger than H_1 , which makes h_w negative.

Dead, Imposed and Water Loads

5.1 Unit weights

Material	Unit weight	Source
Reinforced concrete	25 kN/m ³	IS 875 (Part 1)
Plain concrete	24 kN/m ³	IS 875 (Part 1)
Water	9.81 kN/m ³	1000 kg/m ³ × <i>g</i>
Cement plaster	20.4 kN/m ³	IS 875 (Part 1)
Soil, moist	16 to 20 kN/m ³	site dependent

9.81 or 10?

Water weighs 9.81 kN/m³, and that is the figure IS 1893 (Part 2) uses. Some older texts round to 10, which is 1.9 percent conservative on every water-derived force in the tank. Either is defensible; be consistent, and say which you used. This book uses 9.81 throughout. A convenient consequence is that one cubic metre of water is one tonne exactly, which matters in Chapter 7 where the seismic masses are handled in tonnes.

5.2 Dead load of the container, element by element

The weight of each shell is its surface area times its thickness times the unit weight of concrete:

$$W = A \times \frac{t}{1000} \times \gamma_c \quad (5.1)$$

with A in m², t in mm and γ_c in kN/m³.

The ring beams are the circumference times the section area times γ_c .

5.2.1 The allowance for finishes and fittings

Add a percentage to cover what the geometry does not include: internal and external plaster, waterproofing, the access ladder and its cage, the handrail round the top dome, the manhole and its cover, the inlet, outlet, overflow and washout pipes with their puddle flanges, the water level indicator, the lightning protection, and the fillets and haunches at the junctions — which are real concrete and are not in any of the surface-area formulae.

Note

Eight percent of the shell weight is a reasonable allowance for a routine tank, and 10 to 12 percent where the plaster is thick or the pipework heavy. It is not a fudge factor: leave it out and the dead load is genuinely 8 percent light, which propagates into the seismic mass, the column loads and the foundation.

5.3 Imposed load

Only one imposed load matters: the access load on the top dome. IS 875 (Part 2) gives 1.5 kN/m^2 for a roof that people can walk on, and 0.75 kN/m^2 for one that is inaccessible except for maintenance. Take 1.5 kN/m^2 unless there is genuinely no access.

The imposed load enters the top dome design (Chapter 8) and the girder design (Chapter 10). It is *not* included in the seismic mass — IS 1893 does not require imposed load on a roof to be lumped for seismic purposes when it is less than 3 kN/m^2 .

5.4 Water load

$$W_T = V \times \gamma_w \quad (5.2)$$

where V is the water volume from equation (4.1) — the net capacity, not the gross volume of the container. For the worked example, $450.0 \times 9.81 = 4414.5 \text{ kN}$.

The water load dominates: in the worked example it is 4415 kN against 3815 kN of dead load, so more than half the vertical load on the foundation is the contents. That is worth remembering when thinking about the tank-empty load cases, which are not academic — an empty tank in an earthquake is a real and sometimes governing condition.

5.5 Staging dead load

$$W_{\text{columns}} = n \times b_c^2 \times h_s \times \gamma_c \quad (5.3)$$

$$W_{\text{bracings}} = n_{br} \times n \times L_{ch} \times b_{br} d_{br} \times \gamma_c \quad (5.4)$$

where n is the number of columns, b_c the column side, h_s the staging height, n_{br} the number of bracing levels, and L_{ch} the chord between adjacent columns:

$$L_{ch} = 2R_c \sin \frac{\pi}{n} \quad (5.5)$$

R_c being the radius of the column circle. For 8 columns on a 5.61 m radius, $L_{ch} = 2 \times 5.61 \times \sin 22.5^\circ = 4.29 \text{ m}$.

Number of bracing levels

Set by the panel height, which is kept between 3 and 5 metres — 4 m is usual. With n_{br} bracing levels there are $N_p = n_{br} + 1$ panels, each of height $h_p = h_s / N_p$. The panel height governs the column's effective length and, through N_p^2 , the lateral stiffness of the whole staging, so it is not a trivial choice; Chapter 7 shows how strongly.

5.6 Centres of gravity

The seismic calculation of Chapter 7 needs the height of the centre of gravity of the empty container and of the water. Both are found by the ordinary weighted average $\bar{z} = \sum W_i z_i / \sum W_i$, but the individual z_i need care.

Take the datum at the level of the circular girder B_3 — the bottom of the conical slab, which is also the top of the columns — positive upward. Every part of the container is at or above this level, so all the z values are positive. That is a useful check in itself: in the correct arrangement nothing in the container has a negative z .

Table 5.1: Centroid heights relative to the circular girder

Element	z	Reason
Top dome	$h_2 + H_1 + h_t/2$	the centroid of a spherical zone's surface is at its mid-height, exactly
Ring beam B ₁	$h_2 + H_1$	
Cylindrical wall	$h_2 + H_1/2$	
Ring beam B ₂	h_2	at the wall–cone junction
Conical slab	$\frac{h_2}{3} \cdot \frac{r + 2R}{r + R}$	centroid of a frustum's lateral surface, measured up from the <i>lower</i> (narrow) edge
Circular girder B ₃	0	the datum
Bottom dome	$h_b/2$	spherical zone again, springing at the datum and arching up

The frustum centroid flips

For a frustum of height h with radius R_1 at the bottom and R_2 at the top, the centroid of the lateral surface lies at $\frac{h}{3} \cdot \frac{R_1 + 2R_2}{R_1 + R_2}$ above the bottom edge. In an Intze the cone is *narrow at the bottom*, so $R_1 = r$ and $R_2 = R$, and the centroid sits above mid-height. Writing it the other way round — as one would for a cone hanging point-down — misplaces the container centre of gravity by a few hundred millimetres and shifts the whole seismic moment with it.

For the water, the corresponding centroids are of *volumes*, not surfaces:

Table 5.2: Centroid heights of the three water volumes, relative to the girder

Volume	z	Reason
Cylinder, $+V_1$	$h_2 + h_w/2$	
Frustum, $+V_2$	$\frac{h_2}{4} \cdot \frac{r^2 + 2rR + 3R^2}{r^2 + rR + R^2}$	centroid of a conical frustum volume, up from the narrow lower edge
Segment, $-V_3$	$\approx 0.375 h_b$	the <i>displaced</i> wedge under the arched dome; it enters the weighted mean with a negative weight

Centre of gravity of the water

$$\bar{z}_{\text{water}} = \frac{V_1 \left(h_2 + \frac{h_w}{2} \right) + V_2 \bar{z}_2 - V_3 (0.375 h_b)}{V_1 + V_2 - V_3} \quad (5.6)$$

The minus signs in both numerator and denominator are the bottom dome displacing water. Leaving them as plus signs raises the computed water centre of gravity by roughly 0.1 m and the seismic overturning moment with it.

Heights above the top of the foundation

If h_s is the staging height measured from the top of the foundation to the circular girder — that is, the clear column height — then any element at z relative to the girder is at $h_s + z$ above the foundation. In particular

$$h_{cg} = h_s + \bar{z}_{\text{container}} \quad (5.7)$$

is the height of the centre of gravity of the empty container, which IS 1893 (Part 2) calls h_{cg} and

uses directly in the overturning moment.

Defining the staging height

Three different heights are called “staging height” in practice: to the bottom of the container, to the girder, or to the top water level. They differ by several metres, and the seismic moment is proportional to height. This book uses **top of foundation to the top of the columns, that is to the circular girder**, and states the other levels explicitly wherever they are needed. When reading someone else’s calculation, find out which they meant before comparing numbers.

5.7 The structural mass for seismic purposes

IS 1893 (Part 2) Cl. 4.2.2.3 defines the structural mass of an elevated tank as

$$m_s = \frac{W_{\text{container}} + \frac{1}{3}W_{\text{staging}}}{g} \quad (5.8)$$

Why one third of the staging

The staging is a distributed mass along a cantilever, not a lump at the top. If a uniform cantilever of total mass M is replaced by an equivalent mass at its tip that gives the same fundamental period, the equivalent is $33/140 M \approx 0.236M$ for a pure flexural cantilever. For a framed staging with rigid bracings, deforming in shear rather than flexure, the equivalent concentration is one third — the same result as for a shear beam. The code adopts $1/3$, which is the shear-beam value and is appropriate for a braced frame.

The height of this combined structural mass, used in the overturning moment, is the weighted mean of the container centroid and the mid-height of the staging:

$$h_{s,\text{eff}} = \frac{W_{\text{container}} h_{cg} + \frac{1}{3}W_{\text{staging}} \cdot \frac{h_s}{2}}{W_{\text{container}} + \frac{1}{3}W_{\text{staging}}} \quad (5.9)$$

5.8 Order-of-magnitude checks

Before going further, apply these to whatever you have computed. They catch most gross errors.

Table 5.3: Sanity checks on the loads

Quantity	Typical value	Worked example
Container dead load / water load	0.35 to 0.55	$2100/4415 = 0.48$
Staging dead load / container dead load	0.7 to 1.2 for 20 m	$2023/2100 = 0.96$
Total vertical service load, tank full	—	8538 kN
Concrete volume per kilolitre stored	0.35 to 0.55 m ³ /kL	$218/450 = 0.48$
Container c.g. above the girder	0.30 to 0.40 of $(h_2 + H_1 + h_t)$	$2.61/7.53 = 0.35$
Water c.g. above the girder	0.35 to 0.45 of $(h_2 + H_1)$	$2.93/5.63 = 0.52$ (tank full)

Cross-check with the calculator

Section 3 of the calculator’s booklet prints the element-by-element dead load table with the surface area, the thickness, the weight and the centroid of each, then the container total, the staging

total and the water load. Every figure in Table 5.1 appears in the last column of that table, so the two can be compared line for line. All of them are positive; a negative entry means the datum has been taken at the wrong level.

Wind Analysis — IS 875 (Part 3) : 2015

6.1 The shape of the calculation

Wind pressure is not a single number applied to the whole structure. The procedure is:

1. Take the **basic wind speed** V_b for the location from the map in the code. This is a 3-second gust at 10 m height in terrain category 2, with a 50-year return period.
2. Modify it by four factors to get the **design wind speed** V_z at the height in question.
3. Convert to **wind pressure** $p_z = 0.6V_z^2$.
4. Modify the pressure by three more factors to get the **design wind pressure** p_d .
5. Multiply by a **force coefficient** C_f and the **effective projected area** A_e to get the force.

The container and the staging are treated separately because they are at different heights, have different shapes, and take completely different force coefficients.

The wind chain

$$V_z = V_b k_1 k_2 k_3 k_4, \quad p_z = 0.6V_z^2 \text{ (N/m}^2\text{)}, \quad p_d = K_d K_a K_c p_z, \quad F = C_f A_e p_d \quad (6.1)$$

6.2 The speed factors

6.2.1 k_1 — risk coefficient

Adjusts the 50-year return period to the design life of the structure. For general buildings and structures with a 50-year life, $k_1 = 1.0$. For structures of post-cyclone importance or a 100-year design life, k_1 is about 1.06 to 1.08 depending on the basic speed. For temporary structures with a 5-year life it drops to about 0.92.

An elevated service reservoir is part of a water supply system and is usually treated as important, so $k_1 = 1.07$ is defensible; many designers keep 1.0 and rely on the importance factor in the seismic case instead. Either is acceptable provided it is stated.

6.2.2 k_2 — terrain and height factor

This is the main variable. Wind speed increases with height above ground, and the rate depends on the roughness of the surrounding terrain. The 2015 revision removed the old structure-size classes A, B and C and gives a single value per terrain category.

The categories are: **1** open sea, flat treeless plains; **2** open ground with scattered obstructions up to 10 m — *the default for most Indian sites*; **3** closely spaced obstructions up to 10 m, towns and suburbs; **4** large city centres with obstructions over 25 m.

Interpolate linearly between the tabulated heights.

Table 6.1: k_2 , IS 875 (Part 3) : 2015, Table 2

Height (m)	Category 1	Category 2	Category 3	Category 4
10	1.05	1.00	0.91	0.80
15	1.09	1.05	0.97	0.80
20	1.12	1.07	1.01	0.80
30	1.15	1.12	1.06	0.97
50	1.20	1.17	1.12	1.10
100	1.26	1.24	1.20	1.20

6.2.3 k_3 — topography factor

1.0 for ground sloping less than about 3° , rising to 1.36 at the crest of a steep hill or escarpment. The code gives a full procedure; for a tank on ordinary flat ground, $k_3 = 1.0$.

6.2.4 k_4 — importance factor for cyclonic regions

Introduced in the 2015 revision, this applies only along the east coast and in Gujarat: 1.30 for structures of post-cyclone importance such as hospitals and *water supply installations*, 1.15 for industrial structures, and 1.00 elsewhere. A tank on the West Bengal coast should take $k_4 = 1.30$; one inland takes 1.00.

6.3 The pressure factors

6.3.1 K_d — wind directionality factor

0.90 for buildings and structures generally, but **1.00 for circular and near-circular shapes**. The reduction exists because the worst wind direction and the worst structural direction rarely coincide; a circular container has no preferred direction, so it gets no reduction. The staging frame, being an assembly of rectangular members, takes 0.90.

6.3.2 K_a — area averaging factor

Large areas do not see the peak gust over their whole surface at the same instant.

Tributary area (m ²)	≤ 10	25	≥ 100
K_a	1.00	0.90	0.80

Interpolate between these.

6.3.3 K_c — combination factor

0.90 where wind acts simultaneously on several surfaces of a building in a combination the code recognises. For a free-standing tank, $K_c = 1.0$.

6.4 Force coefficients

6.4.1 The container — a circular cylinder

C_f for a circular section depends on the height-to-diameter ratio and on the surface roughness. For a rough concrete surface at the Reynolds numbers of interest ($DV_z > 6 \text{ m}^2/\text{s}$, which is always true for a tank):

h/D	1	2	5	10	20	40
C_f	0.7	0.8	0.8	0.9	0.9	1.0

An Intze container is squat — h/D is typically 0.4 to 0.6 — so $C_f = 0.7$ throughout.

Why 0.7 and not 1.2

A flat plate normal to the wind has $C_f \approx 1.2$ to 2.0. A circular cylinder is far lower because the flow separates late and the wake is narrow. This is a genuine and substantial saving: a cylindrical container attracts barely half the wind force of a rectangular one of the same projected area.

The projected area of the container is the rectangle $D \times (H_1 + h_t)$. The conical bottom, which also catches wind, has a trapezoidal projection $\frac{1}{2}(D + 2r)h_2$ and is taken with the same C_f . Both forces act at the centroid of their own projected area.

6.4.2 The staging — a framed structure

This is the harder half and it is where most of the wind force actually comes from. The staging is an open frame: the wind sees the windward columns and bracings, and then a second set behind them which is partly sheltered.

Solidity ratio

$$\varphi = \frac{\text{projected area of the members of one frame}}{\text{area of the enclosing envelope}} = \frac{A_s}{2R_c h_s} \quad (6.2)$$

For a circular staging, the “one frame” area is taken as roughly half the columns at full width plus the bracings across the diameter:

$$A_s = \frac{n}{2} b_c h_s + n_{br} (2R_c) d_{br} \quad (6.3)$$

C_f for a single frame of flat-sided members is then read against φ :

φ	0.1	0.2	0.3	0.5	0.7	1.0
C_f	1.9	1.8	1.7	1.6	1.6	2.0

The force on the windward frame is $C_f A_s p_d$. The leeward frame, standing in the wake of the first, takes a reduced force; the reduction is expressed as a shielding factor η , and the total is

Force on the staging

$$F_{\text{staging}} = C_f A_s p_d (1 + \eta) \quad (6.4)$$

with η between 0.3 and 0.6 depending on the spacing-to-width ratio of the frames. 0.45 is a reasonable default for a tank staging.

Where this goes wrong

Two errors are common here. The first is to project *all* n columns at full width with no shielding at all, which overestimates the staging force by roughly 60 percent. The second is the opposite — to take only two columns, as though the staging were a plane frame — which underestimates it by about the same. The half-plus-shielding treatment above lands between them and is what the code’s frame provisions actually intend.

6.5 Assembling the total

Base shear and overturning moment from wind

$$V_{\text{wind}} = F_{\text{container}} + F_{\text{cone}} + F_{\text{staging}} \quad (6.5)$$

$$M_{\text{wind}} = F_{\text{container}} z_{\text{cont}} + F_{\text{cone}} z_{\text{cone}} + F_{\text{staging}} \frac{h_s}{2} \quad (6.6)$$

with each z measured from the top of the foundation to the centroid of the corresponding projected area. The staging force, being on a nearly uniform strip, acts at mid-height.

6.6 What to expect

For an Indian tank of ordinary size the staging contributes two thirds to three quarters of the total wind force, despite being an open frame, simply because it is tall and its force coefficient is more than double the container's. The container's contribution is modest.

Wind almost always *loses* to earthquake in Zones III, IV and V for tanks of this kind, because the seismic mass is dominated by the water and the water is heavy. It can govern in Zone II, on a very exposed coastal site, or for a tall slender staging. Both must be computed; the software prints which one won and by how much.

Cross-check with the calculator

Section 4 of the booklet prints, in order: V_z and p_z for the container and for the staging; the projected areas, K_a , C_f and p_d ; the three forces with their lever arms; and the totals V_{wind} and M_{wind} . The solidity ratio and the shielding factor are printed explicitly so the frame treatment can be audited.

Seismic Analysis — IS 1893 (Part 2) : 2014

7.1 Why a tank is not a building

Put an accelerogram under an ordinary building and the whole mass moves more or less with the structure. Put one under a water tank and something else happens: the water near the base is dragged along with the walls, but the water near the free surface is not. It sloshes, with a period of several seconds — far longer than anything structural — and it carries a large part of the total mass out of the short-period response altogether.

Treating the whole water mass as rigidly attached, which is what a naive single-mass analysis does, is therefore wrong in both directions at once. It overestimates the short-period force, because part of the water is not participating at that period. And it hides the sloshing entirely, so the freeboard is never checked and the roof gets damaged.

Housner's idealisation

G. W. Housner showed in 1963 that the hydrodynamic behaviour of a tank can be represented to good accuracy by *two* lumped masses:

- an **impulsive mass** m_i , rigidly attached to the tank wall at a height h_i , moving with the tank; and
- a **convective mass** m_c , attached to the wall by springs at a height h_c , free to oscillate at its own long period.

IS 1893 (Part 2) : 2014 adopts this model in Clause 4.2, and everything in this chapter follows from it.

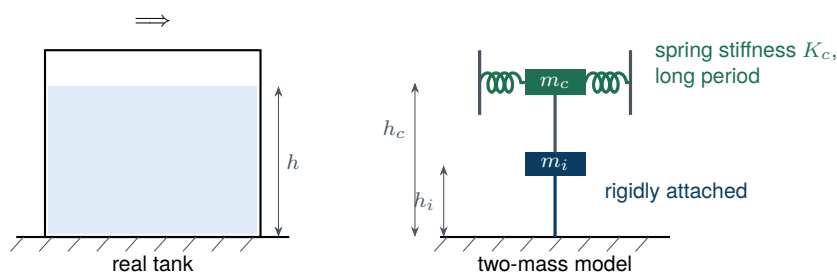


Figure 7.1: Housner's two-mass idealisation, as adopted by IS 1893 (Part 2).

7.2 The spring–mass parameters for a circular tank

The code gives these as graphs; the underlying expressions are collected here. Everything depends on the single ratio h/D , the depth of liquid to the tank diameter.

Masses — IS 1893-2 Cl. 4.2.1.1 and Table 1

$$\frac{m_i}{m} = \frac{\tanh(0.866 D/h)}{0.866 D/h} \quad (7.1)$$

$$\frac{m_c}{m} = 0.23 \frac{D}{h} \tanh\left(3.68 \frac{h}{D}\right) \quad (7.2)$$

where m is the total mass of the liquid.

Heights of the masses

For the moment at the *bottom of the tank wall* (excluding the pressure on the base):

$$\frac{h_i}{h} = \begin{cases} 0.375 & h/D \leq 0.75 \\ 0.5 - \frac{0.09375}{h/D} & h/D > 0.75 \end{cases} \quad (7.3)$$

$$\frac{h_c}{h} = 1 - \frac{\cosh(3.68 h/D) - 1}{(3.68 h/D) \sinh(3.68 h/D)} \quad (7.4)$$

For the *overturning* moment, which includes the moment of the pressure on the tank base:

$$\frac{h_i^*}{h} = \begin{cases} \frac{0.866 D/h}{2 \tanh(0.866 D/h)} - 0.125 & h/D \leq 1.33 \\ 0.45 & h/D > 1.33 \end{cases} \quad (7.5)$$

$$\frac{h_c^*}{h} = 1 - \frac{\cosh(3.68 h/D) - 2.01}{(3.68 h/D) \sinh(3.68 h/D)} \quad (7.6)$$

 h_i or h_i^* ? This is the error most often made

For an **elevated** tank, the moment that the staging and the foundation must carry is the *overturning* moment, and IS 1893 (Part 2) Cl. 4.7.2 requires h_i^* and h_c^* for it. They are not small corrections. For the worked example, $h_i/h = 0.375$ but $h_i^*/h = 0.972$ — nearly three times as high — and the overturning moment computed with the starred values is about 7 percent greater overall. The starred heights also make h_c^* exceed h_c , though by less.

The unstarred h_i and h_c are for the bending moment at the bottom of the tank *wall*, which matters for a ground-supported tank and for the wall design of a tall tank. For the staging of an elevated tank they are the wrong quantity.

Why h_i^* can exceed the water depth

It does, for shallow tanks, and it is not an error. The starred height is not the physical location of anything — it is the lever arm that reproduces the correct total moment when multiplied by the impulsive force. That total includes the moment of the hydrodynamic pressure acting on the tank *base*, which for a wide shallow tank is large. Expressing it as an equivalent force at the wall requires a lever arm above the water surface. For $h/D = 0.406$ in the worked example, $h_i^* = 0.97h$; at $h/D = 0.2$ it would be about $2h$.

Notice two patterns. First, $m_i + m_c$ stays within a few percent of m across the whole range but is not exactly equal to it — the idealisation does not conserve mass, and the code accepts that. Second, for

Table 7.1: Spring–mass parameters at a glance

h/D	m_i/m	m_c/m	h_i/h	h_c/h	h_i^*/h	h_c^*/h
0.2	0.231	0.721	0.375	0.521	2.041	2.228
0.3	0.344	0.615	0.375	0.545	1.327	1.227
0.4	0.450	0.517	0.375	0.574	0.986	0.907
0.5	0.542	0.437	0.375	0.605	0.797	0.784
0.75	0.710	0.304	0.375	0.681	0.580	0.727
1.0	0.808	0.230	0.406	0.742	0.494	0.755
1.5	0.902	0.153	0.438	0.820	0.450	0.822

a *shallow* tank the convective mass dominates, while for a *deep* tank the impulsive mass does. Intze tanks are shallow, with h/D around 0.4, so roughly half the water sloshes.

7.3 The equivalent cylinder — the clause that makes this apply to an Intze

Every formula above is for a plain circular cylinder of uniform depth h . An Intze container is nothing of the kind. The code resolves this in one sentence:

IS 1893 (Part 2) Cl. 4.2.3, quoted

“For tank shapes other than circular and rectangular (like intze, truncated conical shape), the value of h/D shall correspond to that of an equivalent circular tank of same volume and diameter equal to diameter of tank at top level of liquid.”

So: keep the real diameter D , and replace the real depth by the depth a plain cylinder of that diameter would need to hold the same volume:

Equivalent liquid depth

$$h = \frac{V}{\pi D^2/4} \quad (7.7)$$

For the worked example, $450/(\pi \times 11.219^2/4) = 4.552$ m against an actual liquid depth of $h_w + h_2 + h_b = 2.87 + 2.24 + 1.18 = 6.29$ m from the crown of the bottom dome. The equivalent cylinder is shallower because the real container narrows towards the bottom.

Where to put the base of the equivalent cylinder

The code does not say, and it matters for the lever arms. The defensible choice, used in this book and in the software, is to place the base of the equivalent cylinder at the *lowest point of the real liquid* — the crown of the bottom dome — so that h_i , h_c , h_i^* and h_c^* are all measured upward from there. State the assumption in the calculation; a checker will look for it.

7.4 Lateral stiffness of the staging

IS 1893 (Part 2) Cl. 4.3.1.3

K_s is “the lateral stiffness of the staging — the horizontal force required to be applied at the centre of gravity of the tank to cause a corresponding unit horizontal displacement.”

For a frame staging with bracings that are stiff compared with the columns, the deformation is a shear mode: each panel racks, and the total deflection is the sum of the panel deflections.

Within one panel of height h_p , each column is bent in double curvature with points of contraflexure at mid-height, so its stiffness is $12EI/h_p^3$. With n columns acting in parallel, the panel stiffness is

$$k_p = \frac{12 n E_c I_{\text{col}}}{h_p^3}.$$

The N_p panels are in series, so

$$\frac{1}{K_s} = \frac{N_p}{k_p} \quad \Rightarrow \quad K_s = \frac{12 n E_c I_{\text{col}}}{N_p h_p^3}.$$

Substituting $h_p = h_s/N_p$ gives the form that is easiest to use.

Lateral stiffness of a braced frame staging

$$K_s = \beta \frac{12 n E_c I_{\text{col}} N_p^2}{h_s^3} \quad (7.8)$$

with $E_c = 5000\sqrt{f_{ck}}$ N/mm² (IS 456 Cl. 6.2.3.1), $I_{\text{col}} = b_c^4/12$ for a square column, n the number of columns, N_p the number of panels, h_s the staging height, and β a reduction factor for the flexibility of the bracings.

The N_p^2 is not a misprint, and it is the reason bracing matters

Doubling the number of bracing levels roughly quadruples the staging stiffness, halves the time period, and — because the design spectrum is falling as $1/T$ in this range — roughly doubles the seismic force. Adding bracing to an elevated tank makes it *attract more earthquake*, not less. The bracings are there to control drift and to give the columns a sensible effective length, not to reduce the seismic demand.

7.4.1 The bracing flexibility factor β

Equation (7.8) assumes the bracings are rigid. Real bracings bend, the joints rotate, and the true stiffness is lower — typically 80 to 90 percent of the rigid-brace value for a conventionally proportioned staging. Taking $\beta = 0.85$ is reasonable; $\beta = 1.0$ gives the upper bound and is unconservative for the period, though conservative for the drift.

When to abandon the hand formula

For a tall, lightly braced staging, or one with unequal panel heights, or where the bracings are shallow relative to the columns, equation (7.8) is not good enough. Model the frame, apply a unit horizontal load at the container’s centre of gravity, read the deflection, and take $K_s = 1/\delta$ — which is exactly what the code’s definition says to do.

7.5 Time periods

Impulsive period, elevated tank — Cl. 4.3.1.3

$$T_i = 2\pi \sqrt{\frac{m_i + m_s}{K_s}} \quad (7.9)$$

with the masses in tonnes and K_s in kN/m, giving T_i in seconds.

Convective period — Cl. 4.3.2

$$T_c = C_c \sqrt{\frac{D}{g}}, \quad C_c = \frac{2\pi}{\sqrt{3.68 \tanh(3.68 h/D)}} \quad \Rightarrow \quad T_c = 2\pi \sqrt{\frac{D}{3.68 g \tanh(3.68 h/D)}} \quad (7.10)$$

Two observations. The convective period depends only on the tank diameter and the depth ratio — not on the staging, not on the concrete, not on anything structural. And it is long: 3 to 5 seconds for tanks of ordinary size, against 0.4 to 0.9 seconds for the impulsive mode. The two modes are so widely separated that they can be treated as uncoupled, which is exactly what the code does.

Units in equation (7.9)

$T = 2\pi \sqrt{m/K}$ needs consistent units. With m in kilograms and K in newtons per metre it works; but masses here are naturally in tonnes and stiffnesses in kN/m, and since both are a thousand times the base unit the ratio is unchanged. So m in tonnes over K in kN/m gives the period in seconds directly, with no conversion. Use that and you will not go wrong.

7.6 Design horizontal seismic coefficient

IS 1893 (Part 2) Cl. 4.5

$$A_h = \frac{Z}{2} \cdot \frac{I}{R} \cdot \frac{S_a}{g} \quad (7.11)$$

evaluated separately for each mode, with the appropriate period and damping.

Z , zone factor

From IS 1893 (Part 1) Table 3: Zone II 0.10, Zone III 0.16, Zone IV 0.24, Zone V 0.36. Kolkata and most of West Bengal are Zone III; the northern districts and Sikkim are Zone IV.

I , importance factor

IS 1893 (Part 2) Table 1: **1.5** for tanks storing drinking water, non-volatile material or low-inflammable petrochemicals, tanks for emergency services such as fire fighting, and tanks of post-earthquake importance; **1.0** for all other tanks with no risk to life and negligible consequences. A municipal service reservoir is 1.5.

R , response reduction factor

IS 1893 (Part 2) Table 2: **2.5** for an elevated tank on an RC frame staging, 1.8 for an RC shaft, 2.5 for a steel frame. These are far lower than the 3 to 5 used for buildings, because a tank staging has very little redundancy and very little ductility — there is nowhere for a hinge to form without the tank coming down.

S_a/g , spectral acceleration

From IS 1893 (Part 1) Figure 2, for the relevant period, soil type and damping.

7.6.1 The design spectrum

For 5 percent damping:

Design acceleration spectrum, IS 1893 (Part 1)

$$\text{Rock / hard soil (I): } \frac{S_a}{g} = \begin{cases} 1 + 15T & 0 \leq T < 0.10 \\ 2.50 & 0.10 \leq T \leq 0.40 \\ 1.00/T & 0.40 < T \leq 4.00 \\ 0.25 & T > 4.00 \end{cases} \quad (7.12)$$

$$\text{Medium soil (II): } \frac{S_a}{g} = \begin{cases} 1 + 15T & 0 \leq T < 0.10 \\ 2.50 & 0.10 \leq T \leq 0.55 \\ 1.36/T & 0.55 < T \leq 4.00 \\ 0.34 & T > 4.00 \end{cases} \quad (7.13)$$

$$\text{Soft soil (III): } \frac{S_a}{g} = \begin{cases} 1 + 15T & 0 \leq T < 0.10 \\ 2.50 & 0.10 \leq T \leq 0.67 \\ 1.67/T & 0.67 < T \leq 4.00 \\ 0.42 & T > 4.00 \end{cases} \quad (7.14)$$

7.6.2 Damping — and the factor 1.75

IS 1893 (Part 2) Cl. 4.4 requires **5 percent** damping for the impulsive mode of a concrete tank and **0.5 percent** for the convective mode. Sloshing water has almost no damping; there is nothing in a smooth tank to dissipate the energy of the wave.

The spectrum above is for 5 percent. For other damping it is scaled by the multiplying factors of IS 1893 (Part 1) Table 3, and Cl. 4.5.4 of Part 2 states that the factor for 0.5 percent damping shall be taken as **1.75**.

The two coefficients in practice

$$(A_h)_i = \frac{Z}{2} \cdot \frac{I}{R} \cdot \frac{S_a}{g} \Big|_{T_i, 5\%} \quad (7.15)$$

$$(A_h)_c = \frac{Z}{2} \cdot \frac{I}{R} \cdot \frac{S_a}{g} \Big|_{T_c, 5\%} \times 1.75 \quad (7.16)$$

Because T_c is long, S_a/g for the convective mode is small — typically 0.3 to 0.4 — and even after the 1.75 multiplier the convective coefficient comes out at a quarter to a third of the impulsive one. The convective mass is large but it is very lightly excited.

7.7 Base shear and overturning moment

Base shear, elevated tank — Cl. 4.6.2 and 4.6.3

$$V_i = (A_h)_i (m_i + m_s) g \quad (7.17)$$

$$V_c = (A_h)_c m_c g \quad (7.18)$$

$$V = \sqrt{V_i^2 + V_c^2} \quad (7.19)$$

Overturning moment at the top of the foundation — Cl. 4.7.2 and 4.7.3

$$M_i = (A_h)_i [m_i (h_i^* + h_s) + m_s h_{cg}] g \quad (7.20)$$

$$M_c = (A_h)_c m_c (h_c^* + h_s) g \quad (7.21)$$

$$M^* = \sqrt{M_i^2 + M_c^2} \quad (7.22)$$

where h_s is measured from the top of the foundation to the bottom of the (equivalent) tank wall and h_{cg} is the height of the centre of gravity of the empty container.

Why square-root-of-sum-of-squares and not simple addition

The two modes have very different periods, so their peak responses do not occur at the same instant. Adding them arithmetically would assume they do, which is unrealistically severe. SRSS is the standard modal combination for well-separated modes, and with T_c/T_i of order 6 the separation could hardly be better. In this case it barely matters numerically — V_c is small enough that SRSS and simple addition differ by a few percent — but the principle is right.

7.8 The tank-empty case

An empty tank is lighter, so the force is smaller — but the impulsive period is also shorter, which pushes the structure up the spectrum towards the plateau, and the response reduction factor and importance factor are unchanged. The net effect is a base shear perhaps 60 percent of the full case, but with *less* vertical load resisting overturning. For uplift on the windward column, the empty case very often governs.

Tank empty

$$T_{i,\text{empty}} = 2\pi \sqrt{\frac{m_s}{K_s}}, \quad V_{\text{empty}} = (A_h)_{i,\text{empty}} m_s g, \quad M_{\text{empty}} = V_{\text{empty}} \times h_{s,\text{eff}} \quad (7.23)$$

Both cases must be carried into the load combinations of Chapter 13.

7.9 Sloshing wave height and freeboard

IS 1893 (Part 2) Cl. 4.11

$$d_{\max} = (A_h)_c R \frac{D}{2} \quad (7.24)$$

where R is the response reduction factor — appearing in the numerator this time, which is not a misprint.

Why R multiplies rather than divides here

$(A_h)_c$ already contains $1/R$, because it is a *design* coefficient for strength. But the freeboard is not a strength check — the water does not know that the designer has accepted inelastic behaviour in the staging, and it will slosh to its full elastic amplitude regardless. Multiplying by R removes the reduction and restores the unreduced elastic response, which is the correct basis for a serviceability check on the freeboard.

If $d_{\max}$ exceeds the freeboard, the options are to increase the freeboard, which means a taller wall and more concrete; or to accept the wave striking the roof and design the top dome and the wall–dome junction for the resulting pressure. For a routine tank, increase the freeboard.

Note

The traditional Indian freeboard of 0.30 m fails this check for many tanks of ordinary size in Zone III and above. It is not a conservative habit carried over from the past; it predates the sloshing requirement. Check it every time.

7.10 Hydrodynamic pressure on the wall

The two masses also imply a pressure distribution on the wall, given in Cl. 4.9.

Impulsive and convective wall pressure, circular tank

$$p_{iw}(y) = Q_{iw}(y) (A_h)_i \rho g h \cos \phi, \quad Q_{iw}(y) = 0.866 \left[1 - \left(\frac{y}{h} \right)^2 \right] \tanh \left(0.866 \frac{D}{h} \right) \quad (7.25)$$

$$p_{cw}(y) = Q_{cw}(y) (A_h)_c \rho g D \left[1 - \frac{1}{3} \cos^2 \phi \right] \cos \phi, \quad Q_{cw}(y) = 0.5625 \frac{\cosh(3.674 y/D)}{\cosh(3.674 h/D)} \quad (7.26)$$

with y measured up from the base of the liquid and ϕ the circumferential angle from the direction of shaking.

The impulsive pressure is greatest at the base and zero at the surface; the convective pressure is greatest at the surface and small at the base. They are combined with the hydrostatic pressure and with the effect of vertical acceleration, and the result is a modest increase — usually 10 to 25 percent — in the hoop tension used to design the wall.

What this book and the software do, and do not, do with it

The hydrodynamic pressures are computed and reported, but they are *not* added to the hoop tension that sizes the wall steel in Chapter 9. That is a deliberate scope limit, consistent with common Indian practice for tanks of this size, and it is stated in the output so that nobody assumes otherwise. For a large or important tank, add them: compute the combined pressure at the wall base, multiply by R to convert to hoop force, and repeat the crack check of Chapter 9 with the increased value.

7.11 Vertical ground acceleration

Cl. 4.10 requires the hydrodynamic pressure from vertical acceleration to be considered, with $(A_v) = \frac{2}{3}A_h$ applied to the hydrostatic pressure. It increases the effective unit weight of the water during the earthquake. For an elevated tank it is a secondary effect and is normally absorbed within the margins of the crack check; it is noted here for completeness.

Cross-check with the calculator

Section 5 of the booklet reproduces this chapter step by step: the equivalent cylinder and h/D ; m_i , m_c and their fractions of m ; h_i , h_c , h_i^* and h_c^* with both pairs printed and labelled; E_c , I_{col} , the panel count and K_s ; T_i , $T_{i,empty}$ and T_c ; both S_a/g values and both A_h values with the 1.75 factor shown; V_i , V_c and the SRSS; M_i , M_c and M^* ; the sloshing height against the freeboard; and the two wall pressures.

Top Dome and Ring Beam B₁

8.1 Top dome

The top dome is the easiest member in the tank. It carries almost nothing, its stresses are tiny, and its thickness is set by practicality — you cannot place two layers of bar and 45 mm of cover in less than about 100 mm — rather than by any calculation. It is designed anyway, because the horizontal thrust it delivers to the ring beam B₁ is real and must be carried.

8.1.1 Geometry

A shallow spherical cap of rise h_t over a base radius R :

$$R_{d,t} = \frac{R^2 + h_t^2}{2h_t}, \quad \sin \phi_0 = \frac{R}{R_{d,t}}, \quad \cos \phi_0 = \sqrt{1 - \sin^2 \phi_0}, \quad A_{td} = 2\pi R_{d,t} h_t \quad (8.1)$$

8.1.2 Loads

$$w = \frac{t_{td}}{1000} \gamma_c + \text{live load} \quad (8.2)$$

per square metre of dome surface. With $t_{td} = 100$ mm and a live load of 1.5 kN/m^2 , $w = 2.5 + 1.5 = 4.0 \text{ kN/m}^2$.

8.1.3 Membrane forces

Directly from Chapter 3, equations (3.3) and (3.4), evaluated at the springing where both are largest:

Key result

$$N_\phi = \frac{w R_{d,t}}{1 + \cos \phi_0} \quad (\text{compression}) \quad (8.3)$$

$$N_\theta = w R_{d,t} \left(\cos \phi_0 - \frac{1}{1 + \cos \phi_0} \right) \quad (8.4)$$

STEP 8.1 Check the compressive stress

$$\sigma = \frac{N_\phi}{t_{td}} \leq \sigma_{cc} \quad (8.5)$$

with N_ϕ in kN/m and t in mm giving the stress directly in N/mm^2 . For M30, $\sigma_{cc} = 8.0 \text{ N/mm}^2$. The computed stress is typically 0.2 to 0.4 N/mm^2 — a factor of twenty to spare. This check never governs and is done only to confirm nothing has gone badly wrong.

STEP 8.2 Check for buckling

A shallow dome in compression can buckle before it crushes. The classical critical pressure for a complete spherical shell is

$$p_{cr} = \frac{2E}{\sqrt{3(1 - \nu^2)}} \left(\frac{t}{R_d} \right)^2 \quad (8.6)$$

which for concrete with $E = 27\,400 \text{ N/mm}^2$, $t = 100 \text{ mm}$ and $R_d = 10 \text{ m}$ gives about 3.2 N/mm^2 , against an applied 0.004 N/mm^2 . Even allowing the factor of four to eight that imperfections demand, buckling is not remotely critical for a dome of these proportions. It becomes relevant only for very large or very thin domes.

STEP 8.3 Reinforcement

Both membrane forces are compressive, so no steel is required for strength. Provide the IS 3370 minimum of Chapter 2, in two directions — meridional and circumferential — in a single layer at mid-thickness for a 100 mm dome, since the section is under 200 mm and counts as one surface zone.

Key result

$$A_{st} = \frac{\rho_{\min}}{100} \times t_{td} \times 1000 \text{ mm}^2/\text{m, each way} \quad (8.7)$$

For a 100 mm dome in a tank with all dimensions under 15 m, $\rho_{\min} = 0.24$ and $A_{st} = 240 \text{ mm}^2/\text{m}$ — 8 mm bars at 200 mm centres each way.

Practical detailing of dome steel

Circumferential bars are simply rings, getting shorter towards the crown. Meridional bars all converge on the crown, where they would pile up impossibly, so they are stopped off in two or three stages and replaced near the crown by a light mesh. Chapter 17 shows the arrangement.

STEP 8.4 Horizontal thrust delivered to the ring beam

Key result

$$H = N_{\phi} \cos \phi_0 \quad \text{acting outward, in kN per metre of circumference} \quad (8.8)$$

This is the whole reason the top dome was analysed.

8.2 Ring beam B₁

The dome pushes outward on the ring at its springing. The ring resists by going into hoop tension, and hoop tension in a liquid-retaining structure means the IS 3370 crack check.

STEP 8.5 Hoop tension

From equation (3.11):

Key result

$$P = H \times R \quad \text{kN, tension} \quad (8.9)$$

STEP 8.6 Steel

$$A_{st} = \frac{P \times 1000}{\sigma_{st}} \text{ mm}^2 \quad (8.10)$$

with P in kN and $\sigma_{st} = 150 \text{ N/mm}^2$ for HYSD in direct tension.

STEP 8.7 Crack check on the section

The ring beam is a member in direct tension, so equation (2.2) applies with the gross area of the beam:

Key result

$$\sigma_{ct} = \frac{P \times 1000}{A_c + (m - 1)A_{st}} \leq \sigma_{ct,perm} \quad (8.11)$$

and inverting, the minimum section area required is

$$A_{c,req} = \frac{P \times 1000}{\sigma_{ct}} - (m - 1)A_{st} \quad (8.12)$$

If the trial section is too small, deepen the beam rather than widen it — a deeper beam is easier to reinforce and gives a better junction with the dome and the wall.

Do not forget that B₁ is a liquid-retaining member

It is above the top water level and never gets wet in service, so there is a temptation to design it by IS 456 limit state with the steel at $0.87f_y$. Resist it. The beam is monolithic with the wall and any crack in it propagates downward into a part that does get wet; and in any case the tank will be filled to overflowing at the water-tightness test. Use the working stress crack check.

STEP 8.8 Detailing

Provide the computed A_{st} as longitudinal bars distributed around the section — four corner bars plus intermediate bars on the faces — with the minimum governed by 0.24 percent of the gross area if that is larger. Close the ring with full tension laps, staggered, of not less than L_d in tension (Chapter 17). Nominal stirrups of 8 mm at 200 mm centres hold the cage.

The ring beam is not always necessary

Where the top dome is very shallow and the wall is thick, the thrust can be small enough that the wall itself, acting as a ring over a metre or so of its height, carries it without a separate beam. Most designers provide the beam anyway: it stiffens the top of the wall during construction, gives a clean edge to form the dome against, and costs very little.

Cross-check with the calculator

Section 7.1 and 7.2 of the booklet print $R_{d,t}$, $\sin \phi_0$ and $\cos \phi_0$; w ; both membrane forces with the stress check; the minimum steel with the surface-zone rule shown; the thrust H ; and then for B₁ the hoop tension, the steel, the achieved σ_{ct} and the required section area against the one provided.

The Cylindrical Wall

9.1 What the wall carries

Three things, and only one of them governs.

1. **Hoop tension** from the water pressure. This sets the thickness and the horizontal steel, and it is the whole of the design.
2. **Vertical compression** from the top dome, the ring beam B_1 and the wall's own weight, transmitted straight down into the conical slab. The stress is of the order of 0.1 N/mm^2 and is never critical.
3. **Vertical bending** where the base of the wall is restrained by the ring beam B_2 . This is real, it is not computed by the membrane method, and it is discussed at the end of the chapter.

9.2 Hoop tension

From Chapter 3, equation (3.10), the hoop tension per unit height at a depth y below the top water level is $\gamma_w y R$. It is zero at the water surface and maximum at the bottom of the cylindrical portion, where $y = h_w$:

Maximum hoop tension in the wall

$$F = \gamma_w h_w R \quad \text{kN per metre of height} \quad (9.1)$$

Note carefully that h_w is the depth of water *in the cylinder* — from the top water level down to the junction with the conical slab — and not the total depth of water in the container. The water below that level presses on the cone, not on the wall.

9.3 Steel

Key result

$$A_{st} = \frac{F \times 1000}{\sigma_{st}} \text{ mm}^2 \text{ per metre of height} \quad (9.2)$$

Placed as horizontal rings, half near each face. This is the largest single quantity of steel in the container after the conical slab, and it is the reason the wall is reinforced on both faces even though the tension is uniform through the thickness: the bars have to be held apart to be placed at all, and splitting them between the faces gives the best crack control.

9.3.1 Reducing the steel up the height

The hoop tension falls linearly to zero at the water surface, so the steel can be reduced in stages. Practice is to divide the wall into two or three horizontal zones and design each for the tension at its *bottom*:

Zone	Design depth	Typical reduction
Bottom third	h_w	full steel
Middle third	$\frac{2}{3}h_w$	about 67 percent
Top third	$\frac{1}{3}h_w$	about 33 percent, but minimum steel usually governs

Stopping bars off is only worth the drawing effort and the site risk on a tall tank. On a wall of 3 metres, carry the full steel throughout; the saving is a few hundred kilograms and the risk of a bar being stopped in the wrong place is not worth it.

9.4 Thickness — the check that actually governs

Apply equation (2.3) from Chapter 2:

Required wall thickness

$$t_{\text{req}} = F \left(\frac{1}{\sigma_{ct}} - \frac{m-1}{\sigma_{st}} \right) \quad (9.3)$$

with F in kN/m, stresses in N/mm², and t in mm.

Then check what has been provided:

$$\sigma_{ct} = \frac{F \times 1000}{1000 t + (m-1)A_{st}} \leq \sigma_{ct,\text{perm}} \quad (9.4)$$

A practical minimum

Whatever the arithmetic says, do not go below **150 mm** for a wall that has to be built on site with 45 mm cover on both faces and two layers of bar. A 120 mm wall is a formwork and compaction problem, not an economy. A common rule of thumb for the trial thickness is $t = 30h_w + 50$ mm with h_w in metres, which for a 3 m depth gives 140 mm, rounded up to 150.

9.5 Vertical steel

No vertical tension arises from the membrane analysis. Provide the IS 3370 minimum from Chapter 2 — $\rho_{\min}$ of each surface zone, in the vertical direction — distributed equally between the faces for a wall 200 mm or thicker, or in one layer if thinner.

The vertical bars also serve as the distribution steel holding the hoop rings in position, and they lap into the ring beam B₂ below and B₁ above, so they must be detailed to develop full tension at both ends regardless of what the analysis says.

9.6 Base fixity — the limit of the membrane method

The wall is not free to expand at its base, and the membrane solution assumes it is

Hoop tension makes the wall grow radially. The amount is $\delta = \gamma_w y R^2 / (E_c t)$, which for the worked example is about 0.3 mm at the base. The ring beam B₂, which is far stiffer radially than the wall, does not allow that movement. The wall is therefore held back at its base, and the restraint generates a *vertical bending moment* in the wall and a reduction in the hoop tension near the base.

The effect is confined to a height of a metre or two above B₂ and is handled in one of three ways.

1. **Design for it.** IS 3370 (Part 4) tabulates coefficients for the moments and hoop forces in a cylindrical wall as a function of the parameter

$$\frac{H^2}{Dt} \quad (9.5)$$

for walls hinged or fixed at the base. Enter the table with that parameter, read the moment coefficient, and compute $M = \text{coefficient} \times \gamma_w H^3$. Design the vertical steel on the liquid face for that moment, using $\sigma_{st} = 150 \text{ N/mm}^2$ in bending and the bending tension value of σ_{ct} from Table 2.2. This is the rigorous route.

2. **Detail for it.** Provide a generous haunch at the wall–cone junction and carry the vertical steel on the liquid face at not less than the minimum, well anchored into B₂. For a short wall on a relatively flexible ring beam this is normal practice and is what most Indian tanks are built to.
3. **Design it out.** Provide a sliding or flexible joint at the base of the wall so that it genuinely is free to move. This is done for large ground-supported tanks; it is not practical at the top of an elevated staging, where the wall must also transfer vertical load and shear.

What this book does

Route 2, with route 1 recommended and flagged wherever the tank is large. The software prints the parameter $H_1^2/(Dt)$ in the wall section of the booklet precisely so that the reader can go to IS 3370 (Part 4) with it. For the worked example the value is 6.7, which is well inside the range where the base moment is significant, and route 1 should be followed for a real design.

9.7 Summary of the wall design

The whole of Chapter 9 in four lines

$$F = \gamma_w h_w R \quad \text{kN/m} \quad (9.6)$$

$$A_{st} = 1000F/\sigma_{st} \quad \text{mm}^2/\text{m, horizontal, half each face} \quad (9.7)$$

$$t \geq F(1/\sigma_{ct} - (m-1)/\sigma_{st}) \quad \text{mm, not less than 150} \quad (9.8)$$

$$A_{st,v} = \rho_{\min} z/100 \times 1000 \quad \text{mm}^2/\text{m per face, vertical} \quad (9.9)$$

Cross-check with the calculator

Section 7.3 of the booklet gives F with its substitution, the steel with the chosen bar and spacing for each face, the required thickness with the bracket evaluated, the achieved σ_{ct} , and the vertical minimum steel. It closes with the $H_1^2/(Dt)$ note about base fixity, which repeats the warning above so that it cannot be missed by someone reading only the output.

The Circular Girder B₃

10.1 The most heavily worked member in the tank

The circular girder sits on the columns at the *bottom* of the conical slab, on a circle of radius r , and it collects everything: the whole weight of the container, the whole weight of the water, and the roof live load. It then has to carry that load around a curve to a small number of point supports.

Its radius matters twice over. The three design actions all scale with WR_c , and because the girder is at the cone bottom, $R_c = r$ rather than R . For a typical r/R of 0.66 that is a one-third reduction in every moment and torsion compared with a girder at the tank diameter — one of the quiet economies of the Intze form, and a reason not to move the column circle outward without a good cause.

Carrying the load round a curve is the difficulty. A straight beam between two supports carries load in bending alone. A *curved* beam does not: because the line of the beam turns between the load and the support, the bending moment vector at one section is not parallel to that at the next, and part of it appears as **torsion**. Every curved beam on discrete supports carries bending, shear *and* torsion simultaneously, and the girder must be designed for all three.

10.2 Analysis of a circular beam on equally spaced supports

Take a closed circular beam of radius R_c (in the standard arrangement $R_c = r$) carrying a uniformly distributed load w per unit length of its circumference, supported on N equally spaced columns. Let $2\alpha = 2\pi/N$ be the angle subtended by one span, so $\alpha = \pi/N$ is the half-span angle.

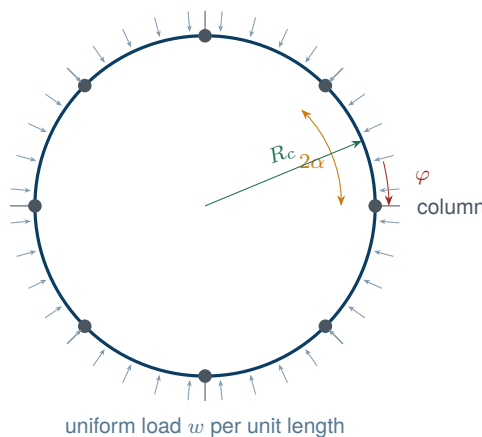


Figure 10.1: The circular girder, seen in plan. φ is measured from a support.

10.2.1 Setting up

By symmetry, at each support the torsion is zero and the shear reverses. Each column carries $V = wR_c \alpha$ from each side. Let M_0 be the (hogging) bending moment at a support. Then at an angle φ from that support, resolving the moment vectors:

$$M_\varphi = M_0 \cos \varphi + V R_c \sin \varphi - w R_c^2 (1 - \cos \varphi) \quad (10.1)$$

$$T_\varphi = M_0 \sin \varphi - V R_c (1 - \cos \varphi) + w R_c^2 (\varphi - \sin \varphi) \quad (10.2)$$

10.2.2 Finding M_0

At mid-span, $\varphi = \alpha$, symmetry again requires the torsion to vanish. Setting $T_\alpha = 0$ with $V = w R_c \alpha$:

$$\begin{aligned} M_0 \sin \alpha - w R_c^2 \alpha (1 - \cos \alpha) + w R_c^2 (\alpha - \sin \alpha) &= 0 \\ M_0 \sin \alpha &= w R_c^2 [\alpha - \alpha \cos \alpha - \alpha + \sin \alpha] = w R_c^2 [\sin \alpha - \alpha \cos \alpha] \end{aligned}$$

Support moment

$$M_0 = w R_c^2 (1 - \alpha \cot \alpha) \quad \text{hogging} \quad (10.3)$$

10.2.3 Mid-span moment and maximum torsion

Substituting back at $\varphi = \alpha$:

Mid-span moment

$$M_\alpha = w R_c^2 \left(\frac{\alpha}{\sin \alpha} - 1 \right) \quad \text{sagging} \quad (10.4)$$

and the torsion becomes

$$T_\varphi = w R_c^2 [\varphi - \alpha + \alpha \cos \varphi - \alpha \cot \alpha \sin \varphi] \quad (10.5)$$

which is zero at both ends of the span and peaks in between, at the angle φ_m satisfying

$$\alpha \sin \varphi_m + \alpha \cot \alpha \cos \varphi_m = 1. \quad (10.6)$$

10.2.4 Expressing everything in terms of the total load

Design offices work with the *total* load W on the girder rather than w . Since $W = 2\pi R_c w$, each result above becomes a coefficient times $W R_c$:

The three design actions

$$M_{\text{support}} = C_1 W R_c, \quad M_{\text{span}} = C_2 W R_c, \quad T_{\text{max}} = C_3 W R_c \quad (10.7)$$

with

$$C_1 = \frac{1 - \alpha \cot \alpha}{2\pi}, \quad C_2 = \frac{\alpha / \sin \alpha - 1}{2\pi}, \quad C_3 = \frac{\varphi_m - \alpha + \alpha \cos \varphi_m - \alpha \cot \alpha \sin \varphi_m}{2\pi} \quad (10.8)$$

Read the table before choosing the number of columns

Going from 6 columns to 8 nearly halves the girder moment; going from 8 to 12 more than halves it again. The torsion falls even faster. Against that, each extra column is another footing, another

Table 10.1: Coefficients for a circular beam on N equally spaced supports, computed from the closed forms above

N	C_1 (support)	C_2 (span)	C_3 (torsion)	φ_m (degrees)
4	0.03415	0.01762	0.005272	19.20
6	0.01482	0.00751	0.001507	12.73
8	0.00827	0.00417	0.000628	9.53
10	0.00527	0.00265	0.000320	7.62
12	0.00365	0.00183	0.000184	6.35
16	0.00205	0.00103	0.000078	4.76

set of bracings, and more formwork. Eight is the usual compromise for a 200 to 600 kL tank and twelve for a large one. If the girder is coming out impossibly deep, add columns before adding depth.

10.3 Designing the section

STEP 10.1 Total load on the girder

$$W = W_{\text{container}} + W_T + W_{\text{live}} \quad (10.9)$$

That is: everything above the columns. The staging's own weight is not included — it is below the girder.

STEP 10.2 Factor the loads

The girder is not a liquid-retaining member, so IS 456 limit state applies and the loads are factored by 1.5 for the gravity case:

$$M_{u,\text{support}} = 1.5 C_1 W R_c, \quad M_{u,\text{span}} = 1.5 C_2 W R_c, \quad T_u = 1.5 C_3 W R_c \quad (10.10)$$

STEP 10.3 Combine torsion with bending

IS 456 Cl. 41.4.2 converts the torsion into an equivalent bending moment:

Key result

$$M_t = T_u \frac{1 + D/b}{1.7}, \quad M_{e1} = M_u + M_t \quad (10.11)$$

where D is the overall depth and b the breadth of the section. The section is then designed in flexure for M_{e1} .

STEP 10.4 Flexural steel

Apply equation (2.5) with $d = D - \text{cover} - \text{link} - \phi/2$. Top steel over the columns for the hogging moment, bottom steel at mid-span for the sagging. Check that $M_{e1} \leq M_{u,\text{lim}}$; if not, deepen the girder.

STEP 10.5 Shear and torsion reinforcement

The equivalent shear is

$$V_e = V_u + 1.6 \frac{T_u}{b} \quad (10.12)$$

[IS 456 Cl. 41.3.1] with $V_u = 1.5W/(2N)$, the factored shear on each side of a column. Compute $\tau_{ve} = V_e/(bd)$ and check it against $\tau_{c,max}$ from IS 456 Table 20 — 2.8 N/mm² for M20, 3.1 for M25, 3.5 for M30, 3.7 for M35, 4.0 for M40. If τ_{ve} exceeds that limit no amount of stirrup steel will help and the section must be widened or deepened; this is the check that most often forces a deeper girder on a large tank. Within the limit, provide closed stirrups, two-legged, designed for combined shear and torsion per IS 456 Cl. 41.4.3. Torsion demands *closed* links — open links are useless against torsion — and longitudinal bars distributed around the perimeter of the section, not concentrated top and bottom.

STEP 10.6 Side-face reinforcement

For a girder deeper than 750 mm, IS 456 Cl. 26.5.1.3 requires side-face steel of not less than 0.1 percent of the web area on each face, at a spacing not exceeding 300 mm or the web thickness. For a 900 mm girder this is not optional and it also helps with the torsion.

10.4 The three rings, and which force each carries

Before the hoop calculation, it helps to have all three rings in one place, because they are routinely confused and they carry quite different things.

Table 10.2: The three ring members of an Intze tank

Ring	Where	What acts on it	Result
B ₁	Top of the wall, radius R	Outward thrust of the top dome	Hoop tension, small. Designed in Chapter 8
B ₂	Wall–cone junction, radius R	Cone pushes outward; water pressure pushes outward	Hoop tension , usually the largest in the tank. Designed in Chapter 11
B ₃	Cone bottom, radius r , on the columns	Cone pushes inward; arched bottom dome pushes outward; water pushes outward	Small <i>residual</i> hoop force — the Intze balance — <i>plus</i> bending, shear and torsion from spanning the columns

Note

Only B₃ carries bending. B₁ and B₂ are pure hoop members: they are continuously supported by the shells on either side and have no span. That is why B₃ is four times their size.

10.5 Hoop force in the girder — the Intze balance

The girder is also a ring, and three radial line loads act on it.

1. The **water pressure** on its inner face, $\gamma_w(h_w + h_2) \times d_3$ per metre of circumference, acting *outward*.
2. The **inward push of the conical slab**. The cone is in meridional compression and arrives at the girder sloping up and outward, so its thrust on the girder points *inward*: $H_{in} = N_{\phi,cone} \sin \alpha$ per metre, from Chapter 11.
3. The **outward push of the bottom dome**. The dome arches up from the girder and is also in compression, so its thrust points *outward*: $H_{out} = N_{\phi,bd} \cos \phi_0$ per metre, from Chapter 12.

Net hoop force in B₃

$$w_{\text{net}} = \underbrace{\gamma_w(h_w + h_2) d_3}_{\text{water, outward}} + \underbrace{N_{\phi, \text{bd}} \cos \phi_0}_{\text{dome, outward}} - \underbrace{N_{\phi, \text{cone}} \sin \alpha}_{\text{cone, inward}}, \quad P = w_{\text{net}} r \quad (10.13)$$

positive meaning net outward, hence hoop **tension** in the ring.

The whole point of the Intze geometry is that the second and third terms very nearly cancel. In the worked example they are 181 and 170 kN/m, leaving a residual of 11 kN/m; the water contributes another 36, and the net hoop force in the girder is 179 kN — against the 272 kN carried by the much smaller ring B₂ above. Without the balance, the dome term alone would put more than 680 kN of hoop tension into a member that is already working hard in bending and torsion.

How good does the balance have to be?

It does not have to be perfect, and in built tanks it never is. RDSO B-1235 leaves roughly eight percent unbalanced. What matters is that the residual is small enough that the hoop steel can be accommodated as ordinary longitudinal bars around the perimeter of a girder that is sized by bending anyway. A residual under about a third of either thrust is comfortable; above that, revisit the bottom dome rise (Chapter 12) or r/R (Chapter 4) before making the girder bigger.

Where equation (10.13) gives tension — it usually does, by a modest amount — provide $A_{st} = P \times 1000 / \sigma_{st}$ as additional longitudinal bars distributed around the perimeter of the section, and run the crack check of Chapter 2 on the gross area:

$$\sigma_c = \frac{P \times 10^3}{A_c + (m - 1)A_{st}} \leq \sigma_{ct}. \quad (10.14)$$

Where it gives compression, nothing is needed beyond the flexural and torsional steel.

Do not add the hoop force to the flexural steel

They act in different directions. The hoop force is along the axis of the curved beam; the flexural steel resists bending about a horizontal axis. Where the hoop force is tensile, the hoop steel is provided as additional longitudinal bars distributed around the perimeter of the section, and the flexural steel is provided on top of that. Where it is compressive, nothing is needed.

The sign trap

If your calculation shows the cone pushing the girder *outward* and the dome pulling it *inward*, both signs are reversed, and equation (10.13) will read as the sum of two large thrusts instead of their difference. Go back to Chapter 1 and check where you have put the columns. A correct Intze calculation always shows the two lower thrusts opposing.

10.6 Where the girder sits

The girder is cast monolithically with the conical slab arriving from above and outboard, the bottom dome springing from it upward and inboard, and the column heads below. It is the junction of four members, and it is the most congested piece of reinforcement in the tank. The detail — with the cone's meridional bars, the dome's meridional bars, the column's longitudinal bars and the girder's own top, bottom, side-face and hoop steel all arriving at the same place — is drawn in Chapter 17, and it is worth drawing full size before the site starts bending bars.

Note which way the two shell reinforcements turn into the girder. The cone's bars come down and *inward* and are anchored towards the inner face; the dome's bars come up and *inward* from the outer face. They cross, and the crossing must be planned, because both need their full development length past the girder centreline.

Cross-check with the calculator

Section 8 of the booklet prints the balance calculation first — the two thrusts side by side, the residual, the percentage unbalance and the rise that would make it exact — then the total load W , the three coefficients for the chosen N , the three actions, the equivalent moment M_t and M_{e1} , the effective depth, $M_{u,lim}$, the top and bottom steel with bar counts, and the equivalent shear stress with a note on the stirrups.

The Conical Slab and Ring Beam B₂

11.1 The member that defines the tank

The conical slab carries the bulk of the water down to the circular girder, is the thickest shell in the container, and contains more reinforcement than anything else. It is also the member most often designed wrongly, for the reason set out in Chapter 1.

What the cone actually does

The girder and the columns are at the *bottom* of the cone. The cone therefore **stands** rather than hangs: it is in **meridional compression**, carrying the wall, the top dome, both upper ring beams and all the water above it down to the girder. Across the meridian, however, it is in **hoop tension**, from the water pressure and from the outward component of the slope. So the cone is a compression member in one direction and a cracked tension member in the other, and it is the hoop tension that fixes its thickness.

This split is the whole design of the member. The meridional direction needs a compressive stress check and nominal steel; the hoop direction needs the IS 3370 crack check and a substantial area of ring bars.

11.2 Geometry and angles

Two angles describe the cone and it is essential to keep them apart.

Symbol	Measured from	Used in
θ_c	the horizontal	the geometry: $h_2 = (R - r) \tan \theta_c$
$\alpha = 90^\circ - \theta_c$	the vertical	every membrane formula

For the common 45° cone the two are equal and the distinction is invisible, which is exactly why it causes trouble when the slope is changed to 30° or 60° . The membrane formulae of Chapter 3 all use α , the angle from the *vertical*.

Other quantities:

$$L_s = \sqrt{(R - r)^2 + h_2^2} \text{ (slant length),} \quad A_c = \pi(R + r)L_s \text{ (surface area),} \quad q = \frac{t_c}{1000} \gamma_c \text{ (11.1)}$$

The cone is narrow at the bottom (radius r , at the girder) and wide at the top (radius R , at the wall). Every formula below is written with that orientation in mind.

11.3 Meridional compression

11.3.1 The weight standing on the cone

Cut the cone on a horizontal plane at a radius r_0 and take the free body *above* the cut. Everything in it is held up by the membrane force across the cut. That includes:

- the cone above the cut, the ring beam B_2 , the cylindrical wall, the ring beam B_1 and the top dome — all concrete, plus the roof live load;
- and *all the water resting on the cone above the cut*, plus the whole column of water standing in the cylinder above it.

That last point needs care.

The vertical load from the water on a shell

The water presses normal to the surface everywhere. Integrating the vertical component over a shell surface gives exactly the weight of the water standing vertically above that surface, up to the free level. So the vertical water load arriving at a horizontal cut of radius r_0 is the entire water weight less the part still standing on the shell *below* the cut:

$$W_{\text{water}, r_0} = \gamma_w [V_{\text{total}} - \pi r_0^2 y_{r_0} - V_{\text{below the cut}}] \quad (11.2)$$

where y_{r_0} is the depth of water above the cut circle.

Check at the bottom of the cone

Put $r_0 = r$. Everything in the container is above the cut except the bottom dome and the water directly on it, and the membrane force must carry the lot. Together with the bottom dome's own compressive thrust, which arrives separately, the girder receives the entire weight of the tank. Which is right: a vertical wall carries no vertical water load, so every drop of water in the tank is ultimately carried down by the cone and the bottom dome.

Load standing on the cone, at each end

At the *top* of the cone, where it meets the wall, only the structure above it is carried (the water in the cylinder presses on the cone surface, not on its top edge):

$$W_{\text{top}} = W_{\text{top dome}} + W_{B_1} + W_{\text{wall}} + W_{B_2} + W_{\text{live}} \quad (11.3)$$

At the *bottom*, at the girder, the cone's own weight and all the water it carries have been added:

$$W_{\text{bot}} = W_{\text{top}} + W_{\text{cone}} + \gamma_w [V_1 + V_2 - \pi r^2 (h_w + h_2)] \quad (11.4)$$

the bracket being the water that rests on the conical surface rather than on the bottom dome.

11.3.2 The meridional force

From equation (3.9), with the free body taken above the cut:

Meridional compression in the conical slab

$$N_\phi = \frac{W}{2\pi r_0 \cos \alpha} \quad \text{compression, kN per metre along the slant} \quad (11.5)$$

evaluated at the top with $r_0 = R$ and $W = W_{\text{top}}$, and at the bottom with $r_0 = r$ and $W = W_{\text{bot}}$.

The bottom always governs, and by a wide margin: the load has grown while the radius has shrunk. In the worked example N_ϕ goes from 51 kN/m at the top to 241 kN/m at the girder.

11.3.3 Meridional check and steel

There is no tension to reinforce against, so the meridional direction needs only a stress check and minimum steel:

Meridional compressive stress

$$\sigma_{cc} = \frac{N_{\phi, \text{bot}} \times 10^3}{t_c \times 10^3} = \frac{N_{\phi, \text{bot}}}{t_c} \leq \sigma_{cc, \text{perm}} \quad (11.6)$$

with $\sigma_{cc, \text{perm}}$ from IS 3370 Table 1 — 6.0 N/mm² for M25, 8.0 for M30. The check is never close for a normally proportioned tank; in the worked example $\sigma_{cc} = 1.20$ against 8.0.

Meridional steel is then the IS 3370 Cl. 8.1 surface-zone minimum, running up and down the slope, half near each face, anchored into the ring beam B₂ above and the girder B₃ below. It is not carrying the water — the concrete is — but it controls shrinkage and thermal cracking, resists the bending that develops at both ends where the membrane assumption breaks down, and holds the shell together during construction.

Do not skip the anchorage because the steel is nominal

The membrane theory says the cone is in pure compression; the real shell is not. At the girder and at B₂ the slab is restrained against rotation, and a local bending moment develops over a length of roughly $0.76\sqrt{r t_c}$ from each edge. The meridional bars must be developed fully into both rings to carry it. Chapter 17 details this.

11.4 Hoop tension

From equation (3.8):

Hoop tension in the conical slab

$$N_{\theta} = \left(\frac{p}{\cos \alpha} + q \tan \alpha \right) \frac{d}{2} \quad (11.7)$$

with $p = \gamma_w y$ the water pressure at the level considered, q the self weight per unit area of the shell, and d the diameter at that level.

This is evaluated at the top ($d = D$, $p = \gamma_w h_w$) and at the bottom ($d = 2r$, $p = \gamma_w (h_w + h_2)$), and the greater governs. The two terms pull in opposite directions as you move down the cone — the diameter shrinks but the pressure grows — so which end governs depends on the proportions, and both must be computed. For the worked example they come out within three percent of each other: 282 kN/m at the top against 289 kN/m at the bottom.

Note how much larger these are than the meridional force. The hoop tension is what this member is really fighting, and it is what sets the thickness.

$$A_{st, h} = \frac{N_{\theta} \times 1000}{\sigma_{st}} \text{ mm}^2/\text{m}, \quad \text{half each face} \quad (11.8)$$

11.5 Thickness

Two crack checks, both from equation (2.3), and the thicker answer wins:

Required thickness of the conical slab

$$t_{\text{req}} = \max \left[\underbrace{N_{\theta} \left(\frac{1}{\sigma_{ct}} - \frac{m-1}{\sigma_{st}} \right)}_{\text{hoop crack check}}, \underbrace{\frac{N_{\phi, \text{bot}}}{\sigma_{cc}}}_{\text{meridional compression}}, 200 \text{ mm} \right] \quad (11.9)$$

Only the hoop term and the 200 mm floor ever govern in practice. The compression term is written in for completeness — it would need a meridional force some six times larger than a normal tank produces before it mattered — but it must still be checked, because on a very large tank with few columns it is not impossible.

The 200 mm floor is practical rather than analytical: a sloping shell with two layers of heavy bar, 45 mm cover each side, and concrete that has to be placed on a slope and compacted without running off cannot sensibly be thinner.

The iteration is not optional here

q depends on t_c ; N_{θ} depends on q ; t_c depends on N_{θ} . And W_{top} contains W_{cone} , which also depends on t_c . Both loops must be closed. Start with 250 mm, compute, and repeat until the thickness stops moving — two passes is normally enough. The software runs the loop up to four times.

11.6 Horizontal components delivered to the rings

These are the outputs that the rest of the design consumes.

Radial components of the conical membrane force

$$\text{At the top, onto } B_2 : \quad H_{\text{out}} = N_{\phi, \text{top}} \sin \alpha \quad \text{outward} \quad (11.10)$$

$$\text{At the bottom, onto } B_3 : \quad H_{\text{in}} = N_{\phi, \text{bot}} \sin \alpha \quad \text{inward} \quad (11.11)$$

The directions follow from the compression argument of Chapter 1: a compressive membrane force pushes its ring *away* from the body of the shell. At the top of the cone the shell lies below and inboard, so it pushes B_2 outward; at the bottom it lies above and outboard, so it pushes the girder inward.

Sanity check on the two thrusts

The thrust at the girder is far the larger of the two, because the meridional force has grown down the slope while the radius has shrunk. In the worked example $H_{\text{out}} = 36 \text{ kN/m}$ at B_2 and $H_{\text{in}} = 170 \text{ kN/m}$ at the girder. It is the second of these that the bottom dome has to balance.

11.7 Ring beam B_2 at the wall–cone junction

B_2 is the ring where the vertical wall turns into the sloping cone. Two outward actions meet there and nothing opposes them, so B_2 is in pure hoop tension and it is usually the most heavily tensioned ring in the tank.

Hoop tension in B₂

$$w_{\text{net}} = \underbrace{\gamma_w h_w d_2}_{\text{water on the beam face}} + \underbrace{N_{\phi, \text{top}} \sin \alpha}_{\text{cone, outward}}, \quad P_2 = w_{\text{net}} R \quad (11.12)$$

with d_2 the depth of the beam. The steel and the crack check follow the usual pattern:

$$A_{st} = \max \left(\frac{P_2 \times 10^3}{\sigma_{st}}, 0.24\% A_c \right), \quad \sigma_c = \frac{P_2 \times 10^3}{A_c + (m - 1)A_{st}} \leq \sigma_{ct} \quad (11.13)$$

where $A_c = b_2 d_2$ is the gross area of the ring.

B₂ gets hoop tension, not compression

Under the mistaken hanging model this ring comes out in hoop *compression* and is given nominal steel. It is not in compression. Both the cone's thrust and the water pressure push it outward, and it needs a full ring of tension steel in closed laps. In the worked example $P_2 = 272$ kN, which is half as much again as the girder's residual hoop force below it.

The wall's base fixity

The membrane calculation treats the wall and the cone as meeting freely at B₂. In reality the junction is monolithic and stiff, the wall's hoop expansion is restrained there, and a local vertical bending moment develops in the wall over roughly $0.76\sqrt{R t_w}$ above the ring. Where the tank is large or the wall thin, design the bottom metre of wall for that moment as well — the coefficients are in IS 3370 (Part 4), the reprinted 1967 tables, for a wall hinged or fixed at the base. The software flags the case; it does not size it for you.

11.8 Practical points

- **Concreting a 45° slope** is the single biggest construction difficulty in an Intze tank. The mix has to be stiff enough not to slump down the slope and workable enough to compact around two layers of congested steel. Top shuttering over part of the slope is normal. A slope of 30° is much easier to build and is worth considering where the geometry allows.
- **Bar spacing** in the cone is often down to 75 or 90 mm, which is at the practical limit for placing and vibrating. If the spacing comes out below 75 mm, increase the thickness or the bar diameter rather than accepting it.
- **The junction with the girder** carries the whole container into a single ring. The meridional bars are taken down into the girder and hooked, and a haunch is provided on the inside of the angle. The hoop bars are the ones that must be lapped properly — staggered laps of at least L_d , never all in one plane. See Chapter 17.

Cross-check with the calculator

Section 7.4 of the booklet opens with an explicit statement of the load path — that the columns are at the cone bottom and the slab is in meridional compression with hoop tension — then prints α and $\cos \alpha$, the load standing on the cone split into structure and water, N_{ϕ} at top and bottom with their substitutions, the compressive stress check, both hoop tensions with their substitutions, the hoop steel, and the required thickness against the one provided. Section 7.5 does B₂.

The Bottom Dome and the Intze Balance

12.1 Bottom dome

12.1.1 Geometry

A shallow spherical cap of rise h_b arching *upward* from a base circle of radius r at the level of the circular girder:

$$R_{d,b} = \frac{r^2 + h_b^2}{2h_b}, \quad \sin \phi_0 = \frac{r}{R_{d,b}}, \quad \cos \phi_0 = \sqrt{1 - \sin^2 \phi_0}, \quad A_{bd} = 2\pi R_{d,b} h_b \quad (12.1)$$

Which way it arches, and why it matters twice

The dome springs from the girder and rises *into* the tank. That has two consequences that are easy to get backwards. First, its bulge occupies space that would otherwise hold water, so the segment volume is *subtracted* from the capacity (Chapter 4) rather than added. Second, it is a compression arch, not a hanging membrane, so it pushes the girder *outward* — which is the whole basis of the Intze balance.

12.1.2 Load

Everything the dome supports: the water standing above it, plus its own weight.

Key result

$$W_{bd} = \gamma_w [\pi r^2 (h_w + h_2) - V_3] + W_{\text{dome}} \quad (12.2)$$

The bracketed term is the cylinder of water of radius r from the free surface down to the springing level, *less* the volume V_3 of the bulge, which contains no water.

12.1.3 Membrane forces

The dome stands, so both membrane forces are compressive. From equation (3.2), with the free body taken as the cap above the springing:

Meridional compression

$$N_\phi = \frac{W_{bd}}{2\pi r \sin \phi_0} \quad \text{compression} \quad (12.3)$$

The hoop force follows from the first membrane equation with $r_1 = r_2 = R_{d,b}$ and the normal pressure $p_n = \gamma_w (h_w + h_2)$ pressing down on the dome at the springing:

Hoop force

$$N_\theta = p_n R_{d,b} - N_\phi \quad (12.4)$$

For a shallow cap both come out compressive — ϕ_0 is well under the $51^\circ 49'$ threshold of Chapter 3 — and in the worked example they are 205 and 201 kN/m. Nothing is cracking, so no IS 3370 crack check applies to this member; only a compressive stress check and minimum steel.

The dome gets flatter, the thrust gets worse

$\sin \phi_0 = r/R_{d,b}$, and a flatter dome has a much larger $R_{d,b}$, so a small rise makes $\sin \phi_0$ small and N_ϕ enormous. Halving the rise roughly doubles the meridional force — and, because $H_{\text{out}} = N_\phi \cos \phi_0$ and $\cos \phi_0 \rightarrow 1$, it roughly doubles the outward thrust on the girder too. This is the same behaviour as a shallow masonry arch: the flatter it is, the harder it pushes its abutments. It is also why the rise cannot simply be minimised to save concrete.

12.1.4 Steel and thickness

$$A_{st,m} = A_{st,\min} \text{ (Chapter 2, Cl. 8.1 surface zones)} \quad \text{meridional, mm}^2/\text{m} \quad (12.5)$$

$$A_{st,h} = A_{st,\min} \quad \text{circumferential, mm}^2/\text{m} \quad (12.6)$$

$$t_{\text{req}} = \max \left[\frac{\max(N_\phi, |N_\theta|)}{\sigma_{cc}}, 120 \right] \quad \text{mm} \quad (12.7)$$

The minimum steel of Chapter 2 always governs the bottom dome of a normally proportioned tank, because the member is in compression throughout. The 120 mm floor is practical — two layers of bar with cover — and in practice 150 to 200 mm is used so that the dome is stiff enough not to be sensitive to a construction imperfection in its curvature.

Why minimum steel, not none

A compression shell needs steel for three reasons that have nothing to do with the membrane forces: shrinkage and thermal cracking; the local bending that develops at the springing, where the girder restrains the dome's rotation; and buckling. A shallow compression shell can snap through if it is too thin, and the reinforcement, together with the thickness floor, keeps it comfortably away from that. IS 3370 Cl. 8.1 minimum in both directions, both faces, is the standard provision.

12.1.5 Horizontal component onto the girder

Key result

$$H_{\text{out}} = N_\phi \cos \phi_0 \quad \text{acting **outward** on the girder} \quad (12.8)$$

Outward because the dome lies above and inboard of the ring, and a compressive force pushes the ring *away* from the shell. This is the arch thrusting against its abutment, and the girder is the abutment.

12.2 The Intze balance

12.2.1 The three radial forces at the girder

Three things push on the circular girder radially:

Source	Magnitude, kN per metre	Direction
Conical slab (Chapter 11)	$H_{in} = N_{\phi, \text{cone, bot}} \sin \alpha$	inward
Bottom dome	$H_{out} = N_{\phi, bd} \cos \phi_0$	outward
Water pressure on the beam face	$\gamma_w (h_w + h_2) d_3$	outward

Net radial load and hoop force

$$w_{\text{net}} = H_{\text{out}} - H_{\text{in}} + \gamma_w (h_w + h_2) d_3, \quad P = w_{\text{net}} \times r \quad (12.9)$$

Positive is outward, giving hoop tension.

The first two terms are the ones that matter, and they oppose. That is the Intze balance, and it is the reason the whole form exists.

12.2.2 The balance condition

Set the two thrusts equal and solve for the rise of the bottom dome that makes them cancel.

Ignoring the small water-pressure term on the beam face and writing W_{bd} for the load on the bottom dome:

$$H_{\text{out}} = N_{\phi, bd} \cos \phi_0 = \frac{W_{bd}}{2\pi r \sin \phi_0} \cos \phi_0 = \frac{W_{bd}}{2\pi r} \cot \phi_0.$$

Setting this equal to H_{in} gives

$$\cot \phi_0 = \frac{2\pi r H_{\text{in}}}{W_{bd}} \quad \Rightarrow \quad \phi_0 = \arctan \left(\frac{W_{bd}}{2\pi r H_{\text{in}}} \right).$$

The rise then follows from the geometry of a spherical cap, $h_b = R_{d,b}(1 - \cos \phi_0)$ with $R_{d,b} = r / \sin \phi_0$.

Rise of the bottom dome for exact balance

$$\phi_0 = \arctan \left(\frac{W_{bd}}{2\pi r H_{\text{in}}} \right), \quad h_b = \frac{r}{\sin \phi_0} (1 - \cos \phi_0) = r \tan \frac{\phi_0}{2} \quad (12.10)$$

The last form is neat: **the balancing rise is $r \tan(\phi_0/2)$.**

What the formula says, in words

W_{bd} is the weight the dome carries; $2\pi r H_{\text{in}}$ is the total inward pull the cone exerts on the ring. Their ratio is the tangent of the springing angle. A heavy dome load and a light cone thrust want a steep dome; a light dome load and a heavy cone thrust want a flat one. Since the cone thrust grows with the whole container above it while the dome load is only the water in a cylinder of radius r , larger and taller tanks want *flatter* bottom domes — which runs against the intuition most people bring to it.

How to use equation (12.10) in practice

It cannot be applied at the start, because W_{bd} and H_{in} both depend on the geometry that is being solved for. The practical method is:

1. Start with $h_b = 0.25r$.
2. Complete the design of Chapters 11 and 12.
3. Evaluate equation (12.10) and compare.
4. If the balancing value is more than about 15 percent away, adopt it and repeat once.

The software does this automatically and prints the balancing rise alongside the one you used.

The balancing rise is shallower than the textbooks say

For the worked example, equation (12.10) gives $h_b = 0.264r$; RDSO B-1235, an issued drawing for a tank of almost identical proportions, uses $0.245r$. Neither is anywhere near the $0.30r$ to $0.45r$ commonly quoted. That range comes from the hanging model, in which the two thrusts *add* rather than oppose and the equation has a different form. If your balance calculation is pointing you at $0.40r$ or beyond, the signs are wrong — go back to Chapter 1.

12.2.3 How close is close enough

Perfect balance is neither achievable nor necessary. A useful measure is the residual:

unbalanced fraction = $\frac{|H_{out} - H_{in}|}{H_{in}} \times 100$ percent (12.11)

Residual	Verdict
under 15 percent	excellent; the residual hoop steel is a handful of bars
15 to 35 percent	acceptable; run the crack check on the girder's gross section
over 35 percent	the geometry is working against you — adjust h_b first, then r/R

What a large residual actually costs

Not very much, in a small tank — the girder simply carries some extra longitudinal steel around its perimeter. What it costs is the elegance of the form, and in a large tank it can become the governing action on a member that is already working hard in bending, shear and torsion. The balance is worth chasing because it is free: changing the rise of the bottom dome by 200 mm costs nothing and can halve the hoop force.

12.3 Why the bottom dome is not flat

It is worth seeing what the balance buys. If the bottom dome were replaced by a flat slab, it would deliver *no* horizontal thrust at all — a flat plate pushes only vertically. Then equation (12.9) would read $w_{net} = -H_{in} + \gamma_w(\cdot)d_3$, and the girder would carry a hoop *compression* of roughly $H_{in} \times r = 170 \times 3.79 = 646$ kN — against the 179 kN of tension it actually carries.

Hoop compression sounds harmless, and in a plain ring it would be. But this ring is also a beam in bending and torsion, and 646 kN of axial compression on a 550×720 section, combined with a support moment of 314 kN m, pushes it into the biaxial regime and demands a full P–M check. The balance does not merely reduce the force; it keeps the girder a simple flexural member.

And that is before considering that the flat slab would itself have to be designed in bending, at several times the thickness of the dome, adding its own weight to everything below.

Two ways to see that the balance is real

First, dimensionally: the two thrusts in the worked example are 181 and 170 kN/m — within six percent of each other, from two entirely different calculations. That near-equality is not an accident of the numbers; it is what the geometry was chosen to produce.
Second, against a built tank: apply the procedure of this chapter to RDSO B-1235's propor-

tions and equation (12.10) returns a rise within a few percent of the $0.245r$ actually detailed. A method that reproduces an issued drawing from first principles is a method you can trust.

Cross-check with the calculator

Sections 7.6 and 8.1 of the booklet cover this chapter. 7.6 prints $R_{d,b}$, the springing angles, the load W_{bd} with the displaced volume shown explicitly, both membrane forces, the compressive stress check, the minimum steel and H_{out} . 8.1 prints the three radial contributions side by side with their directions, the net, the hoop force in the girder, the steel and the crack check, the unbalanced percentage — and then the rise that would balance exactly, from equation (12.10), next to the rise actually used.

Load Combinations and Column Forces

13.1 Sharing the overturning moment between the columns

The staging delivers to the foundation a vertical load P , a horizontal shear V and an overturning moment M . The vertical load divides equally among the n columns. The moment does not: it puts extra load on the leeward columns and relieves the windward ones, in proportion to their distance from the neutral axis of the column group.

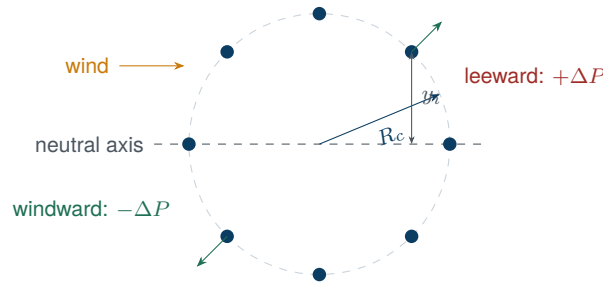


Figure 13.1: Plan of the column ring. The moment is shared in proportion to y_i , the distance of each column from the neutral axis.

13.1.1 The section modulus of a ring of columns

Treat the columns as a set of equal areas at radius R_c — which, the columns standing under the circular girder at the bottom of the cone, is r and not R . Each column at an angle θ_i from the wind direction is at a distance $y_i = R_c \cos \theta_i$ from the neutral axis. Then

$$\sum y_i^2 = R_c^2 \sum \cos^2 \theta_i \quad (13.1)$$

and for n columns equally spaced, $\sum \cos^2 \theta_i = n/2$ regardless of where the wind comes from — which is why the answer does not depend on the wind direction.

Column load from the overturning moment

$$\sum y^2 = \frac{n}{2} R_c^2, \quad \Delta P = \frac{M R_c}{\sum y^2} = \frac{2M}{n R_c} \quad (13.2)$$

so

$$P_{\text{leeward}} = \frac{P}{n} + \Delta P, \quad P_{\text{windward}} = \frac{P}{n} - \Delta P \quad (13.3)$$

R_c is the column circle, not the tank radius

$\Delta P = 2M/(nR_c)$ has R_c in the *denominator*: a narrower column ring makes the overturning effect on each column *larger*, not smaller. Using R where $R_c = r$ belongs understates the windward uplift and the leeward load by the factor R/r — about 50 percent for normal proportions — which is exactly the wrong direction to err in.

Read backwards, the same equation is the most reliable way to recover the column circle from an existing drawing's load table: $R_c = 2M/(n \Delta P)$. Chapter 1 applies it to RDSO B-1235.

Why $\sum \cos^2 \theta_i = n/2$

Because $\cos^2 \theta = \frac{1}{2}(1 + \cos 2\theta)$, and summing $\cos 2\theta_i$ over n equally spaced angles gives zero for any $n \geq 3$. So $\sum \cos^2 \theta_i = n/2$ exactly. The neat consequence is that the column group has the same section modulus in every direction, so the worst wind or seismic direction need not be searched for.

13.1.2 Shear and moment in each column

The base shear divides equally:

$$V_{\text{col}} = \frac{V}{n} \quad (13.4)$$

and within a panel the column bends in double curvature with a point of contraflexure at mid-height, so

Key result

$$M_{\text{col}} = \frac{V}{n} \cdot \frac{h_p}{2} \quad (13.5)$$

This is the portal-frame result and it assumes the bracings are stiff enough to enforce the contraflexure point. It is the standard assumption for a braced staging.

13.2 The load cases

Five service cases and six factored cases cover everything that matters.

13.2.1 Service combinations — for the foundation

The foundation is checked against the safe bearing capacity, which is a working-stress quantity, so unfactored loads are used.

Table 13.1: Service load combinations

#	Combination	Vertical	Lateral
S1	DL + water	$W_D + W_T$	none
S2	DL + water + wind	$W_D + W_T$	V_w, M_w
S3	DL + water + seismic, tank full	$W_D + W_T$	V_{eq}, M^*
S4	DL + seismic, tank empty	W_D	$V_{eq,e}, M_e$
S5	DL + wind, tank empty	W_D	V_w, M_w

S1 usually gives the largest *average* pressure; S3 gives the largest *maximum* pressure; and S4 or S5 give the largest *uplift*, because the vertical load that holds the tank down is at its smallest while the lateral load is unchanged.

The tank-empty cases are not academic

A service reservoir spends a good deal of its life partly or wholly empty — during commissioning, during cleaning, during a supply failure, and overnight in many systems. An empty tank in

a Zone IV earthquake with the full wind blowing is an entirely credible state, and it is the case that produces net tension in the windward column. Leave S4 and S5 out and the foundation may be designed for the wrong thing altogether.

13.2.2 Factored combinations — for the members

Table 13.2: Factored combinations, after IS 456 Table 18 and IS 1893

#	Combination	Comment
U1	$1.5(DL + W_T)$	maximum axial, no moment
U2	$1.2(DL + W_T + EQ \text{ full})$	the three-way combination
U3	$1.5(DL + W_T) + 1.5 EQ \text{ full}$	usually governs the column
U4	$1.5(DL + W_T) + 1.5 \text{ wind}$	
U5	$0.9 DL + 1.5 EQ, \text{ tank empty}$	governs uplift and the bracings
U6	$0.9 DL + 1.5 \text{ wind, tank empty}$	

Why 0.9 on the dead load in U5 and U6

When the dead load *helps* — as it does against overturning and uplift — it must not be relied on at its full value, because it might be less than calculated. The 0.9 factor is the code’s recognition of that. It is the mirror image of the 1.5 used when the dead load is adverse.

13.2.3 Treating the water

The water is not an imposed load in the ordinary sense. It is present for most of the structure’s life, it is fully accounted for in the seismic masses, and the code treats it as a dead load for combination purposes. That is why W_T appears with the same factor as the dead load in the table above, and why there is no separate “live load” combination for it.

13.3 Which case governs what

Design quantity	Governing case, typically
Maximum column axial load	U3 (full tank plus seismic)
Maximum column moment	U3, or U5 if the empty-tank period is on the spectrum plateau
Bracing beam moment	U5, because the shear is largest relative to the axial restraint
Maximum bearing pressure	S3
Uplift on the windward column	S4 or S5
Girder bending, shear, torsion	the gravity case, U1 — lateral load reaches the girder only as the column reactions, which are self-equilibrating around the ring
Girder hoop force	the full-tank service case, from the Intze balance of Chapter 12
Ring beam B_2	the full-tank service case
Container reinforcement	the full-tank service case; seismic adds only the hydrodynamic increment

Do not design every member for the same combination

It is tempting to find “the worst case” and use it everywhere. There is no such single case. The combination that maximises the column axial load minimises the uplift, and vice versa. Carry all eleven through the table and let each member pick its own.

13.4 Uplift

If P_{windward} comes out negative, the windward column is being pulled out of the ground. This changes the foundation problem completely:

- An isolated pad resists uplift only by its own weight plus the weight of the soil above it — and that has to exceed the uplift with a factor of safety of about 1.5. A small pad will not.
- Tying all the footings together with plinth beams in two directions lets the whole foundation act as one, which usually solves it and costs very little.
- An annular raft has the same advantage automatically, which is one reason it is preferred for tall staging.
- Where none of these is enough, the answer is tension piles or rock anchors.

Where this goes wrong

A negative windward load is flagged, not absorbed. Never let a spreadsheet quietly take the modulus of it and design for bearing.

Cross-check with the calculator

Section 6 of the booklet prints R_c — equal to r unless a manual column circle has been entered — then $\sum y^2$ and the ΔP formula, then two tables: the five service combinations and the six factored ones, each with P , V , M , the windward column load (shown in red with the word “uplift” if negative) and the leeward load. The governing row of each table is highlighted.

Staging Columns and Bracings

14.1 The column problem

Each staging column carries a large axial load and a moderate bending moment simultaneously. That combination cannot be designed by the flexure formula of Chapter 2, and it cannot be designed by the axial formula either. It needs an **interaction curve**: the locus of all the combinations of axial load and moment that the section can just carry.

This chapter builds that curve from first principles. It is more work than reading a chart from SP 16, and the point of doing it is that once you understand where the curve comes from, the chart stops being a black box and the edge cases — very high axial load, unusual bar arrangements, the neutral axis outside the section — stop being mysterious.

14.2 The interaction curve

14.2.1 Strain compatibility

Everything follows from one assumption: **plane sections remain plane**. The strain varies linearly across the depth of the section. A straight line needs two parameters, and IS 456 fixes one of them for each regime.

The two strain regimes of IS 456 Cl. 39

Neutral axis inside the section ($x_u \leq D$): the strain at the most compressed face is fixed at $\varepsilon_{cu} = 0.0035$, and the strain is zero at the neutral axis.

$$\varepsilon(z) = 0.0035 \frac{x_u - z}{x_u}$$

with z measured from the most compressed face.

Neutral axis outside the section ($x_u > D$), the whole section in compression: the code fixes the strain at a depth of $3D/7$ from the most compressed face at 0.002.

$$\varepsilon(z) = 0.002 \frac{x_u - z}{x_u - 3D/7}$$

Why $3D/7$ and 0.002

As $x_u \rightarrow \infty$ the strain becomes uniform, and the second expression correctly tends to a constant 0.002 — the strain at which the concrete stress block reaches its peak. At the other end, $x_u = D$ gives ε at the top of $0.002D/(D - 3D/7) = 0.002/(4/7) = 0.0035$, matching the first expression exactly. The two regimes join smoothly, which is the whole purpose of the rule.

14.2.2 Material stress–strain

Concrete, IS 456 Cl. 38.1

$$\sigma_c(\varepsilon) = \begin{cases} 0.446f_{ck} \left[2\frac{\varepsilon}{0.002} - \left(\frac{\varepsilon}{0.002}\right)^2 \right] & 0 \leq \varepsilon \leq 0.002 \\ 0.446f_{ck} & 0.002 < \varepsilon \leq 0.0035 \\ 0 & \varepsilon < 0 \end{cases} \quad (14.1)$$

The coefficient $0.446 = 0.67/1.5$, the characteristic strength reduced for sustained loading and then by the partial safety factor.

Steel, IS 456 Fig. 23A for cold-worked bars

Linear at $E_s = 2 \times 10^5 \text{ N/mm}^2$ up to $0.8 \times 0.87f_y$, then a curve to $0.87f_y$:

$\sigma_s/0.87f_y$	0.80	0.85	0.90	0.95	0.975	1.00
Strain, Fe 415	0.00144	0.00163	0.00192	0.00241	0.00276	0.00380
Strain, Fe 500	0.00174	0.00195	0.00226	0.00277	0.00312	0.00417

Interpolate linearly between the tabulated points. The curve is the same in tension and compression.

14.2.3 Integrating the section

For a chosen neutral axis depth x_u :

1. Divide the depth into strips. A hundred or so is ample.
2. For each strip, find the strain at its centroid from the appropriate regime, then the concrete stress, then the force $\sigma_c b dz$ and its moment about the centroid of the section.
3. For each bar, find the strain at its level, then the steel stress σ_s , and take the force as $(\sigma_s - \sigma_c)A_{\text{bar}}$ — subtracting the concrete stress because that concrete has already been counted in the strip integration and the bar displaces it.
4. Sum. The totals are one point (P_u, M_u) on the interaction curve.

The section resultants

$$P_u = \int_0^D \sigma_c(z) b dz + \sum_i [\sigma_s(z_i) - \sigma_c(z_i)] A_i \quad (14.2)$$

$$M_u = \int_0^D \sigma_c(z) b \left(\frac{D}{2} - z \right) dz + \sum_i [\sigma_s(z_i) - \sigma_c(z_i)] A_i \left(\frac{D}{2} - z_i \right) \quad (14.3)$$

Sweeping x_u from a small value to several times D traces the whole curve, from pure bending at one end to pure compression at the other.

14.2.4 Reading the curve

- The **balanced point** is where the concrete reaches 0.0035 at the same instant as the outermost steel reaches yield. It is the point of maximum moment capacity.
- **Above** the balanced point the failure is compression-controlled and brittle. Adding axial load

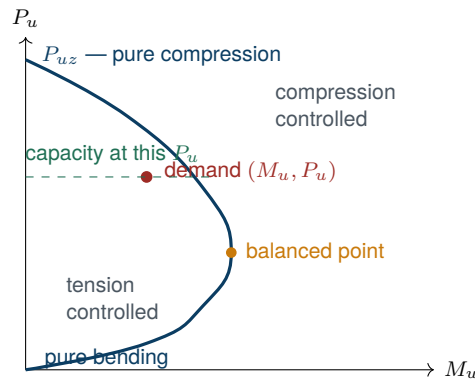


Figure 14.1: A column interaction curve. The section is adequate if the demand point lies inside it.

reduces the moment capacity. Staging columns normally sit here.

- **Below** it the failure is tension-controlled and ductile, and adding axial load *increases* the moment capacity — which surprises people the first time they see it.
- $P_{uz} = 0.45f_{ck}A_c + 0.75f_yA_{sc}$ is the pure-compression capacity, IS 456 Cl. 39.6.

14.2.5 Finding the required steel

Bisect on the steel percentage: guess a value, build the curve, read the moment capacity at the demand axial load, and compare with the demand moment. Sixteen bisection steps between 0.8 and 6 percent converge to three decimal places, which is more precision than the problem deserves.

Practical bounds

IS 456 Cl. 26.5.3 sets the minimum longitudinal steel in a column at 0.8 percent and the maximum at 6 percent. Six percent is unbuildable — the bars cannot be lapped. Keep the answer below 4 percent; if it exceeds that, the column is too small and should be enlarged, which is almost always cheaper than congested steel.

14.3 Slenderness

Effective length and the slenderness check

$$l_e = 1.2 h_p, \quad \frac{l_e}{b_c} \leq 12 \Rightarrow \text{short column} \quad (14.4)$$

The factor 1.2 is for an unbraced (sway) frame with reasonably stiff bracings. A staging is a sway frame in the global sense — there is nothing preventing the whole thing from leaning — so the braced-frame factor of 0.65 to 0.85 is not appropriate.

If the column is slender, IS 456 Cl. 39.7.1 requires an additional moment:

Key result

$$M_a = \frac{P_u b_c}{2000} \left(\frac{l_e}{b_c} \right)^2 \quad (14.5)$$

added to the primary moment before entering the interaction curve.

Keeping the columns short

The panel height controls both the slenderness and the moment. Halving the panel height quarters the column moment and halves the slenderness ratio. That is the real reason for bracings — not to reduce the seismic force, which they increase, but to keep each column short and lightly bent.

14.4 Detailing the columns

- Minimum four bars in a rectangular column; in practice eight or twelve, distributed on all four faces, so that the section has the same capacity in every direction.
- Minimum bar diameter 12 mm; 16 or 20 mm is usual for a staging.
- **Lateral ties:** diameter not less than one quarter of the largest longitudinal bar and not less than 6 mm; spacing not more than the least lateral dimension, sixteen times the smallest longitudinal bar diameter, or 300 mm, whichever is least [IS 456 Cl. 26.5.3.2].
- In Zones III, IV and V, IS 13920 requires **special confining reinforcement** over a length of at least the larger dimension of the member, one sixth of the clear height, or 450 mm, at each end of every column — at a spacing not exceeding one quarter of the minimum dimension, and not less than 75 mm. Ties must have 135° hooks with a 10-diameter extension. This is the single most important detailing requirement in the staging and it is the one most often skipped on site.
- Laps should be staggered and located in the middle half of the panel height, never at the joints.

14.5 Bracing beams**14.5.1 The moment**

At a joint where a column meets bracings, equilibrium requires the moments in the columns above and below to be balanced by the moments in the bracings framing in. With contraflexure at mid-panel, the column moment on each side of the joint is $V_{\text{col}} h_p / 2$, so the total moment arriving at the joint from the columns is

$$M_{\text{joint}} = 2 \times \frac{V_{\text{col}} h_p}{2} = V_{\text{col}} h_p \quad (14.6)$$

Two bracings meet at each joint, and they are not in line — they run along the chords of the column circle, at an angle of π/n each side of the tangent. Resolving:

Design moment in one bracing

$$M_{\text{brace}} = \frac{M_{\text{joint}}}{2 \cos(\pi/n)} = \frac{V_{\text{col}} h_p}{2 \cos(\pi/n)} \quad (14.7)$$

The moment is hogging at both ends of the bracing and sagging at mid-span, so the beam needs equal steel top and bottom — which is convenient, because it also means one detail serves for wind or earthquake from any direction.

14.5.2 Shear

The bracing is bent in double curvature with a moment M_{brace} at each end and no significant gravity load, so the shear is constant along it and equal to the sum of the end moments divided by the span:

$$V_u = \frac{2M_{\text{brace}}}{L_{ch}} \quad (14.8)$$

where L_{ch} is the chord from equation (5.5). Provide two-legged stirrups designed for this, closed and with 135° hooks, closely spaced near the joints where IS 13920 requires confinement over twice the effective depth.

14.5.3 Sizing

A bracing should be at least as deep as it is wide and preferably deeper — 300×600 mm is typical for a 20 metre staging. A shallow bracing is flexible, which erodes the staging stiffness assumed in Chapter 7 and pushes β down.

Bracings are seismic members, not architectural ones

They are sometimes drawn as nominal tie beams and detailed accordingly. They are not. They carry the full joint moment, they determine the column's effective length, and in an earthquake they are where the frame's energy dissipation has to happen. Detail them to IS 13920 with the same care as the columns.

Cross-check with the calculator

Section 9 of the booklet gives the governing factored combination, P_u and the portal moment, the effective length and slenderness with the additional moment if any, a statement of how the interaction curve was built, the required steel percentage and bar arrangement, the moment capacity achieved at that P_u against the demand, P_{uz} , and the tie arrangement including the IS 13920 confinement. Section 9.2 does the same for the bracings.

Foundation

15.1 Choosing the type

Type	Suits	Watch out for
Isolated footings with plinth beams	good soil, short staging, low overturning	uplift on the windward footing; differential settlement between footings, which twists the staging
Annular (ring) raft	the usual choice for staging over about 12 m	needs a competent bearing stratum over the whole ring; heavier and more reinforcement
Circular raft	poor soil, or where the ring raft is so wide that the hole in the middle is small	a great deal of concrete
Pile cap on piles	soft soil, high water table, or net uplift that cannot be resisted otherwise	outside the scope of this manual

The deciding questions are: does any service case produce uplift, and is the soil uniform under the whole footprint? A yes to the first or a no to the second points to a raft.

15.2 Net pressure and net bearing capacity

The most common foundation error in tank design

Comparing a *gross* pressure with a *net* safe bearing capacity. They are not the same quantity and the difference is large.

Net safe bearing capacity

The *net* safe bearing capacity is the intensity of pressure *in excess of* the existing overburden pressure at founding level that the soil can carry safely. A geotechnical report normally quotes the net value; it should say so, and if it does not, ask.

The soil at founding level already carries $\gamma_{\text{soil}} D_f$ from the ground above it before anything is built. Excavating removes that; the structure then puts it back, plus its own load. What the soil experiences as a *new* pressure is therefore

Net bearing pressure

$$p_{\text{net}} = \frac{P_{\text{total}}}{A} - \gamma_{\text{soil}} D_f \pm \frac{M}{Z} \quad (15.1)$$

where P_{total} includes the load from the staging, the self weight of the raft, and the weight of the backfill standing on the raft.

For the worked example the overburden deduction is $18 \times 2.5 = 45 \text{ kN/m}^2$ against a gross pressure of 127 — more than a third. Ignore it and the foundation comes out a third too large.

15.2.1 The permissible increase for wind and seismic

IS 1904 and IS 6403 allow the safe bearing capacity to be increased for load combinations including wind or earthquake, because these are transient and the soil's response to a short-duration load is stiffer. Twenty-five percent is usual for most soils. It is **not** allowed for soft clays and loose saturated sands, where the transient load can cause more settlement rather than less, and it is never allowed for the gravity-only combination.

Combination	Allowable	Reason
S1 gravity only	q_{safe}	no transient component
S2 to S5 with wind or seismic	$1.25 q_{\text{safe}}$	transient loading

The consequence is that the *gravity* case often governs the foundation size even though the seismic case produces a larger pressure, because the seismic case is allowed 25 percent more. Check the ratio $p_{\text{max}}/p_{\text{allowable}}$ for every case, not the pressure alone.

15.3 Annular raft

15.3.1 Section properties

With the columns on a circle of radius R_c and the raft projecting c each side:

$$R_o = R_c + c, \quad R_i = R_c - c \quad (15.2)$$

$$A = \pi (R_o^2 - R_i^2), \quad I = \frac{\pi}{4} (R_o^4 - R_i^4), \quad Z = \frac{I}{R_o} \quad (15.3)$$

15.3.2 Pressures

For each service combination:

$$p_{\text{max,min}} = \frac{P + W_{\text{raft}} + W_{\text{soil}}}{A} - \gamma_{\text{soil}} D_f \pm \frac{M}{Z} \quad (15.4)$$

and two things are checked:

1. $p_{\text{max}} \leq$ the allowable for that case;
2. $p_{\text{min}} \geq 0$ — no tension, because soil cannot pull.

If p_{min} goes negative

Part of the raft has lifted off and the whole linear-pressure assumption is invalid. Strictly the problem must be re-solved with a partially effective bearing area. Practically, widen the raft until the resultant falls inside the middle third and p_{min} is positive — which for a ring is approximately the condition

$$\frac{M}{P} \leq \frac{Z}{A} = \frac{R_o^4 - R_i^4}{4R_o(R_o^2 - R_i^2)}.$$

A ring raft is very efficient at this, because almost all of its area is at a large radius, so its section modulus per unit area is high. That is its main advantage over a circular raft of the same area.

15.3.3 Reinforcement

The raft cantilevers outward from the ring of columns and inward towards the hole, so it needs *radial* steel at the top and bottom, plus *circumferential* distribution steel.

Radial cantilever moment, per metre of circumference

$$M_u = 1.5 p_{\max} \frac{c^2}{2} \quad (15.5)$$

treating the projection as a cantilever carrying the peak pressure — conservative, since the pressure falls away towards the edge.

Design for this with equation (2.5) and provide not less than 0.12 percent of the gross section as distribution steel both ways, at both faces. The raft is a soil-facing member and is not liquid-retaining, so IS 456 limit state applies with the usual 50 mm cover against blinding.

15.3.4 Thickness

Governed by shear rather than moment. Check:

- **One-way shear** at a distance d from the face of the column line;
- **Punching shear** on a perimeter at $d/2$ around each column, with $\tau_c = 0.25\sqrt{f_{ck}}$ — this is usually the critical check for a ring raft under a heavily loaded column;
- Minimum thickness 300 mm, and 500 to 900 mm for a tank of ordinary size.

15.4 Isolated footings**15.4.1 Sizing**

$$A_{\text{req}} = \frac{P_{\max, \text{service}}}{q_{\text{allowable}}}, \quad L = \sqrt{A_{\text{req}}} \text{ for a square pad} \quad (15.6)$$

using the leeward column load from the governing service case and the allowable appropriate to that case.

15.4.2 Uplift

Check the windward column in every service case. If $P_{\min} < 0$:

$$\text{Factor of safety against uplift} = \frac{W_{\text{footing}} + W_{\text{soil above}}}{|P_{\min}|} \geq 1.5 \quad (15.7)$$

If that is not satisfied, the options in order of cost are: tie the footings with plinth beams so that the group acts together; enlarge the pad and deepen it so that more soil sits on it; go to a ring raft; or provide tension piles.

15.4.3 Flexure

$$M_u = 1.5 p \frac{(L - b_c)^2}{8} \text{ per metre width, each way} \quad (15.8)$$

with p the net upward pressure. Design with equation (2.5), bars both ways at the bottom, and check punching shear around the column.

15.5 Plinth beams — the cheapest thing you will ever specify

Recommendation

Tie all the footings together with reinforced concrete plinth beams in two directions at the top of the foundation, regardless of the foundation type or the seismic zone.

They cost perhaps two percent of the foundation and they do four things: they prevent differential settlement from twisting the staging; they distribute uplift from one footing to the group; they give the columns a defined base level and fixity; and they are required by IS 1893 for structures in Zones IV and V. There is no argument against them.

15.6 Settlement

Bearing capacity is only half the problem; the other half is settlement, and for a tall slender structure *differential* settlement matters far more than the absolute value. A differential settlement of 20 mm across a 12 metre column ring tilts the tank by 1 in 600, which puts the top of a 25 metre tank 42 mm out of plumb and adds a permanent overturning moment of $P \times 0.042$ — for the worked example, 350 kN m, comparable with a fifth of the wind moment.

IS 1904 limits for a raft on clay are 65 to 100 mm total and about 1 in 500 angular distortion. A ring raft under a water tank should be designed to do much better than that, and the geotechnical report should be asked for a settlement estimate, not just a bearing capacity.

Cross-check with the calculator

Section 10 of the booklet prints, for a ring raft: R_o , R_i , A , I and Z ; the raft and soil weights and the overburden deduction; a table of all five service combinations with the gross pressure, the net maximum and minimum, the allowable for that case, and a pass or fail chip on each; and then the cantilever moment and the radial steel. For isolated footings it prints the maximum and minimum column loads, the required and provided areas, the actual pressure, the flexural steel, and — if the minimum is negative — an explicit uplift warning with the options above.

Worked Example

16.1 The brief

Design data

Capacity	450 kilolitres, net
Staging height	20.0 m, top of foundation to the circular girder
Location	Barasat, North 24 Parganas, West Bengal
Seismic zone	III, $Z = 0.16$; medium soil, Type II; $I = 1.5$, $R = 2.5$
Basic wind speed	47 m/s; terrain category 2; non-cyclonic
Soil	net safe bearing capacity 150 kN/m ² at 2.5 m depth; unit weight 18 kN/m ³
Materials	M30 concrete, Fe 500 steel, clear cover 45 mm
Permissible steel stress	150 N/mm ² in direct tension and in bending on the liquid face
Columns	8 nos, on the circular girder at the bottom of the cone
Foundation	annular raft, 2.2 m projection each side, 700 mm thick
Freeboard	0.50 m
Ratios	$H_1/D = 0.32$; $r/R = 0.66$; cone slope 45°; top dome rise $0.165D$; bottom dome rise $0.25r$

Throughout this chapter, $\gamma_c = 25 \text{ kN/m}^3$, $\gamma_w = 9.81 \text{ kN/m}^3$, and for M30: $\sigma_{ct} = 1.5$, $\sigma_{cc} = 8.0$, $\sigma_{cbc} = 10.0 \text{ N/mm}^2$, $m = 280/30 = 9.333$, so $m - 1 = 8.333$.

Why these ratios

They are close to RDSO standard drawing B-1235, an issued drawing for a tank of this size, so the answers can be compared against something that has actually been built. Appendix E does that comparison line by line.

16.2 Step 1 — Geometry

STEP 16.1 Solve for the radius

With $k_H = 0.32$, $k_r = 0.66$, $\theta_c = 45^\circ$, $k_b = 0.25$ and $f = 0.50 \text{ m}$, equation (4.3) gives $V(R) = AR^3 - \pi f R^2$ with

$$\begin{aligned}
 A &= 2\pi(0.32) + \frac{\pi(0.34)(1)}{3} (1 + 0.66 + 0.4356) - \frac{\pi(0.25)(0.66)^3}{6} (3 + 0.0625) \\
 &= 2.0106 + 0.7461 - 0.1153 = 2.6415
 \end{aligned}$$

Note the *minus* sign on the third term: the bottom dome arches up into the tank and its bulge displaces water. Solving $2.6415R^3 - 1.5708R^2 = 450$ by bisection:

$$R = 5.7490 \text{ m}, \quad D = 11.4980 \text{ m}$$

STEP 16.2 Derived dimensions

$H_1 = 0.32D$	3.6794 m	$r = 0.66R$	3.7944 m
$h_w = H_1 - f$	3.1794 m	$h_2 = (R - r) \tan 45^\circ$	1.9547 m
$h_t = 0.165D$	1.8972 m	$h_b = 0.25r$	0.9486 m
$L_s = \sqrt{(R - r)^2 + h_2^2}$	2.7643 m	$\alpha = 90^\circ - 45^\circ$	45°
$R_{d,t} = (R^2 + h_t^2)/2h_t$	9.6592 m	$R_{d,b} = (r^2 + h_b^2)/2h_b$	8.0630 m
$\sin \phi_{0,t} = R/R_{d,t}$	0.5952	$\sin \phi_{0,b} = r/R_{d,b}$	0.4706
$\cos \phi_{0,t}$	0.8036	$\cos \phi_{0,b}$	0.8824
Column circle = $2r = 7.589$ m, at the bottom of the conical slab			

STEP 16.3 Capacity check

$$V_1 = \pi(5.7490)^2(3.1794) = 330.13 \text{ m}^3$$

$$V_2 = \frac{\pi(1.9547)}{3} (33.051 + 21.814 + 14.398) = 141.77 \text{ m}^3$$

$$V_3 = \frac{\pi(0.9486)}{6} (3 \times 14.398 + 0.900) = 21.90 \text{ m}^3 \text{ (displaced)}$$

$$V_1 + V_2 - V_3 = 450.00 \text{ m}^3 \quad \text{against 450 required } \checkmark$$

STEP 16.4 Surface areas

$A_{td} = 2\pi(9.6592)(1.8972) = 115.14 \text{ m}^2$; $A_w = 2\pi(5.7490)(3.6794) = 132.91 \text{ m}^2$; $A_c = \pi(5.7490 + 3.7944)(2.7643) = 82.88 \text{ m}^2$; $A_{bd} = 2\pi(8.0630)(0.9486) = 48.06 \text{ m}^2$.

16.3 Step 2 — Dead loads

Trial thicknesses: top dome 100, wall 150, cone 200, bottom dome 200 mm. Ring beams B₁ 320 × 210, B₂ 620 × 370, girder B₃ 550 × 720 mm.

Element	Area or volume	t or section	Weight, kN	z above girder, m
Top dome	115.14 m ²	100 mm	287.85	6.583
Ring beam B ₁	$2\pi(5.749)$	320 × 210	60.69	5.634
Cylindrical wall	132.91 m ²	150 mm	498.40	3.794
Ring beam B ₂	$2\pi(5.749)$	620 × 370	207.16	1.955
Conical slab	82.88 m ²	200 mm	414.39	1.044
Circular girder B ₃	$2\pi(3.794)$	550 × 720	236.02	0
Bottom dome	48.06 m ²	200 mm	240.28	0.474
Shells and rings			1944.79	
Finishes, 8 percent of the shells			155.58	
Container dead load $W_{DL,c}$			2100.38	$\bar{z}_c = 2.612$
Roof live load, 1.5 kN/m ²	103.83 m ² plan		155.75	
Water W_T	450 m ³		4414.50	$\bar{z}_w = 2.932$
Columns, 8 × 20 × 0.55 ² × 25		550 sq	1210.00	
Bracings, 5 levels, 400 × 700			813.14	
Staging dead load			2023.14	

STEP 16.5 Container centre of gravity

Every z is positive, measured up from the girder, because the whole container stands above it:

$$\begin{aligned}\Sigma W_i z_i &= 287.85(6.583) + 60.69(5.634) + 498.40(3.794) \\ &\quad + 207.16(1.955) + 414.39(1.044) + 240.28(0.474) = 5079 \text{ kN m} \\ \bar{z}_c &= \frac{5079}{1944.79} \text{ (with finishes taken at the wall mid-height)} = 2.612 \text{ m}\end{aligned}$$

so $h_{cg} = 20.0 + 2.612 = 22.612 \text{ m}$ above the foundation.

STEP 16.6 Water centre of gravity

$$\bar{z}_w = \frac{330.13 \left(1.955 + \frac{3.179}{2}\right) + 141.77(1.109) - 21.90(0.356)}{450.00} = 2.932 \text{ m}$$

so the water acts at $20.0 + 2.932 = 22.932 \text{ m}$. Note the displaced segment entering with a minus sign in both numerator and denominator.

STEP 16.7 Totals

$$W_{DL} = 2100.38 + 2023.14 = 4123.52 \text{ kN}, \quad W_{\text{full}} = 4123.52 + 4414.50 = 8538.02 \text{ kN}$$

16.4 Step 3 — Wind**STEP 16.8 Container**

Top of container = $20.0 + 1.955 + 3.679 + 1.897 = 27.53 \text{ m}$. With $V_b = 47$, $k_1 = 1.0$, $k_2 = 1.1077$ at that height in terrain 2, $k_3 = k_4 = 1.0$:

$$V_z = 47(1.1077) = 52.06 \text{ m/s}, \quad p_z = 0.6(52.06)^2 = 1626 \text{ N/m}^2 = 1.626 \text{ kN/m}^2$$

With $K_a = 0.848$ for 64.1 m^2 and $K_d = 1.0$ for a circular shape:

$$p_d = K_d K_a p_z = 1.0(0.848)(1.626) = 1.3787 \text{ kN/m}^2, \quad C_f = 0.70$$

$$F_{\text{cyl+dome}} = 1.3787 \times 64.12 = 61.88 \text{ kN at } z = 24.74 \text{ m}$$

$$F_{\text{cone}} = 1.3787 \times 18.65 = 18.00 \text{ kN at } z = 21.04 \text{ m}$$

STEP 16.9 Staging

At the top of the staging, $k_2 = 1.070$, $V_z = 50.29 \text{ m/s}$, $p_z = 1.517 \text{ kN/m}^2$. With $K_d = 0.90$ for a framed structure and $K_a = 0.839$, $p_d = 1.146 \text{ kN/m}^2$. The frame's projected solid area is 70.56 m^2 within an enveloping area of 151.77 m^2 , so the solidity ratio $\phi = 0.465$ and $C_f = 1.6175$ including the shielding allowance for the leeward columns:

$$F_{\text{staging}} = C_f A p_d (1 + \eta) = 1.6175(70.56)(1.1462)(1.45) = 189.69 \text{ kN at } z = 10.0 \text{ m}$$

with $\eta = 0.45$, the shielding allowed for the leeward frame.

STEP 16.10 Totals

$$V_{\text{wind}} = 61.88 + 18.00 + 189.69 = 269.57 \text{ kN}$$

$$M_{\text{wind}} = 61.88(24.74) + 18.00(21.04) + 189.69(10.0) = 3806.8 \text{ kN m}$$

16.5 Step 4 — Seismic

STEP 16.11 Equivalent cylinder

[IS 1893(2) Cl. 4.2.3] For an Intze the two-mass model is set up on a cylinder of the same diameter holding the same volume:

$$h_{eq} = \frac{V}{\pi R^2} = \frac{450}{\pi(5.749)^2} = 4.334 \text{ m}, \quad \frac{h}{D} = \frac{4.334}{11.498} = 0.3769$$

STEP 16.12 Masses

From the curves of Cl. 4.2.1 at $h/D = 0.377$:

$$\frac{m_i}{m} = 0.4265 \Rightarrow m_i = 191.94 \text{ t}, \quad \frac{m_c}{m} = 0.5385 \Rightarrow m_c = 242.33 \text{ t}$$

$$m_s = \frac{2100.38 + \frac{1}{3}(2023.14)}{9.81} = 282.85 \text{ t}$$

STEP 16.13 Heights

$$h_i/h = 0.375, \quad h_c/h = 0.5673, \quad h_i^*/h = 1.0472, \quad h_c^*/h = 0.9553$$

Measured from the foundation, and using the *starred* heights for the overturning moment as Cl. 4.7.2 requires:

$$h_{Mi}^* = 24.539 \text{ m}, \quad h_{Mc}^* = 24.140 \text{ m}$$

Where this goes wrong

The unstarred values, 21.625 and 22.458 m, are for the base shear's point of application, not for the moment at the foundation. Using them here understates the overturning moment by about 12 percent.

STEP 16.14 Staging stiffness and periods

8 columns of 550 mm square, 6 panels of 3.333 m between 5 bracing levels:

$$I = \frac{0.55^4}{12} = 7.626 \times 10^{-3} \text{ m}^4, \quad E_c = 5000\sqrt{30} = 27\,386 \text{ MPa}$$

$$K_s = \frac{12 n E_c I N_p^2}{h_s^3} \beta = 76\,684 \text{ kN/m}$$

$$T_i = 2\pi \sqrt{\frac{m_i + m_s}{K_s}} = 2\pi \sqrt{\frac{191.94 + 282.85}{76\,684}} = 0.4944 \text{ s}$$

$$T_c = 2\pi \sqrt{\frac{D}{3.68g \tanh(3.68h/D)}} = 3.7746 \text{ s}$$

Tank empty, $T_{i,e} = 0.3816 \text{ s}$.

STEP 16.15 Seismic coefficients

$Z = 0.16$, $I = 1.5$, $R = 2.5$, soil type II.

$$\left(\frac{S_a}{g}\right)_i = 2.50 \text{ at } T_i = 0.494 \text{ s} \quad \Rightarrow \quad (A_h)_i = \frac{0.16}{2} \cdot \frac{1.5}{2.5} \cdot 2.50 = 0.1200$$

$$\left(\frac{S_a}{g}\right)_c = 0.3603 \text{ at } T_c = 3.775 \text{ s, multiplied by 1.75 for 0.5\% damping}$$

$$(A_h)_c = \frac{0.16}{2} \cdot \frac{1.5}{2.5} \cdot 0.3603 \times 1.75 = 0.0303$$

[Cl. 4.5.4 — the convective mode is taken at 0.5 percent damping, the impulsive at 5 percent]

STEP 16.16 Base shear

$$V_i = (A_h)_i (m_i + m_s) g = 0.1200 (191.94 + 282.85) (9.81) = 558.93 \text{ kN}$$

$$V_c = (A_h)_c m_c g = 0.0303 (242.33) (9.81) = 71.95 \text{ kN}$$

$$V = \sqrt{V_i^2 + V_c^2} = \sqrt{558.93^2 + 71.95^2} = 563.54 \text{ kN}$$

STEP 16.17 Overturning moment

$$M_i = (A_h)_i [m_i h_i^* + m_s h_{s,\text{eff}}] g = 0.12 [191.94(24.539) + 282.85(19.547)] 9.81 = 12\,053.1 \text{ kN m}$$

$$M_c = (A_h)_c m_c h_c^* g = 1736.9 \text{ kN m}$$

$$M = \sqrt{M_i^2 + M_c^2} = 12\,177.6 \text{ kN m}$$

STEP 16.18 Tank empty

$V_e = 332.97 \text{ kN}$, $M_e = 6508.4 \text{ kN m}$.

STEP 16.19 Sloshing

[Cl. 4.11]

$$d_{\max} = (A_h)_c R \frac{D}{2} = 0.0303 (2.5) \frac{11.498}{2} = 0.435 \text{ m}$$

against the freeboard of 0.500 m. ✓ It is close; a freeboard of 0.45 m would have failed.

STEP 16.20 Which governs

Seismic, decisively: 563.5 kN against 269.6 kN of wind, and 12 178 kN m against 3807 kN m. Both are carried through the combinations all the same, because the wind case has a different load factor on the gravity part and can govern the foundation in some soils.

16.6 Step 5 — Container design**16.6.1 Top dome and ring beam B₁**

Self weight $0.100 \times 25 = 2.5$, finishes 0.5, live 1.5, total $w = 4.5 \text{ kN/m}^2$ — take 4.0 kN/m^2 on the surface with the live load treated separately in the girder.

$$N_\phi = \frac{w R_{d,t}}{1 + \cos \phi_{0,t}} = \frac{4.0(9.6592)}{1.8036} = 21.42 \text{ kN/m (compression)}$$

$$\sigma = \frac{21.42}{100} = 0.214 \text{ N/mm}^2 \ll 8.0 \checkmark$$

$$N_\theta = wR_{d,t} \left(\cos \phi_0 - \frac{1}{1 + \cos \phi_0} \right) = 9.63 \text{ kN/m, compression}$$

Minimum steel governs: 240 mm²/m each way, 8 mm at 205 mm c/c, provided 245 mm²/m.

Horizontal thrust onto B₁: the meridional force acts along the tangent, so its horizontal component is $H = N_\phi \cos \phi_{0,t} = 21.42(0.8036) = 17.21 \text{ kN/m}$ outward, giving a hoop tension

$$P_1 = 17.215 \times 5.749 = 98.97 \text{ kN}$$

$$A_{st} = \frac{98.97 \times 10^3}{150} = 660 \text{ mm}^2 \rightarrow 9 \text{ nos } 10 \text{ mm} = 707 \text{ mm}^2$$

$$\sigma_c = \frac{98.97 \times 10^3}{67\,200 + 8.333(707)} = 1.36 \leq 1.5 \checkmark$$

16.6.2 Cylindrical wall

Maximum hoop tension at the base of the cylinder, $y = h_w = 3.1794 \text{ m}$:

$$F = \gamma_w h_w R = 9.81(3.1794)(5.749) = 179.31 \text{ kN/m}$$

$$A_{st} = \frac{179.31 \times 10^3}{150} = 1195 \text{ mm}^2/\text{m} \rightarrow 10 \text{ mm at } 130 \text{ c/c both faces}$$

$$t_{\text{req}} = 179.31 \left(\frac{1}{1.5} - \frac{8.333}{150} \right) = 109.6 \text{ mm} \leq 150 \checkmark$$

$$\sigma_c = \frac{179.31 \times 10^3}{150\,000 + 8.333(1195)} = 1.12 \leq 1.5 \checkmark$$

Vertical steel: IS 3370 Cl. 8.1 minimum, 360 mm²/m total, 10 mm at 300 c/c each face.

16.6.3 Conical slab

Load path

The circular girder and the columns are at the *bottom* of this slab, on the 7.589 m circle. The slab therefore stands, carrying everything above it down in **meridional compression**, with hoop tension across it. The hoop tension is what fixes the thickness.

STEP 16.21 Load standing on the cone, at its top edge

Everything above the cone reaches the girder through it: top dome 287.85, B₁ 60.69, wall 498.40, B₂ 207.16, the finishes those four carry (8% of 1054.10 = 84.33) and the roof live load 155.75. Total $W_{\text{top}} = 1294.18 \text{ kN}$.

$$N_{\phi, \text{top}} = \frac{1294.18}{2\pi(5.749) \cos 45^\circ} = 50.67 \text{ kN/m (compression)}$$

STEP 16.22 Load at the bottom edge, at the girder

Add the cone's own weight, 414.39 kN, and the water resting on the conical surface,

$$W_{\text{wat,cone}} = 9.81 [330.13 + 141.77 - \pi(3.7944)^2(3.1794 + 1.9547)] = 2351.33 \text{ kN}$$

$$W_{\text{bot}} = 1294.18 + 414.39 + 2351.33 = 4059.90 \text{ kN}$$

$$N_{\phi,\text{bot}} = \frac{4059.90}{2\pi(3.7944) \cos 45^\circ} = 240.83 \text{ kN/m (compression)}$$

$$\sigma_{cc} = \frac{240.83}{200} = 1.20 \text{ N/mm}^2 \leq 8.0 \checkmark$$

STEP 16.23 Hoop tension

$$q = 0.200 \times 25 = 5.0 \text{ kN/m}^2.$$

$$N_{\theta,\text{top}} = \left(\frac{9.81(3.1794)}{\cos 45^\circ} + 5.0 \tan 45^\circ \right) 5.749 = 282.33 \text{ kN/m}$$

$$N_{\theta,\text{bot}} = \left(\frac{9.81(3.1794 + 1.9547)}{\cos 45^\circ} + 5.0 \tan 45^\circ \right) 3.7944 = 289.23 \text{ kN/m}$$

The bottom governs, by three percent.

STEP 16.24 Steel and thickness

$$A_{st,h} = \frac{289.23 \times 10^3}{150} = 1928 \text{ mm}^2/\text{m} \rightarrow 10 \text{ mm at } 80 \text{ c/c both faces}$$

$$t_{\text{req}} = \max \left[289.23 \left(\frac{1}{1.5} - \frac{8.333}{150} \right), \frac{240.83}{8.0}, 200 \right] = \max[176.8, 28.3, 200] = 200 \text{ mm} \checkmark$$

Meridional steel is the Cl. 8.1 minimum, 480 mm²/m, 10 mm at 300 c/c each face.

STEP 16.25 Horizontal components

$$H_{\text{out, onto } B_2} = 50.67 \sin 45^\circ = 35.83 \text{ kN/m outward}$$

$$H_{\text{in, onto } B_3} = 240.83 \sin 45^\circ = 170.29 \text{ kN/m inward}$$

16.6.4 Ring beam B₂ at the wall–cone junction

Both actions push outward, so this ring is in pure hoop tension:

$$w = \underbrace{9.81(3.1794)(0.370)}_{11.54} + \underbrace{35.83}_{\text{cone}} = 47.37 \text{ kN/m}$$

$$P_2 = 47.37 \times 5.749 = 272.32 \text{ kN}$$

$$A_{st} = \max \left(\frac{272.32 \times 10^3}{150}, 0.24\% \times 229\,400 \right) = \max(1815, 551) = 1815 \text{ mm}^2$$

→ 10 nos 16 mm = 2011 mm², in staggered closed laps.

$$\sigma_c = \frac{272.32 \times 10^3}{229\,400 + 8.333(2011)} = 1.11 \leq 1.5 \checkmark$$

16.6.5 Bottom dome

The dome springs from the girder and arches *up* into the tank.

$$W_{bd} = 9.81 [\pi(3.7944)^2(5.1341) - 21.90] + 240.28 = 2303.45 \text{ kN}$$

$$N_\phi = \frac{2303.45}{2\pi(3.7944)(0.4706)} = 205.31 \text{ kN/m (compression)}$$

$$p_n = 9.81(5.1341) = 50.36 \text{ kN/m}^2, \quad N_\theta = 50.36(8.0630) - 205.31 = 200.78 \text{ kN/m (compression)}$$

$$\sigma_{cc} = \frac{205.31}{200} = 1.03 \leq 8.0 \checkmark$$

Both forces compressive, so no crack check applies; provide the Cl. 8.1 minimum of 480 mm²/m both ways, 10 mm at 300 c/c each face. Thickness required 205.31/8.0 = 25.7 mm against a practical floor of 120 mm; 200 mm adopted for stiffness and buildability.

$$H_{\text{out}} = 205.31 \cos \phi_{0,b} = 205.31(0.8824) = 181.16 \text{ kN/m outward}$$

16.6.6 The Intze balance at the girder

Source	kN/m	Direction
Water on the beam face, 9.81(5.1341)(0.720)	36.26	outward
Bottom dome	181.16	outward
Conical slab	170.29	inward
Net	47.13	outward

$$P = 47.13 \times 3.7944 = 178.83 \text{ kN, hoop tension}$$

$$A_{st} = \frac{178.83 \times 10^3}{150} = 1192 \text{ mm}^2 \rightarrow 11 \text{ nos } 12 \text{ mm} = 1244 \text{ mm}^2$$

distributed around the perimeter of the girder section, and

$$\sigma_c = \frac{178.83 \times 10^3}{396\,000 + 8.333(1244)} = 0.44 \leq 1.5 \checkmark$$

The two shell thrusts differ by

$$\frac{|181.16 - 170.29|}{170.29} = 6.4 \text{ percent}$$

which is comfortably inside the 15 percent band of Chapter 12. Without the balance, the dome's 181 kN/m alone would put $181.16 \times 3.794 = 687 \text{ kN}$ of hoop tension into the girder — nearly four times what it actually carries.

STEP 16.26 What rise would balance exactly

$$\phi_0 = \arctan \left(\frac{W_{bd}}{2\pi r H_{\text{in}}} \right) = \arctan \left(\frac{2303.45}{2\pi(3.7944)(170.29)} \right) = \arctan(0.5674) = 29.57^\circ$$

$$h_b = r \tan \frac{\phi_0}{2} = 3.7944 \tan(14.79^\circ) = 1.0014 \text{ m} = 0.264 r$$

against the 0.250r adopted. Close enough to leave alone; adopting 0.264 would reduce the residual to near zero but changes the capacity solution slightly and is not worth a second pass here.

Compare with the issued drawing

RDSO B-1235 uses $h_b/r = 0.245$ on almost exactly these proportions. Our adopted 0.250 and our computed balancing 0.264 sit either side of it, within six percent. Had the signs been the other way round — the container hanging — this calculation would have demanded about $0.40r$, and the discrepancy with the built drawing would be unmistakable.

16.6.7 Circular girder B₃ — bending, shear and torsion**STEP 16.27 Total load carried**

$$W = W_{DL,c} + W_T + W_{\text{live}} = 2100.38 + 4414.50 + 155.75 = 6670.63 \text{ kN}$$

STEP 16.28 Actions

With $N = 8$, $\alpha = \pi/8 = 22.5^\circ$, from Table 10.1: $C_1 = 0.00827$, $C_2 = 0.00417$, $C_3 = 0.000628$, $\varphi_m = 9.53^\circ$. And $R_c = r = 3.7944 \text{ m}$ — the girder is at the cone bottom, not at the tank radius.

$$M_{u,\text{support}} = 1.5(0.00827)(6670.63)(3.7944) = 313.98 \text{ kN m (hogging)}$$

$$M_{u,\text{span}} = 1.5(0.00417)(6670.63)(3.7944) = 158.32 \text{ kN m (sagging)}$$

$$T_u = 1.5(0.000628)(6670.63)(3.7944) = 23.84 \text{ kN m}$$

$$V_u = 1.5(6670.63)/(2 \times 8) = 625.37 \text{ kN}$$

Note

Had the girder been at the tank radius, $R_c = 5.749$ and every one of these would be 51 percent larger — $M_{u,\text{support}} = 476 \text{ kN m}$. The compactness of the Intze staging is paid for nowhere and earned here.

STEP 16.29 Equivalent moment and flexural steel

$$M_t = T_u \frac{1 + D/b}{1.7} = 23.84 \frac{1 + 720/550}{1.7} = 32.39 \text{ kN m}$$

$$M_{e1} = 313.98 + 32.39 = 346.36 \text{ kN m}$$

$d = 720 - 45 - 8 - 12 = 655 \text{ mm}$; $M_{u,\text{lim}} = 0.133f_{ck}bd^2 = 941.5 \text{ kN m} > 346.4 \checkmark$ (the coefficient is 0.133 for Fe 500; it is 0.138 for Fe 415 and 0.148 for Fe 250)

$$A_{st,\text{top}} = 1294 \text{ mm}^2 \rightarrow 12 \text{ nos } 12 \text{ mm}; \quad A_{st,\text{bot}} = 692 \text{ mm}^2 \rightarrow 9 \text{ nos } 10 \text{ mm}$$

plus the 8 nos 16 mm of hoop steel distributed around the perimeter.

STEP 16.30 Shear and torsion

$$V_e = 625.37 + 1.6 \frac{23.84}{0.550} = 694.7 \text{ kN}, \quad \tau_{ve} = \frac{694.7 \times 10^3}{550 \times 655} = 1.93 \text{ N/mm}^2$$

against $\tau_{c,\text{max}} = 3.5$ for M30 $\checkmark$. Two-legged closed stirrups, 12 mm at 125 c/c near the supports, with 135° hooks; side-face steel 0.1 percent each face since the depth exceeds 750 mm is not required at 720 mm, but is provided anyway at 10 mm at 250 c/c.

16.7 Step 6 — Load combinations

$R_c = 3.7944 \text{ m}$, $n = 8$, $\sum y^2 = \frac{n}{2} R_c^2 = 57.59 \text{ m}^2$, so $\Delta P = 2M/(nR_c) = M/15.18$.

Service combination	P , kN	V , kN	M , kN m	Windward, kN	Leeward, kN
S1 DL + water, no lateral	8538	0	0	1067	1067
S2 DL + water + wind	8538	270	3807	816	1318
S3 DL + water + seismic (full)	8538	564	12 178	265	1870
S4 DL + seismic (tank empty)	4124	333	6508	87	944
S5 DL + wind (tank empty)	4124	270	3807	265	766

Ultimate combination	P_u , kN	V_u , kN	M_u , kN m	Column P_u	Column M_u
U1 1.5(DL + water)	12 807	0	0	1601	0
U2 1.2(DL + water + EQ)	10 246	676	14 613	2244	141
U3 1.5(DL + water) + 1.5 EQ	12 807	845	18 266	2804	176
U4 1.5(DL + water) + 1.5 wind	12 807	404	5710	1977	84
U5 0.9 DL + 1.5 EQ (empty)	3711	499	9763	1107	104
U6 0.9 DL + 1.5 wind (empty)	3711	404	5710	840	84

No combination produces net tension in a column: the minimum leeward–windward value is 87 kN, compressive, under S4. ✓

16.8 Step 7 — Staging

STEP 16.31 Column

550 mm square, $l_e = 1.2 \times 3.333 = 4.00 \text{ m}$, $l_e/D = 7.27 < 12$, so short — no additional moment. Governing case U3: $P_u = 2804 \text{ kN}$, $M_u = 176 \text{ kN m}$.

By strain-compatibility interaction for M30 / Fe 500 with 8 bars equally on four faces and $d'/D = 0.10$:

$$p = 0.80 \text{ percent} \Rightarrow A_{st} = 2420 \text{ mm}^2 \rightarrow 8 \text{ nos } 20 \text{ mm} = 2513 \text{ mm}^2$$

$$M_{\text{cap}} = 355 \text{ kN m at } P_u = 2804 \text{ kN} \geq 176 \checkmark, \quad P_{uz} = 4959 \text{ kN}$$

Ties: 8 mm at 300 c/c, closed, reduced to 150 c/c over a depth of 550 mm each side of every bracing joint.

STEP 16.32 Bracings

400 × 700 mm, 5 levels, chord length $2r \sin(\pi/8) = 2.904 \text{ m}$. Column shear at a joint = 105.66 kN; joint moment = 352.21 kN m, shared between the two bracings meeting there at $\pi/8$ each side of the tangent:

$$M = 190.62 \text{ kN m}, \quad d = 637 \text{ mm}, \quad A_{st} = 722 \text{ mm}^2 \rightarrow 10 \text{ nos } 10 \text{ mm}$$

$$V_u = \frac{2M}{L} = 131.27 \text{ kN}, \quad \tau_v = 0.52 \text{ N/mm}^2$$

Nominal shear steel governs: 8 mm two-legged at 200 c/c.

16.9 Step 8 — Foundation

Annular raft, 2.2 m projection each side of the 3.794 m column circle:

$$R_o = 5.994 \text{ m}, \quad R_i = 1.594 \text{ m}, \quad A = 104.90 \text{ m}^2, \quad I = 1009.0 \text{ m}^4, \quad Z = 168.3 \text{ m}^3$$

Raft self weight = $104.90(0.700)(25) = 1835.7 \text{ kN}$; soil over the raft = 3398.7 kN; overburden relief at 2.5 m depth = $18(2.5) = 45 \text{ kN/m}^2$.

Case	P , kN	M , kN m	$p_{\max}$	$p_{\min}$	Allowed
S1	13 772	0	86.3	86.3	150 ✓
S2	13 772	3807	108.9	63.7	187.5 ✓
S3	13 772	12 178	158.6	13.9	187.5 ✓
S4	9358	6508	82.9	5.5	187.5 ✓
S5	9358	3807	66.8	21.6	187.5 ✓

All pressures are net of the 45 kN/m² overburden relief, and the 25 percent increase of IS 1904 is applied to the transient cases. No case produces uplift: the minimum is 5.5 kN/m², still bearing.

The column circle drives the raft

The raft here is much smaller than it would be if the columns stood at the tank diameter — outer radius 5.99 m against something over 8 m. That is the compactness of the Intze arrangement again. The price is that the pressures are higher for the same load, so the projection has to be generous: at the 1.5 m projection often used as a default, $p_{\max}$ under S3 comes to 232 kN/m² and the raft fails. Size the projection from the seismic case, not from the gravity case.

STEP 16.33 Raft reinforcement

Cantilever of 2.2 m under $p_{\max} = 158.6$ kN/m²:

$$M_u = 1.5 \frac{158.6(2.2)^2}{2} = 575.9 \text{ kN m/m}, \quad d = 700 - 45 - 12 = 643 \text{ mm}$$

$$A_{st} = 2183 \text{ mm}^2/\text{m} \rightarrow 16 \text{ mm at } 90 \text{ c/c radial, top and bottom}$$

Circumferential steel: 0.12 percent of gross, 840 mm²/m, 12 mm at 130 c/c both faces.

16.10 Step 9 — Summary

Member	Size	Reinforcement
Top dome	100 mm	8 mm @ 205 both ways
Ring beam B ₁	320 × 210	9 nos 10 mm, 8 mm links @ 200
Cylindrical wall	150 mm	10 mm @ 130 hoop both faces; 10 mm @ 300 vertical both faces
Ring beam B ₂	620 × 370	10 nos 16 mm hoop, staggered laps; 8 mm links @ 200
Conical slab	200 mm	10 mm @ 80 hoop both faces; 10 mm @ 300 meridional both faces
Bottom dome	200 mm	10 mm @ 300 both ways, both faces
Circular girder B ₃	550 × 720	12 nos 12 mm top, 9 nos 10 mm bottom, 11 nos 12 mm hoop distributed; 12 mm closed stirrups @ 125
Columns, 8 nos	550 sq	8 nos 20 mm, 8 mm ties @ 300 / 150 at joints
Bracings, 5 levels	400 × 700	10 nos 10 mm, 8 mm stirrups @ 200
Annular raft	R_i 1.594 to R_o 5.994, 700 mm	16 mm @ 90 radial top and bottom; 12 mm @ 130 circumferential

Three-way verification

Every number above has been produced three times independently: by hand from the formulae of Chapters 3 to 15; by the *Intze Tank Designer PRO* engine; and by a separate reference imple-

mentation written from the equations alone. All three agree to within rounding. The load path has additionally been checked against RDSO B-1235 — see Appendix E.

Detailing and Construction

The analysis says how much steel. This chapter says where it goes, and it matters more than the analysis does, because a tank that leaks almost never leaks because the steel area was wrong. It leaks at a junction, a joint, or a lap.

17.1 Cover, bar sizes and spacing

Item	Rule	Source
Clear cover, liquid face	45 mm, not more	IS 3370 (Part 1)
Clear cover, other faces	45 mm, severe exposure	IS 456 Table 16
Cover to a raft on blinding	50 mm	IS 456 Cl. 26.4.2.2
Minimum bar spacing	the larger of the bar diameter and 25 mm, plus 5 mm over the maximum aggregate size	IS 456 Cl. 26.3.2
Maximum bar spacing in a shell	the lesser of $3t$ and 300 mm	IS 3370
Minimum bar diameter in a shell	8 mm	practice
Minimum bar diameter in a column	12 mm	IS 456 Cl. 26.5.3.1

More cover is not safer

Crack width at the surface increases with cover, roughly in proportion. IS 3370 specifies 45 mm because that is the balance between protecting the steel and controlling the crack. Site staff who “give it a bit extra to be safe” are making the tank more likely to leak, not less. Use spacer blocks of the correct depth and check them.

17.2 Laps and anchorage

Development length, IS 456 Cl. 26.2.1

$$L_d = \frac{\phi \sigma_s}{4 \tau_{bd}} \quad (17.1)$$

with τ_{bd} from IS 456 Table, increased by 60 percent for deformed bars and a further 25 percent for bars in compression.

For M30 concrete, plain $\tau_{bd} = 1.5 \text{ N/mm}^2$, so for deformed bars $\tau_{bd} = 2.4 \text{ N/mm}^2$. With $\sigma_s = 0.87 f_y = 435 \text{ N/mm}^2$ for Fe 500:

$$L_d = \frac{435\phi}{4(2.4)} = 45.3\phi \quad \text{say } 47\phi \text{ in tension, } 38\phi \text{ in compression}$$

Bar, mm	8	10	12	16	20	25
L_d tension, mm (M30, Fe 500)	380	470	570	755	945	1180
Tension lap, $1.3L_d$, mm	495	615	740	980	1230	1535

Rules that matter on site:

- Laps in the **hoop steel** of a tank must be staggered, with not more than half the bars lapped at any one section, and the laps distributed around the circumference — never all on the same radius. A ring of laps all at one place is a plane of weakness right where the tension is greatest.
- Laps in the **conical slab's hoop steel** must be full tension laps, staggered so that no more than half the bars are lapped in any one plane — those rings are the tension member in the cone. Laps in the meridional steel matter less, the meridian being in compression, but they should still be staggered and kept away from the two springings where the local bending is.
- Column laps go in the **middle half** of the panel height, never at a beam joint, and not more than half the bars at a section.
- Every bar that ends must develop its force: a hook, a bend, or a lap into the adjoining member. “Stopping the bar at the face” is not anchorage.

17.3 The four critical junctions

17.3.1 Top dome springing and ring beam B_1

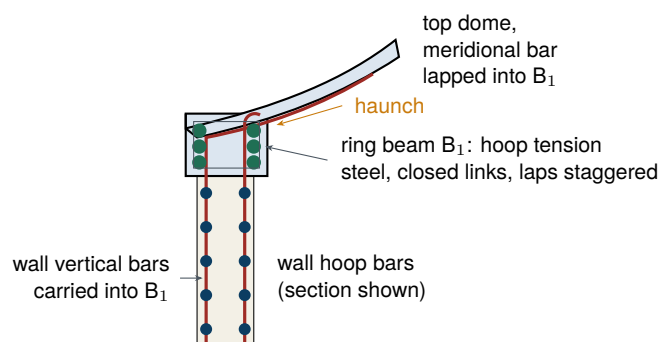


Figure 17.1: Top dome springing at ring beam B_1 . The dome's meridional bars turn down into the beam and develop full anchorage; the wall's vertical bars continue up through it.

Key points: the dome bar must be anchored for the meridional thrust, which is compressive but reverses under wind uplift on the roof; the beam's hoop steel is in tension and its laps are staggered; and a small haunch is formed at the inside corner to avoid a re-entrant crack.

17.3.2 Wall, ring beam B_2 and the top of the cone

Points to insist on:

- B_2 is in **hoop tension**, and usually the largest in the tank. Its ring bars are tension bars: full laps of at least L_d , staggered so that no more than half the bars are lapped in any one plane, with steel near both faces.
- The **conical slab's meridional bars** are in compression under the membrane analysis, but the junction is monolithic and a local bending moment develops there. Anchor them properly into the ring — a bar turned in by 150 mm is not anchored.
- The **wall's vertical bars** pass down through the ring and lap with the cone's meridional bars, so the load path across the junction is continuous in the steel as well as the concrete.
- A **haunch** on the inside of the wall-cone corner, 150 to 300 mm, to relieve the re-entrant stress concentration and to help the concrete flow.
- No construction joint at this level if it can be avoided. If one is unavoidable it goes in the wall a little *above* the ring, never at the junction itself, and it gets a water bar.

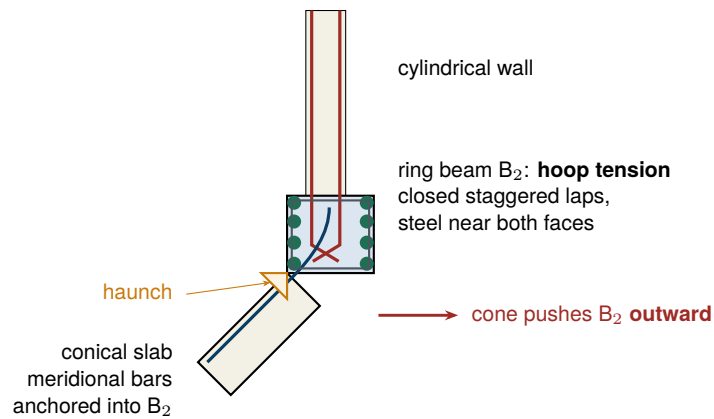


Figure 17.2: Wall, ring beam B_2 and the top of the conical slab. There is no column here — B_2 is a pure hoop member. The cone, being in compression, pushes the ring outward, and the water pressure adds to it, so the ring is in tension and its laps must be full and staggered.

17.3.3 Cone, circular girder B_3 , bottom dome and column head — the busiest joint in the tank

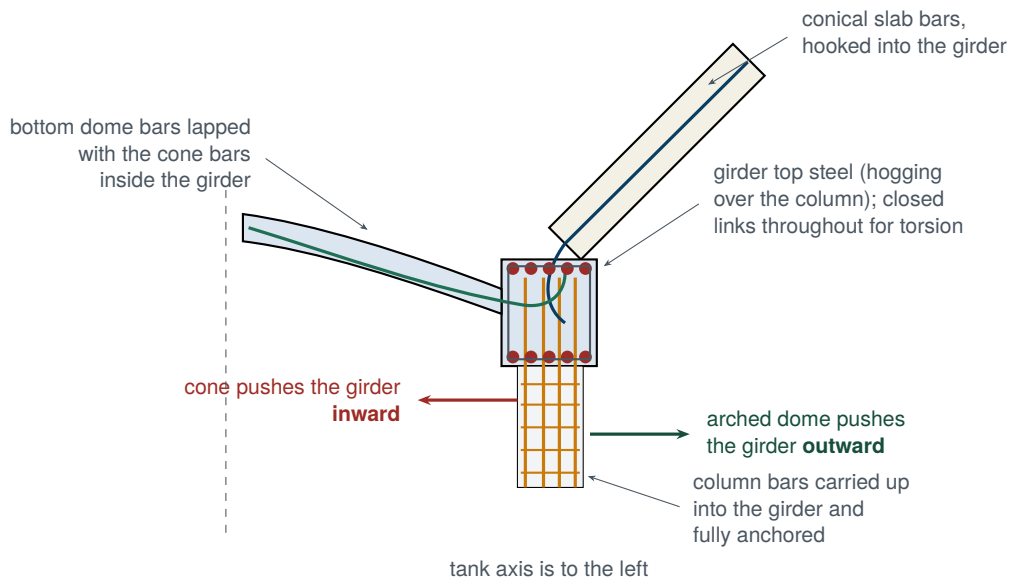


Figure 17.3: The balance junction, which is also the column head. The conical slab arrives from above and outboard in compression, pushing inward; the bottom dome springs upward and inboard, pushing outward; the column delivers its reaction from below. Four members meet and every bar must be anchored.

This is where tanks are lost. Points to insist on:

- The two shells are both in meridional *compression* and they push in nearly opposite directions. The bars still have to be continuous: carry them through the girder and lap them with each other inside it, so that the local bending at both springings — which the membrane theory does not see — is carried across.
- The **column longitudinal bars** continue up through the girder and are anchored in it. They are the reason the girder's links have to be planned around a clear column cage.
- The **girder's own top steel** is over the columns and its bottom steel at mid-span between them — the reverse of a simply supported beam, and a frequent site error.
- **Closed links throughout the girder** for torsion, with 135° hooks, and side-face steel of 0.1 percent

of the web area on each face where the depth exceeds 750 mm.

- The **residual hoop steel** from the Intze balance goes in as extra longitudinal bars distributed around the perimeter of the section, not concentrated top and bottom.
- Haunches on both inside corners — cone-to-girder and dome-to-girder.
- **One continuous pour** for the bottom dome, the girder and the cone if the volume allows it. This joint should not contain a construction joint.

17.3.4 Column and bracing joint

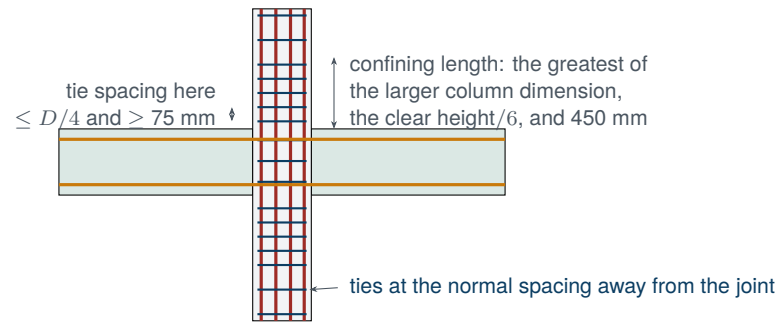


Figure 17.4: Column–bracing joint with IS 13920 confining reinforcement. The bracing’s top and bottom bars run continuously through the column.

IS 13920 requirements that apply in Zones III, IV and V:

- Special confining reinforcement over a length, at each end of the column, of the greatest of the larger member dimension, one sixth of the clear height, and 450 mm.
- Spacing of the confining ties not more than a quarter of the minimum member dimension and not less than 75 mm.
- All ties with 135° hooks and a ten-diameter extension, not less than 75 mm. A 90° hook opens out under cyclic load and is worthless.
- Ties continued through the joint itself.
- Bracing bars continuous through the column, and where they must be lapped, lapped away from the joint with confining ties over the lap length.

17.4 Construction joints and water bars

Where	Treatment
Horizontal joint in the wall	Kicker of 75 to 100 mm cast with the lift below; laitance removed by green cutting or by hacking and washing to expose the aggregate; surface saturated but surface-dry; a 150 to 230 mm PVC or rubber centre-bulb water bar cast in; no joint within 300 mm of a change of section.
Wall to B ₂	Avoid. If unavoidable, place it in the wall 300 mm above the ring beam, never at the junction.
Cone to ring beam	Avoid entirely; cast the cone and B ₃ monolithically.
Vertical joints	Avoid in a circular tank. A tank of this diameter can be cast in one continuous circumferential pour.
Raft to column	A kicker, and the column starter bars set accurately with a template before the raft concrete sets.

Water bars fail when they are installed badly, not when they are the wrong type

The three common failures are: the bar is displaced during the pour so the bulb sits against the formwork; the concrete is not compacted under the lower flange, leaving a void that is a direct leak path; and joints in the water bar itself are lapped instead of being heat-welded. All three are inspection items, not design items.

17.5 Construction sequence

1. Foundation: excavate, blind, set out the column ring accurately from a permanent benchmark, cast the raft with the column starter bars held by a template.
2. Staging: cast columns and bracings panel by panel, the whole ring together at each level so that the frame is always stable. Do not cast all the columns to full height and add the bracings afterwards.
3. Circular girder B_3 on a scaffold spanning between the column heads.
4. Bottom dome, conical slab and ring beam B_2 — *one continuous pour* if at all possible. This is the most difficult concreting operation in the job and it is worth planning in detail: sufficient mixers, a standby, a pour sequence that works round the ring, and a decision in advance about top shuttering on the steeper part of the cone.
5. Cylindrical wall, in one lift if the height allows.
6. Ring beam B_1 and top dome.
7. Plaster, waterproofing, pipework, ladder, railing.
8. Curing, then the water-tightness test, then commissioning.

Curing is a structural requirement here

IS 3370 requires a minimum of 14 days of continuous wet curing for liquid-retaining members. Shrinkage cracking is the leading cause of leakage in new tanks, and it is entirely a curing problem. Ponding the dome, wrapping the wall in hessian kept permanently wet, and starting as soon as the surface will take it without damage — these are not optional finishing items.

17.6 Water-tightness test

IS 3370 (Part 1) sets out the test and it should be written into the contract.

1. Complete curing. Do not plaster or apply any waterproof coating before the test — the point is to test the structure, not the coating.
2. Fill the tank slowly, at not more than about 2 metres of depth per day, to the full top water level.
3. Let it stand for **seven days** to allow absorption to stabilise.
4. Record the level accurately and leave it a further **seven days**.
5. The drop in level, after allowing for evaporation measured on an open pan alongside, must not exceed **1/500 of the average water depth**, or 10 mm, whichever is less.
6. Inspect the whole external surface. No damp patch or visible leakage is acceptable anywhere, whatever the level drop says.

If it fails

Find the cause before choosing the remedy. A general sweat over a large area is a concrete quality or curing problem, and is treated by a crystalline or cementitious coating on the inner

face. A discrete leak at a joint or a crack is treated by cutting out, grouting and re-sealing at that point. An external coating over a live crack simply moves the leak. Re-test after any repair.

17.7 Maintenance, briefly

An elevated tank needs an annual inspection: the external surface for cracks, damp patches and spalling; the internal surface after emptying and cleaning; the condition of the ladder, railing and manhole cover; the pipework and valves; the staging columns and bracings, particularly at the joints; and the foundation for settlement or tilting. Levels on four points of the girder, taken at each inspection and recorded, will detect differential settlement long before anything else does.

Cleaning and disinfection of a potable water reservoir should follow the local health authority's requirement, typically twice a year.

Notation

Geometry

D, R	internal diameter and radius of the cylindrical portion
H_1	height of the cylindrical wall
f	freeboard above the top water level
h_w	depth of water standing in the cylinder, $= H_1 - f$
r	radius of the bottom ring beam B_3
h_2	vertical height of the conical slab
L_s	slant length of the conical slab
θ_c	slope of the conical slab measured from the horizontal
α	generator angle of the cone measured from the vertical, $= 90^\circ - \theta_c$
h_t, h_b	rise of the top and bottom domes
$R_{d,t}, R_{d,b}$	radius of the sphere of the top and bottom domes
ϕ_0	springing angle of a dome, $\sin \phi_0 = r_0/R_d$
t	thickness of a shell; t_{td}, t_w, t_c, t_{bd} for the four shells
h_s	staging height, top of foundation to the circular girder
B_1	top ring beam, at the springing of the top dome, radius R
B_2	ring beam at the wall–cone junction, radius R ; pure hoop tension
B_3	circular girder at the bottom of the conical slab, radius r ; carried on the columns
R_c	radius of the column circle, equal to r in the standard arrangement
R_c	radius of the column circle
n	number of staging columns
n_{br}, N_p, h_p	number of bracing levels, number of panels, panel height
b_c	column side
L_{ch}	chord between adjacent columns
D_f	founding depth below ground level
R_o, R_i	outer and inner radius of the annular raft

Loads and forces

$\gamma_c, \gamma_w, \gamma_{soil}$	unit weight of concrete, water and soil
W_D, W_T	total dead load and total water load
V	volume of water
N_ϕ, N_θ	meridional and hoop membrane force per unit length, tension positive
p_n	pressure normal to a shell surface, outward positive
q	self weight of a shell per unit area of its surface
H	radial line load on a ring beam, outward positive
P	hoop force in a ring beam, or axial load on a column
F	hoop tension per unit height in the cylindrical wall
W	total load on the circular girder, or the load transferred across a cut
M, T, V	bending moment, torsion and shear

Materials and design

f_{ck}, f_y	characteristic strength of concrete and steel
E_c, E_s	modulus of elasticity of concrete and steel
m	modular ratio, $280/3\sigma_{cbc}$
σ_{ct}	permissible direct tensile stress in concrete
$\sigma_{cbc}, \sigma_{cc}$	permissible compressive stress in bending and direct
σ_{st}	permissible tensile stress in steel
A_{st}, A_{sc}	area of tension and compression reinforcement
A_c	gross area of a concrete section
ρ_{min}	minimum reinforcement percentage
z	surface zone thickness for the minimum reinforcement rule
L_d	development length
x_u	neutral axis depth
P_u, M_u	factored axial load and moment
P_{uz}	pure axial capacity of a column section
l_e	effective length of a column

Wind

V_b, V_z	basic and design wind speed
k_1 to k_4	risk, terrain-height, topography and cyclonic-importance factors
p_z, p_d	wind pressure and design wind pressure
K_d, K_a, K_c	directionality, area-averaging and combination factors
C_f	force coefficient
A_e, A_s	effective projected area; projected area of one staging frame
φ	solidity ratio
η	shielding factor for the leeward staging frame

Seismic

m	total mass of the contained liquid
m_i, m_c	impulsive and convective mass
m_s	structural mass, container plus one third of the staging
h	depth of the equivalent circular cylinder
h_i, h_c	height of the impulsive and convective mass above the liquid base, for the moment at the tank base
h_i^*, h_c^*	the same, including the pressure on the tank base, for the overturning moment
h_{cg}	height of the centre of gravity of the empty container
K_s	lateral stiffness of the staging
β	bracing flexibility factor applied to K_s
T_i, T_c	impulsive and convective time period
Z, I, R	zone factor, importance factor, response reduction factor
S_a/g	spectral acceleration coefficient
A_h	design horizontal seismic coefficient
V_i, V_c	impulsive and convective base shear
M_i, M_c, M^*	impulsive, convective and total overturning moment
d_{max}	maximum sloshing wave height
Q_{iw}, Q_{cw}	hydrodynamic wall pressure coefficients

Design Tables

B.1 Concrete properties

Grade	σ_{ct} N/mm ²	σ_{cbc} N/mm ²	σ_{cc} N/mm ²	m	$E_c = 5000\sqrt{f_{ck}}$ N/mm ²	τ_{bd} deformed N/mm ²
M25	1.3	8.5	6.0	10.98	25 000	2.24
M30	1.5	10.0	8.0	9.33	27 386	2.40
M35	1.6	11.5	9.0	8.12	29 580	2.72
M40	1.7	13.0	10.0	7.18	31 623	3.04
M45	2.0	14.5	11.0	6.44	33 541	3.20
M50	2.1	16.0	12.0	5.83	35 355	3.36

σ_{ct} from IS 3370 (Part 2) Table 1; σ_{cbc} and σ_{cc} from IS 456 Annex B Table 21; τ_{bd} from IS 456 Cl. 26.2.1.1 with the 60 percent increase for deformed bars. The IS 456 bond table stops at M40; the M45 and M50 values here are extrapolated and should be checked before use.

B.2 The thickness coefficient

The bracket in equation (2.3), $(1/\sigma_{ct} - (m - 1)/\sigma_{st})$, evaluated for the usual combinations. Multiply by the membrane tension in kN/m to get the required thickness in mm.

Grade	$\sigma_{st} = 150$		$\sigma_{st} = 130$	
	$m - 1$	coefficient	$m - 1$	coefficient
M25	9.98	0.7027	9.98	0.6925
M30	8.33	0.6111	8.33	0.6026
M35	7.12	0.5775	7.12	0.5702
M40	6.18	0.5470	6.18	0.5406

B.3 Minimum reinforcement, IS 3370 (Part 2) Cl. 8.1

Area in mm² per metre *per face*, each way, for HYSD bars.

Thickness mm	Surface zone mm	Faces	$\rho = 0.24\%$ (all dims ≤ 15 m)	$\rho = 0.35\%$ (otherwise)
100	100	1	240	350
150	150	1	360	525
200	100	2	240	350
250	125	2	300	438
300	150	2	360	525
400	200	2	480	700
500	250	2	600	875
700	250	2	600	875

Note the discontinuity at 200 mm: a 150 mm section is one zone of 150 mm with all the steel in one

face, while a 200 mm section is two zones of 100 mm with steel in both faces — so the *total* steel jumps from 360 to 480 mm²/m at that thickness.

B.4 Bar areas

Diameter, mm	8	10	12	16	20	25	32
Area, mm ²	50.3	78.5	113.1	201.1	314.2	490.9	804.2
Mass, kg/m	0.395	0.617	0.888	1.579	2.466	3.854	6.313
<i>Area in mm² per metre width at the spacing shown</i>							
75 mm	670	1047	1508	2681	4189	6545	—
100 mm	503	785	1131	2011	3142	4909	8042
125 mm	402	628	905	1608	2513	3927	6434
150 mm	335	523	754	1340	2094	3272	5362
200 mm	251	393	565	1005	1571	2454	4021
250 mm	201	314	452	804	1257	1963	3217
300 mm	168	262	377	670	1047	1636	2681

B.5 Development length and lap, M30 with Fe 500

Diameter, mm	8	10	12	16	20	25
L_d tension (47ϕ), mm	380	470	570	755	945	1180
L_d compression (38ϕ), mm	305	380	455	610	760	950
Tension lap ($1.3L_d$), mm	495	615	740	980	1230	1535

B.6 Circular girder coefficients

Reproduced from Table 10.1 for convenience. $M_{\text{support}} = C_1 W R_c$, $M_{\text{span}} = C_2 W R_c$, $T_{\text{max}} = C_3 W R_c$.

N	C_1	C_2	C_3	φ_m , degrees
4	0.03415	0.01762	0.005272	19.20
6	0.01482	0.00751	0.001507	12.73
8	0.00827	0.00417	0.000628	9.53
10	0.00527	0.00265	0.000320	7.62
12	0.00365	0.00183	0.000184	6.35
16	0.00205	0.00103	0.000078	4.76

B.7 Seismic spring–mass parameters

For a circular tank, as a function of h/D . See Chapter 7 for the expressions.

h/D	m_i/m	m_c/m	h_i/h	h_c/h	h_i^*/h	h_c^*/h
0.2	0.231	0.721	0.375	0.521	2.041	2.228
0.3	0.344	0.615	0.375	0.545	1.327	1.227
0.4	0.450	0.517	0.375	0.574	0.986	0.907
0.5	0.542	0.437	0.375	0.605	0.797	0.784
0.75	0.710	0.304	0.375	0.681	0.580	0.727
1.0	0.808	0.230	0.406	0.742	0.494	0.755
1.5	0.902	0.153	0.438	0.820	0.450	0.822

B.8 Seismic zone factors and basic wind speeds for selected Indian cities

City	Seismic zone	Z	V_b , m/s
Kolkata	III	0.16	50
Siliguri	IV	0.24	47
Delhi	IV	0.24	47
Mumbai	III	0.16	44
Chennai	III	0.16	50
Bengaluru	II	0.10	33
Hyderabad	II	0.10	44
Ahmedabad	III	0.16	39
Bhuj	V	0.36	50
Guwahati	V	0.36	50
Patna	IV	0.24	47
Lucknow	III	0.16	47

Where this goes wrong

This table is a convenience for checking an order of magnitude. Always read the zone and the basic wind speed for the actual site from the maps in IS 1893 (Part 1) and IS 875 (Part 3), not from a city list — zone boundaries pass through districts and the difference between Zone III and Zone IV is a 50 percent increase in the seismic force.

Index of Code References

Every code reference used in this manual, with the chapter in which it appears. Clause numbers are quoted against the editions listed on the title page; verify them against your own copy before relying on them, and note that all the Indian Standards named here are periodically amended.

C.1 IS 456 : 2000 — Plain and Reinforced Concrete

Clause or table	Subject	Chapter
Cl. 6.2.3.1	$E_c = 5000\sqrt{f_{ck}}$	7
Table 5	Minimum cement content and maximum w/c for exposure	2
Table 16	Nominal cover for durability	17
Table 18	Load combinations and partial safety factors	13
Cl. 26.2.1	Development length	17
Cl. 26.2.1.1	Design bond stress τ_{bd}	17, App. B
Cl. 26.3.2	Minimum distance between bars	17
Cl. 26.4.2.2	Cover to a member cast against earth	17
Cl. 26.5.1.1	Minimum and maximum flexural steel	2
Cl. 26.5.1.3	Side-face reinforcement in deep beams	10
Cl. 26.5.3.1	Minimum and maximum column steel; bar size	14
Cl. 26.5.3.2	Lateral ties in columns	14
Cl. 38.1	Stress block for concrete in flexure	14
Cl. 39	Members subject to combined axial load and bending	14
Cl. 39.6	P_{uz}	14
Cl. 39.7.1	Additional moment in a slender column	14
Cl. 41.3.1	Equivalent shear for combined shear and torsion	10
Cl. 41.4.2	Equivalent bending moment for torsion	10
Cl. 41.4.3	Transverse reinforcement for torsion	10
Annex B, Table 21	Permissible stresses in concrete, working stress	2, App. B
Fig. 23A	Design stress–strain curve for cold-worked bars	14

C.2 IS 3370 — Concrete Structures for Storage of Liquids

Reference	Subject	Chapter
Part 1 : 2009	Minimum grade M30; cover 45 mm on the liquid face	2, 17
Part 1 : 2009	Water-tightness test procedure and acceptance	17
Part 2 : 2009, Table 1	Permissible concrete stresses for resistance to cracking	2, App. B
Part 2 : 2009, Table 4	Permissible stresses in steel reinforcement	2
Part 2 : 2009, Cl. 8.1	Minimum reinforcement, surface zone rule	2, 8–12
Part 2 : 2009	Modular ratio $m = 280/3\sigma_{cbc}$	2
Part 4 : 1967	Coefficients for moments and forces in cylindrical walls	9

C.3 IS 875 — Design Loads

Reference	Subject	Chapter
Part 1	Unit weights of building materials	5
Part 2	Imposed loads, including access loads on roofs	5
Part 3 : 2015, Cl. 6	Basic wind speed and the wind speed map	6
Part 3 : 2015, Cl. 6.3	Risk coefficient k_1	6
Part 3 : 2015, Table 2	Terrain and height factor k_2	6, App. B
Part 3 : 2015	Topography factor k_3	6
Part 3 : 2015	Importance factor for cyclonic regions k_4	6
Part 3 : 2015, Cl. 7.2	Design wind pressure; K_d , K_a , K_c	6
Part 3 : 2015, Table 4	Area averaging factor K_a	6
Part 3 : 2015	Force coefficients for circular sections	6
Part 3 : 2015	Force coefficients for single frames and shielding	6

C.4 IS 1893 — Criteria for Earthquake Resistant Design

Reference	Subject	Chapter
Part 1 : 2016, Table 3	Zone factor Z	7
Part 1 : 2016, Fig. 2	Design acceleration spectrum S_a/g	7
Part 1 : 2016	Multiplying factors for damping other than 5 percent	7
Part 2 : 2014, Cl. 4.2	Spring-mass model for seismic analysis	7
Part 2 : 2014, Cl. 4.2.1.1	Impulsive and convective mass, circular tanks	7
Part 2 : 2014, Cl. 4.2.2.3	Structural mass: container plus one third of the staging	5, 7
Part 2 : 2014, Cl. 4.2.3	Equivalent circular tank for Intze and other shapes	7
Part 2 : 2014, Cl. 4.3.1.3	Impulsive time period of an elevated tank; definition of K_s	7
Part 2 : 2014, Cl. 4.3.2	Convective time period	7
Part 2 : 2014, Cl. 4.4	Damping: 5 percent impulsive, 0.5 percent convective	7
Part 2 : 2014, Cl. 4.5	Design horizontal seismic coefficient	7
Part 2 : 2014, Cl. 4.5.4	Multiplying factor 1.75 for 0.5 percent damping	7
Part 2 : 2014, Cl. 4.6.2, 4.6.3	Base shear and its combination	7
Part 2 : 2014, Cl. 4.7.2, 4.7.3	Overturning moment using h_i^* and h_c^*	7
Part 2 : 2014, Cl. 4.9.1, 4.9.2	Hydrodynamic pressure on the wall	7
Part 2 : 2014, Cl. 4.10	Effect of vertical ground acceleration	7
Part 2 : 2014, Cl. 4.11	Sloshing wave height and freeboard	7
Part 2 : 2014, Table 1	Importance factor I	7
Part 2 : 2014, Table 2	Response reduction factor R	7

C.5 Other standards

Reference	Subject	Chapter
IS 11682 : 1985	Criteria for the design of RCC staging for overhead water tanks	2, 14
IS 13920 : 2016	Ductile detailing: confining reinforcement, 135° hooks	14, 17
IS 1904 : 1986	Design and construction of foundations: settlement limits	15
IS 6403 : 1981	Bearing capacity of shallow foundations	15
IS 1786	High strength deformed steel bars	2
IS 2502	Bending and fixing of bars for concrete reinforcement	17
SP 16 : 1980	Design aids: column interaction charts	14
SP 34 : 1987	Handbook on reinforcement and detailing	17

Validation Against RDSO Standard Drawing B-1235

A design method is only as good as its agreement with something that has been built. This appendix takes the procedure of Chapters 4 to 15, applies it to the capacity and proportions of RDSO standard drawing B-1235 — a 450 kilolitre Intze tank issued for use on Indian Railways — and compares the result with the drawing itself.

The comparison matters for a particular reason. As Chapter 1 explains, the whole design hinges on where the columns are, and that single choice reverses every horizontal thrust below the wall. If the arrangement adopted here were wrong, the disagreement would not be a matter of a few percent; it would be structural.

D.1 Reading the arrangement off the drawing

B-1235 does not state the column circle in words, but its load table gives it away twice.

Recovering the column circle from a load table

For n columns equally spaced on a circle of radius R_c , each column's load deviates from the mean by $\Delta P = 2M/(nR_c)$, so the leeward and windward columns differ by $2\Delta P$. Hence

$$\Delta P = \frac{P_{\text{lee}} - P_{\text{wind}}}{2} \implies R_c = \frac{2M}{n \Delta P} \quad (\text{D.1})$$

Applying it to the seismic row of the drawing's load table, with the tank full:

Leeward column load	157 340 kg = 1543.5 kN
Windward column load	81 340 kg = 797.9 kN
$\Delta P = (P_{\text{lee}} - P_{\text{wind}})/2$	372.8 kN
Overturning moment M	576 810 kg m = 5658.5 kN m
Columns n	8
$R_c = 2M/(n\Delta P)$	3.795 m, a circle of 7.59 m

The tank diameter on the same drawing is 11.48 m. The columns are therefore *not* at the tank diameter; they are at 0.661 of it, which is precisely the r/R of the conical slab. This is the arrangement adopted throughout this manual.

Check the arithmetic against a second row

Where a drawing tabulates both a wind and a seismic case, the same formula applied to each should return the same R_c . If two rows disagree by a clean factor — two, most often — the likely cause is that one row tabulates the *deviation from the mean* where the other tabulates the leeward and windward loads themselves, or that the two moments are quoted about different levels. Resolve it before trusting either; do not average them.

An independent confirmation, from the vertical load

This check uses no assumption about R_c at all. The drawing shows eight columns and a mean column load of 1170.7 kN with the tank full. Eight times that is 9366 kN. Subtracting the tabulated dead load of 4957 kN leaves 4409 kN — against $450 \times 9.81 = 4415$ kN of water. The difference is 0.14 percent, so the column count is eight and the whole water load is indeed arriving at the columns through the girder, exactly as the standing model requires.

D.2 Geometry, derived from capacity alone

The procedure of Chapter 4 was run with nothing taken from the drawing but the capacity (450 kL), the freeboard, and the four customary ratios. The dimensions fall out of the cubic; none of them was fitted.

Dimension	This manual	B-1235	Difference
Tank diameter D	11.498 m	11.480 m	+0.16%
Cone bottom radius r	3.794 m	3.800 m	−0.16%
Column circle $2r$	7.589 m	7.600 m	−0.15%
Cylindrical height H_1	3.679 m	3.670 m	+0.25%
Cone height h_2	1.955 m	1.915 m	+2.1%
Top dome rise h_t	1.897 m	1.883 m	+0.74%
Bottom dome rise h_b	0.949 m	0.930 m	+2.0%
Bottom dome rise / r	0.250	0.245	—

Every dimension is within about two percent, and the two that are not within one percent — the cone and bottom dome heights — are the two most sensitive to the cone-slope and rise ratios, which were taken as round numbers rather than fitted.

D.3 The decisive test: the balance

The geometry above would come out much the same under either load-path assumption, because it is driven by volume. The balance is the test that distinguishes them, and it is unambiguous.

	Standing model (this manual)	Hanging model (the common error)
Conical slab at the girder	170.3 kN/m inward	146 kN/m outward
Bottom dome at the girder	181.2 kN/m outward	108 kN/m inward
Do they oppose?	Yes	Yes, but reversed
Net residual	10.9 kN/m	38 kN/m
Balancing rise h_b/r , eq. (12.10)	0.264	0.40
B-1235 actually uses		0.245

The standing model predicts a balancing rise of $0.264r$ against the drawing's $0.245r$ — an eight percent difference, well inside the scatter one would expect from round-number ratios. The hanging model predicts $0.40r$, which is 63 percent high and would have produced a visibly different tank.

Why the hanging model cannot be rescued

It is tempting to suppose the two models differ only by a sign convention and that either would do if applied consistently. They would not. Under the hanging model the girder sits at the tank diameter, so $R_c = R = 5.749$ m rather than 3.794 m, and:

- every girder moment is 51 percent larger;
- the column circle is 11.5 m rather than 7.6 m, and the recovered R_c from the drawing's own load table contradicts it outright;
- the ring beam at the wall-cone junction comes out in hoop compression rather than the 272 kN of tension it actually carries, so it would be detailed with nominal steel;
- the bottom dome's segment is added to the capacity rather than subtracted, making the tank about 10 percent smaller than intended.

These are not conventions. They are different structures.

D.4 Forces, where the drawing gives them

Quantity	This manual	B-1235	Note
Number of columns	8	8	recovered from the load table
Total dead load	4124 kN	4957 kN	the drawing includes a larger staging
Water load	4415 kN	4409 kN	0.14%
Mean column load, tank full	1067 kN	1171 kN	follows the dead-load difference
Column size	550 sq	600 sq	the drawing's staging is heavier

The dead-load difference is expected and not a discrepancy: B-1235 is drawn for a specific staging height and bracing arrangement that differ from the worked example's 20 m with five bracing levels. What matters is that the water load — the quantity fixed purely by the geometry — agrees to one-seventh of one percent, and that the column count and circle recovered from the drawing's own table match the arrangement the method assumes.

D.5 Conclusion

The procedure in this manual reproduces an issued standard drawing from capacity and four ratios alone: the geometry to within two percent, the water load to within 0.15 percent, the column circle to within 0.15 percent, and the bottom-dome rise to within eight percent of the balancing value. The load path — columns at the cone bottom, cone and bottom dome both in compression, the two thrusts opposing at the girder — is confirmed by two independent readings of the drawing's own load table: the seismic row's column-load spread, and the total vertical load carried by the eight columns.

Do this on your own drawings

The three checks of this appendix take ten minutes and will tell you immediately whether a design you have been handed is arranged the way you think it is:

1. Recover R_c from the load table: $\Delta P = (P_{lee} - P_{wind})/2$, then $R_c = 2M/(n\Delta P)$. Compare with $D/2$ and with r .
2. Divide the total container-plus-water weight by $2\pi r$. That is roughly the meridional force arriving at the girder, and it must be compressive.
3. Check the bottom dome rise against $r \tan(\phi_0/2)$ from equation (12.10). If your answer is near $0.40r$ and the drawing says $0.25r$, your signs are reversed.

References and Further Reading

E.1 Indian Standards

1. IS 456 : 2000, *Plain and Reinforced Concrete — Code of Practice*, Bureau of Indian Standards, New Delhi.
2. IS 3370 (Part 1) : 2009, *Concrete Structures for Storage of Liquids — Code of Practice: General Requirements*.
3. IS 3370 (Part 2) : 2009, *Concrete Structures for Storage of Liquids: Reinforced Concrete Structures*.
4. IS 3370 (Part 4) : 1967, *Code of Practice for Concrete Structures for the Storage of Liquids: Design Tables*.
5. IS 875 (Part 1) : 1987, *Code of Practice for Design Loads: Dead Loads*.
6. IS 875 (Part 2) : 1987, *Code of Practice for Design Loads: Imposed Loads*.
7. IS 875 (Part 3) : 2015, *Code of Practice for Design Loads: Wind Loads*.
8. IS 1893 (Part 1) : 2016, *Criteria for Earthquake Resistant Design of Structures: General Provisions and Buildings*.
9. IS 1893 (Part 2) : 2014, *Criteria for Earthquake Resistant Design of Structures: Liquid Retaining Tanks*.
10. IS 11682 : 1985, *Criteria for Design of RCC Staging for Overhead Water Tanks*.
11. IS 13920 : 2016, *Ductile Design and Detailing of Reinforced Concrete Structures Subjected to Seismic Forces — Code of Practice*.
12. IS 1904 : 1986, *Code of Practice for Design and Construction of Foundations in Soils: General Requirements*.
13. IS 6403 : 1981, *Code of Practice for Determination of Breaking Capacity of Shallow Foundations*.
14. SP 16 : 1980, *Design Aids for Reinforced Concrete to IS 456*.
15. SP 34 : 1987, *Handbook on Concrete Reinforcement and Detailing*.

E.2 The commentary behind IS 1893 (Part 2)

The seismic provisions of IS 1893 (Part 2) : 2014 were developed from a project at IIT Kanpur, and the resulting guideline document is published openly. It contains the same expressions as the code, with a full commentary explaining where each came from and a set of worked examples — including an elevated tank. Anyone doing tank seismic design regularly should have it.

16. Jaiswal, O. R., Rai, D. C. and Jain, S. K., *IITK-GSDMA Guidelines for Seismic Design of Liquid Storage Tanks: Provisions with Commentary and Explanatory Examples*, National Information Centre of Earthquake Engineering, IIT Kanpur.
Available at <https://www.iitk.ac.in/nicee/IITK-GSDMA/EQ08.pdf>
17. Jaiswal, O. R., Rai, D. C. and Jain, S. K., “Review of Seismic Codes on Liquid-Containing Tanks”, *Earthquake Spectra*, Vol. 23, No. 1, 2007.

E.3 The original theory

18. Housner, G. W., “The Dynamic Behaviour of Water Tanks”, *Bulletin of the Seismological Society of America*, Vol. 53, No. 2, 1963, pp. 381–387. The two-mass idealisation in its original form.

19. Housner, G. W., “Dynamic Pressures on Accelerated Fluid Containers”, *Bulletin of the Seismological Society of America*, Vol. 47, No. 1, 1957.
20. Timoshenko, S. and Woinowsky-Krieger, S., *Theory of Plates and Shells*, 2nd edn, McGraw-Hill. The membrane theory of Chapter 3 and the edge-disturbance decay.
21. Flügge, W., *Stresses in Shells*, 2nd edn, Springer. A more complete treatment of shells of revolution.

E.4 Indian textbooks on tank design

22. Punmia, B. C., Jain, A. K. and Jain, A. K., *Reinforced Concrete Structures*, Vol. II, Laxmi Publications. The standard Indian treatment of the Intze tank with worked examples; note that older printings use the superseded IS 3370 : 1965 permissible stresses and minimum reinforcement rule.
23. Krishna Raju, N., *Advanced Reinforced Concrete Design*, CBS Publishers. Circular beam coefficients and tank design.
24. Ramamrutham, S., *Design of Reinforced Concrete Structures*, Dhanpat Rai. Membrane analysis of domes and conical shells.
25. Varghese, P. C., *Advanced Reinforced Concrete Design*, PHI Learning. Good on the limit state members and on detailing.
26. Sushil Kumar, *Treasure of RCC Designs*, Standard Book House.

E.5 On construction and water-tightness

27. Anchor, R. D., *Design of Liquid Retaining Concrete Structures*, 2nd edn, Edward Arnold. The clearest account in English of why liquid-retaining design is a crack-control problem, and the origin of the surface-zone rule that IS 3370 : 2009 adopted from BS 8007.
28. BS 8007 : 1987, *Design of Concrete Structures for Retaining Aqueous Liquids*. Withdrawn and replaced by Eurocode 2 Part 3, but it is the direct ancestor of the 2009 revision of IS 3370 and remains the best explanation of the crack-width approach.
29. CPHEEO, *Manual on Water Supply and Treatment*, Ministry of Urban Development, Government of India. Service reservoir capacity, staging height and the water supply context in which the tank sits.

E.6 Software used in this manual

30. *Intze Tank Designer PRO*, Nirmaanpro. The worked example of Chapter 16 was computed independently by hand and by this software, and the two agree.