3. Chapter 3: Material and methods

3.1. General

This Chapter deals with the detailed design methodology for HWTs (both Intze and Circular water type tanks), design of the ferrocement lining and the Software development for the HWT alongside the design and software development for CWT for comparative analysis. To begin with, understanding the detailed design procedures and software development for CWT is essential. The foundational design methodology and software development process for HWT are similar to those for CWT.

Chapter 1 introduced the fundamentals of water tank design. This chapter now focuses on the formulation and development of software programs for both Conventional and HWTs. For the computations, analysis, and design of water tank structures, four unique software programs were created for each type, as outlined below:

Table 3.1: Details of the Software program s Developed

Intze tanks	**Circular tanks**
• Conventional Intze tank Software	• Conventional Circular tank Software
• Hybrid Intze tank Software	• Hybrid Circular tank Software

This chapter is divided into four major Sections, each addressing both the design methodology and software development:

- **Section 3.2**: Design Process and Software Development for Conventional Intze water tanks.
- **Section 3.3**: Design Process, Concept, and Software Development for Hybrid Intze tanks.
- **Section 3.4**: Design Process and Software Development for Conventional Circular water tanks.
- **Section 3.5:** Design Process and Software Development for Hybrid Circular water tanks.

Details of the Modules for the design of water tanks are given in Table 3.2.

Table 3.2: Details of the Modules of the Software program s

Modules	CWTs	HWTs
Module I	Input and dimensioning of tanks	
Module II	Analysis of RCC tanks	
Module III	Design of members of tank	
Module IV	Staging configurations and Lateral analysis of tanks	
Module V	Staging design	
Module VI	Foundation design	
Module VII	Estimation and costing	Ferrocement lining design
Module VIII	-	Estimation and costing

Each Section will discuss the design technicities alongside the software development modules.

3.2. Design process and software development for conventional Intze water tanks

In the development process for CWT, the software program is tailored specifically to address the requirements and intricacies of this design type. The software is divided into 7 major sections called Modules. Flow chart of the methodology is given in Fig 3.1.

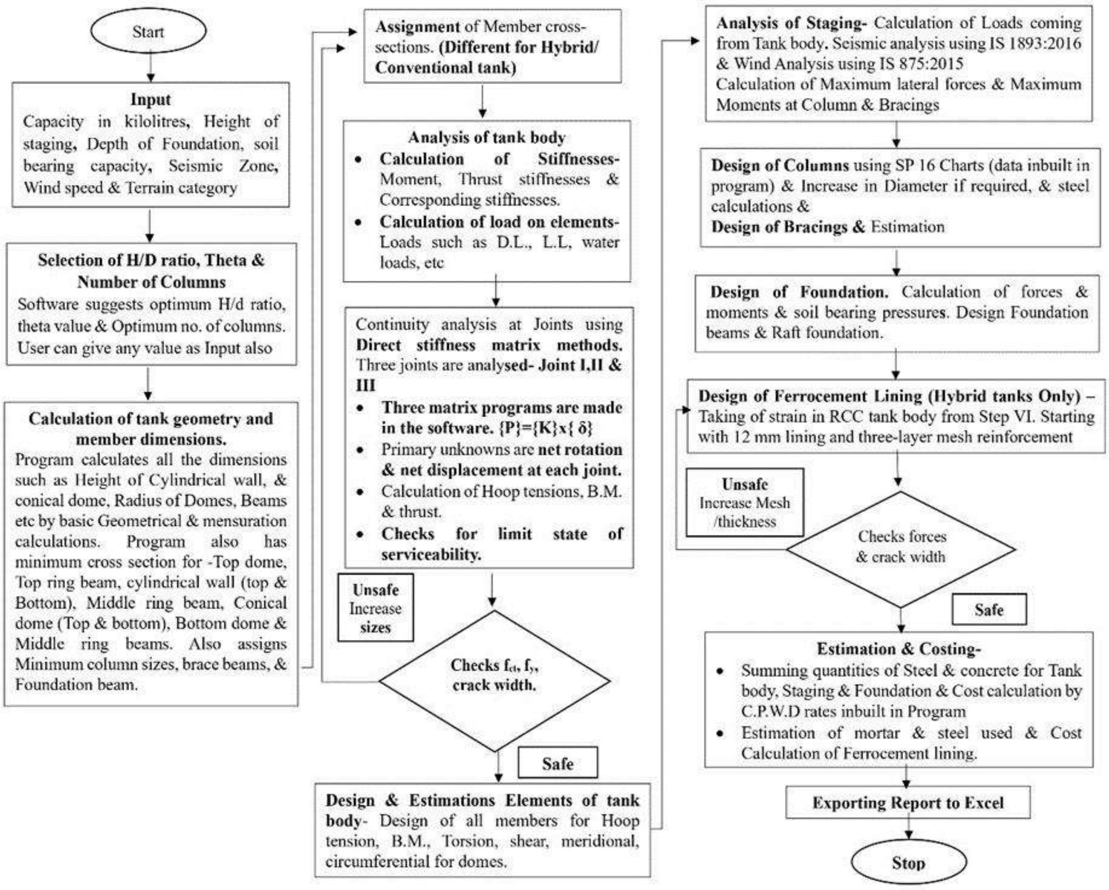

Fig. : 3.1:Flow chart of the software methodology for Intze water tanks

3.2.1. Module I: Input and dimensioning of tanks

The initial step involves determining the tank dimensions, for which the primary input is the desired capacity of the tank. This is essential as it drives the overall design process. Additional crucial inputs include the column height, which defines the vertical dimensions of the tank, and the bearing capacity of the soil, which assesses the soil's ability to support the tank. The depth of the foundation is also important to ensure stability. The seismic zone of the location, which affects the tank's design to withstand earthquakes, along with the wind velocity and terrain topography, are vital factors that influence the structural integrity and suitability of the tank.

Step I: Input required

Input required are provided in the Table 3.3 as given below-

Table 3.3: Input required for the design of Conventional Intze water tanks

Sr.No	Input required
1	Capacity of tanks in kL.
2.	Staging Height.
3.	Net Safe Bearing capacity of Soil and foundation depth (As per Soil Report).
4.	Seismic Zone.
5.	Wind Speed and Terrian type along with region type (Coastal /Non-Coastal)
6	H/D ratio (Software is inbuilt with Optimal values for 100-1000 kL capacity tanks, but user can input as per own)
7	Angle of Conical dome (Software is inbuilt with Optimal values for 100-1000 kL capacity tanks, but user can input as per own)
8	Optimal Number of columns (Software is inbuilt with Optimal values for 100-1000 kL capacity tanks, but user can input as per own)
9	The characteristic strength of Concrete for tank body elements (f_{cktank}) and for Staging and foundation ($f_{ckstaging}$) is kept as 30 N/mm^2. User can change these parameters.
10	The ultimate strength of Steel for tank body elements (f_{ytank}) and for Staging and foundation (f_y) is kept as 500 N/mm^2. User can change these parameters. For Conventional type I tank f_{ytank} is kept as 130 N/mm^2 and for Conventional type II tank f_{ytank} is kept as 435 N/mm^2 (0.87 times f_y)

Step II: Calculations of member dimensions for the desired capacity of the tanks

Capacity is the basis of data required to design the tank. It is required to calculate the dimensions of the tanks from basic mensuration formula. In this module, various mensuration formulas are applied to calculate the dimensions and other essential parameters of the tank. Here is a detailed description of the formulas used in Table 3.4. Loop will terminate when the required capacity is achieved. And the dimensions are finalized.

Once the specified capacity is reached, the iterative loop terminates, and the final dimensions are calculated based on the last iteration. These final dimensions incorporate the optimized parameters determined throughout the iterative process, ensuring that the tank design meets the desired capacity requirement.

Table 3.4: Calculation of the dimensions for Conventional Intze water tanks

Dimension	Formula or starting value	Eq. No
Radius of Cylindrical wall: r_1	1 m (At the start of loop with increment of 0.1m)	-
H/D ratio: c_1	User Input	-
Height of the Top dome h_1	$h_1 = (2 \times r_1 + 0.2)/7$	(3.1)
Radius of Bottom beam r_2	$r_2 = (0.75 \times r_1);$	(3.2)
Height of the Bottom dome h_2	$h_2 = (2 \times r_2 + 0.2)/5;$	(3.3)
Radius of the Top dome R_1	$R_1 = 0.5 \times \{(r_1 + 0.1)^2 + {h_1}^2\}/h_1;$	(3.4)
Radius of the Bottom dome R_2	$R_2 = 0.5 \times \{(r_2 + 0.1)^2 + {h_2}^2\}/h_2;$	(3.5)
Slant length of conical dome	$l = (r_1 - r_2) \times \tan A_r$	(3.6)
Volume Calculation= Cylinder Volume + Conical dome Volume- Bottom dome Volume	$\pi {r_1}^2 H + \pi l \frac{4{r_1}^2 + 4{r_2}^2 + 4r_1 r_2}{12.0} - \pi \times h_2^2 \frac{3R_2 - h_2}{3};$	(3.7)
Free Board	0.15 m	-
Height of Cylindrical Portion	$H = 2r_1 c_1 + 0.15;$	(3.8)

This culmination of the iterative loop results in a comprehensive and well-defined set of dimensions for the Intze tank, ready for further detailing and implementation in the construction phase.

Step III Member properties and material selection for water tank design

In this step, minimum cross-Sectional properties are assigned to all structural elements of the water tank, such as the top dome, top ring beam, cylindrical walls at the top and bottom, conical domes at the top and bottom, balcony beam, bottom dome, main girder beam, column diameter, bracing size, and foundation beam size, in accordance with IS 3370:2021. Additionally, concrete and steel grades are specified to meet the required standards.

Table 3.5: Minimum cross section and thicknesses of members for conventional tanks

Member	**Minimum cross section/ Thickness**
Top dome thickness t_1	0.12 m
Top ring beam (b_2 x d_2)	0.2 x 0.2 m
Cylindrical wall at top and bottom (t_{3top} & $t_{3bottom}$)	0.2 m at top and 0.2 at bottom initially.
Middle ring beam (b_4 x d_4)	1.0 x 0.12 m
Conical dome at top and at bottom ((t_{5top} & $t_{5bottom}$)	1.5 x t_{3top} at top and 0.2 m at bottom
Bottom dome t_6	0.12 m
Main girder beam (b_7 x d_7)	0.4 x 0.5 m
Column ($D_{c)}$	0.4 m
Braces (b_9 x d_9)	0.2 x 0.5 m
Foundation beam (b_{10} x d_{10})	0.4 x 0.5 m
Raft area	As per calculations

In conclusion, Module I systematically determine the dimensions and structural requirements for elevated water tanks through precise calculations and optimization.

By incorporating key inputs and adhering to IS 3370:2021 standards, this module ensures that the tank design is both efficient and robust. The iterative process guarantees that the final dimensions meet the desired capacity, laying a strong foundation for the subsequent construction phases and ensuring the reliability and durability of the water supply infrastructure.

3.2.2. Module II: Analysis of RCC tank body

This procedure involves systematically analyzing the tank body, joint by joint, through membrane and continuity analyses. Initially, the focus lies on determining the stiffness parameters for the members at the top joint. Subsequently, the process moves to conducting membrane analysis for the members of Joint I, followed by stiffness analysis for Joint II. This sequence continues with membrane analysis for all members of Joint II and then the stiffness calculation for all members of Joint III, alongside membrane analysis for Joint III. At the conclusion of this Section, matrix methods come into play, facilitating the determination of the primary unknowns: the **Net Rotation and Net Displacement** of each joint. Furthermore, based on these findings, member forces for each member of every joint are derived. Steps involved are as follows:

3.2.2.1. Stiffnesses and member forces for all joints

Stiffness parameters are determined utilizing the equations outlined in Table 1.2 of Chapter 1. Furthermore, membrane analysis adheres to the methodology detailed in Section 1.5 of Chapter 1. Rotations and displacements are also computed according to the equations provided in Table 1.2 of Chapter 1. Upon completion of membrane analysis for each member, Total weight of each member i.e. self-weight multiplied by volume and external force loads such as water are calculated. The loads acting on different members of the tank body are as follows:

- **Top dome:** Self weight and Live loads
- **Top ring beam:** Self-weight.
- **Cylindrical wall:** Self-weight and vertical load due to water.

- **Middle ring beam:** Self-weight, bending moments, total vertical load per meter, hoop tension, and horizontal thrust at the top.
- **Conical dome:** Self-weight and water load at the conical dome, total load per meter at the bottom of the conical dome, and horizontal thrust imposed on the bottom support.
- **Bottom dome:** Self-weight, water load at the bottom dome, total load per square meter, and horizontal thrust imposed at the bottom support.
- **Main girder beam:** Self-weight, total load, total load per meter, and net horizontal load at the bottom support

The analysis of each member of the tank body is as follows:

a). Top dome analysis Self-weight, live loads (0.75 kN/m^2) and Finishing loads (0.1 kN/ m^2) are the external forces acting on the Top dome. The thickness of the top dome is t$_1$. Total area of the Dome is calculated as - $2\pi Rh$. For the analysis purposes Forces and stiffnesses calculation are given below-

Table 3.6: Forces and stiffnesses calculations for Top dome

Force calculations		
Forces/Stiffnesses	Formula used	
Self-weight and live loads	$w_1 = (0.75 + 0.1 + t_1 \times 25\ )$	(3.9)
Total weight (SW1)	$SW1 = w_1(2\pi R_1 h_1)$	(3.10)
Meridional Forces T_1	$T_1 = \frac{w_1 \times R_1}{1+\cos\theta_{td}}$	(3.11)
Horizontal forces	$H_1 \frac{= w_1 \times R_1 \cos\theta_{td}}{1+\cos\theta_{td}}$	(3.12)
Horizontal displacement	$\frac{x_{td} = wR_1^2\{(1+\cos\theta_{td})^{-1} - cos\,\theta_{td}\}}{Et_1}$	(3.13)
Rotation	$\psi_{td} = \frac{2wR_1\, sin\,\theta_{td}}{Et_1}$	(3.14)
Stiffnesses calculations		
Moment stiffness (MS1)	$MS1 = \frac{R_1 Et_1(k_{td}+k_{td}^{-1})}{4\times{\lambda_{td}}^3}$	(3.15)
Corresponding stiffness (CorrMS1 or CorrTS1)	$CorrMS1 = \frac{Et_1}{2{\lambda_{td}}^2 k_{td} sin\theta_{td}}$	(3.16)
Thrust stiffness (TS1)	$TS1 = \frac{Et_1}{\lambda_{td} R_1 k_{td} sin^2\theta_{td}}$	(3.17)

b). Top ring beam - For the top ring beam, the self-weight per unit length is calculated and then Total self-weight is then determined by multiplying by the circumference of the beam. Stiffness is then calculated.

Table 3.7: Forces and stiffnesses calculations for Top ring beam

Forces calculations		
Forces/Stiffnesses	Formula used	Equation Number
Self-weight	$w_2 = 25b_2d_2$	(3.18)
Self-weight ($SW2$)	$SW2 = (w_2) \times (2\pi r_1)$	(3.19)
Horizontal displacement	x_1	-
Rotation	ψ_1	-
Stiffness calculations		
Moment stiffness (MS2)	$\frac{Eb_2{d_2}^3}{12{r_1}^2}$	(3.20)
Thrust stiffness (TS2)	$\frac{Eb_2d_2}{{r_1}^2}$	(3.21)

c). Cylindrical wall:

Table 3.8: Forces calculations for cylindrical wall

Forces calculations		
Forces	Formula used	
Self-weight for uniform wall	$w_3 = (25 \times t_{3top})$	(3.22)
Self-weight for tapered	$w_{31} = 25 \times 0.54(t_{3bottom} - t_{3top})0.5$	(3.23)
Self-weight ($SW3$)	$SW3 = 2\pi r_1 w_3(H + 0.5t_{3top}) + 2\pi r_1 w_{31} H$	(3.24)
Vertical Load due to water	$WL_{Wall} = \pi(2r_1 - (t_{3bottom} - t_{3top}))(t_{3bottom} - t_{3top})\gamma_w(H - 0.27);$	(3.25)
Horizontal displacement at top	x_1	-
Rotation at top or bottom	$\psi_{wall} = \frac{\gamma_w \times (r_1)^2}{Et_{3top\ or\ bottom}}$	(3.26)
Horizontal displacement at bottom of wall	$x_{wall} = \frac{\gamma_w \times H \times (r_1)^2}{Et_{3bottom}}$	(3.27)

Table 3.9: Stiffnesses calculations for cylindrical wall

Stiffness calculations		
Moment of Inertia	$Z_{top} = \frac{t_{3top}^3}{12}$; $Z_{bottom} = \frac{t_{3bottom}^3}{12}$	(3.28)
Moment stiffness at top or bottom (MS3t & MS3b)	$\text{MS3t} = 2\mu Z_{top}$ $\text{MS3b} = 2\mu Z_{bottom}$	(3.29) (3.30)
Corresponding stiffness	$2\mu^2 Z_{top}$ and $2\mu^2 Z_{bottom}$ respectively	(3.31)
Thrust stiffness at top or bottom (TS3t & TS3b)	$4\mu^3 Z_{top}$ and $4\mu^3 Z_{bottom}$ respectively	(3.32)

For the cylindrical wall, the self-weight for a uniform wall is determined by the material density and the top wall thickness. For a tapered wall, it involves the difference between the bottom and top thicknesses. Horizontal displacement and rotation at the top of the wall are influenced by the wall's height, radius, and material elasticity. Stiffness calculations include the moment of inertia, Moment stiffness, and thrust stiffness at both the top and bottom, reflecting the wall's resistance to bending and axial loads.

d) **Balcony/ Middle ring beam-** Middle ring beam as mentioned earlier is used as Balcony beam, hence subjected to live load (3 kN/m^2) and Bending moment occurs. Usually the Balcony beam width (b_4) is 1m and depth is in general (d_4) 120 -150 mm.

Table 3.10:Forces calculations for middle ring beam

Force calculations		
Forces/Stiffnesses	Formula used	Eq.No.
Self-weight and Live loads	$w_4 = (3 + 25 \times d_4)$	(3.33)
Self-weight ($SW4$) kN	$SW4 = w_4 \times \{\pi(2r_1 + b_4)b_4\}$	(3.34)
Bending moment	$BM\ at\ MRB = -0.5 w_4 b_4 (b_4 + t_{3bottom})$	(3.35)
Total Vertical per meter Load at level of MRB	$\text{TL}_{MRB} = \frac{\text{SW1+SW2+SW3+}WL_{Wall}\text{+SW4}}{2\pi r_1}$	(3.36)
Hoop tension	$HT\ at\ MRB = 10 \times H \times d_4$	(3.37)
Horizontal thrust at the top	$HThrust\ at\ MRB = \text{TL}_{MRB} / \cot\theta$	(3.38)
Horizontal displacement	x_2	(3.39)
Rotation	ψ_2	(3.40)

Table 3.11: Stiffnesses calculations for Middle ring beam

Stiffnesses calculations		
Moment stiffness (MS4)	$\frac{Eb_4 {d_4}^3}{12{r_1}^2}$	(3.41)
Corresponding stiffness	–	
Thrust stiffness (TS4)	$\frac{Eb_4 d_4}{{r_1}^2}$	(3.42)

The beam also experiences total vertical loads from the structure, hoop tension, which is due to water pressure, and horizontal thrust, which is the lateral force acting at the top of the beam.

e) **Conical dome-** The conical dome structure is subjected to various forces and loads. Thickness of Conical dome is variable. Thickness at top is t_{5top} and $t_{5bottom}$.. The self-weight is calculated based on the thickness at both the top and bottom of the dome. This total weight is distributed across the dome's surface area. Additionally, the dome experiences a water load, which accounts for the volume of water it holds and the geometric dimensions of the dome.

Table 3.12: Stiffnesses calculations for Conical dome

Stiffness calculations		
Moment stiffnesses at Top	$MS5t = \frac{Et_{5top}k_4}{l\tan^2\theta_{cd}(k_1k_4-k_2k_3)}$	(3.56)
Moment stiffnesses at Bottom	$MS5b = \frac{Et_{5bottom}k_4}{l\tan^2\theta_{cd}(k'_1k'_4-k'_2k'_3)}$	(3.57)
Corresponding stiffnesses at top	$\mathrm{CorrMS5t/TS5t} = \frac{Et_{5top}k_2}{l\tan^2\theta_{cd}(k_1k_4-k_2k_3)}$	(3.58)
Corresponding stiffnesses at Bottom	$\mathrm{CorrMS5b} = \frac{Et_{5bottom}k_2}{l\tan^2\theta_{cd}(k'_1k'_4-k'_2k'_3)}$	(3.59)
Thrust stiffnesses at Top	$TS5t = \frac{Et_{5top}k_1}{l\tan^2\theta_{cd}(k_1k_4-k_2k_3)}$	(3.60)
Thrust stiffnesses at Bottom	$TS5b = \frac{Et_{5bottom}k_1}{l\tan^2\theta_{cd}(k'_1k'_4-k'_2k'_3)}$	(3.61)

The total load per meter at the bottom of the conical dome includes the cumulative self-weights of all components and the water load, adjusted for the dome's geometry and angle. This load results in a meridional force at the top, which is the force along the curved surface of the dome, and a horizontal thrust imposed on the bottom support. The horizontal forces at both the top and bottom of the conical dome are influenced by the water pressure and the dome's self weight. These calculations ensure the dome's stability and ability to withstand both vertical and horizontal stresses.

Table 3.13: Forces calculations for conical dome

Force calculations		
Forces/Stiffnesses	Formula used	Eq.No
Self-weight (kN/m^2)	$w_5 = 25 \times 0.5 \times (t_{5bottom} + t_{5top})$	(3.43)
Total weight (kN)	$SW5 = w_5 \times \pi(\mathrm{r}_1 + \mathrm{r}_2)l;$	(3.44)
Water load at C.D. WL_{Cd}	$WL_{CD} = \gamma_w\ (\pi r_1{}^2 H + \pi l \frac{r_1{}^2+r_2{}^2+r_1 r_2}{3} - \pi r_2^2 \frac{H+l}{3});$	(3.45)
Total load up to C.D.	$SW1 + SW2 + SW3 + SW4 + WL_{Wall} + SW5 + WL_{Cd}$	(3.46)
Total load per meter at Bottom of C.D.	$\mathrm{TL}_{CD} = \frac{\text{Total load up to C.D.}}{(2\pi r_2 \cos\theta)}$	(3.47)
Meridional force	$T_{5Top} = \mathrm{TL}_{CD}/\cos A_r$	(3.48)
Horizontal thrust imposed on the bottom support	$HThrust = \mathrm{TL}_{CD} \times \sin\theta$	(3.49)
Horizontal force at Top of CD	$HF_{CDTop} = \gamma_w\ H r_1 \sec A_r + 25 t_{5top} r_1\ \tan A_r$	(3.50)
Horizontal force at Bottom of CD	$HF_{CDBottom} \frac{= \gamma_w\ r_2(H + r_1 - r_2)}{\cos A_r} + 25 r_2\ t_{5bottom} \tan A_r$	(3.51)
Horizontal displacement at top	$x_{cd1} = \frac{HF_{CDTop} \times r_1}{E t_{5top}}$	(3.52)
Horizontal displacement at Bottom	$x_{cd2=} \frac{HF_{CDTop} \times r_2}{E t_{5bottom}}$	(3.53)
Rotation at top	$\psi_{cd1=} \frac{2HF_{CDTop} + T_{5Top}}{E t_{5top}} (\tan A_r)$	(3.54)
Rotation at bottom	$\psi_{cd2=} \frac{(2HF_{CDBottom} + T_{5Bottom}) \tan A_r}{E t_{5Bottom}}$	(3.55)

f). Bottom dome- Bottom dome is having a thickness of t_6 has self load and water load acting on the Bottom dome. Forces and stiffnesses are calculated as per Section. For Horizontal displacement is due Case I and Case II both are considered.

Table 3.14: Forces and stiffnesses calculations for Bottom dome

Force calculations		
Forces/Stiffnesses	Formula used	Equation Number
Self-weight (kN/m^2)	$w_6 = 25t_6$	(3.62)
Total weight (kN)	$SW6 = 2\pi h_2 R_2 w_6$	(3.63)
water load at Bottom dome	$WL_{BD} = \{\pi {r_2}^2 (H + r_1 - r_2) - \pi h_2^2 \frac{3R_2 - h_2}{3}\}\gamma_w$	(3.64)
Total load at Bottom dome per square meter	$\mathrm{TL}_{BD} = \frac{\{SW6+WL_{BD}\}}{2\pi R_2 h_2}$	(3.65)
Horizontal thrust imposed at bottom support	$HForce_{BD} = \frac{\{SW6+WL_{BD}\}R_2 cos\theta_2}{(1+cos\theta_2)\{2\pi h_2 R_2\}}$	(3.66)
Horizontal displacement	$x_{bd} = \frac{w_6 R_2^2 \sin\theta_{bd}\{(1+\cos\theta_{bd})^{-1} - cos\,\theta_{bd}\}}{Et_6} + \frac{w_6 R_2^3 \sin\theta_{bd}\{2\ \cos 2\theta_{bd} + \cos\theta_{bd} - 3\}}{(6(1+\cos\theta_{bd})Et_6)} - \frac{\gamma_w R_2^2 (H + r_1 - r_2 - h_2) cos\theta_{bd}}{2Et_6}$	(3.67)
Rotation	$\psi_{bd=} \frac{2w_6 R_2 \sin\theta_{bd}}{Et_6} - \frac{10R_2^2 \sin\theta_{bd}}{Et_6}$	(3.68)
Stiffness calculations		
Moment stiffnesses MS6	$MS6 = \frac{R_2 Et_6 (k_{bd} + k_{bd}^{-1})}{4 \times {\lambda_{bd}}^3}$	(3.69)
Corresponding stiffnesses	$CorrMS6\ or\ Corr\ TS6 = \frac{Et_6}{2{\lambda_{bd}}^2 k_{bd} sin\theta_{bd}}$	(3.70)
Thrust stiffnesses TS6	$\mathrm{TS6} = \frac{Et_6}{\lambda_{bd} R_2 k_{bd} sin^2\theta_{bd}}$	(3.71)

f). Main ring beam or Main girder beam-

Equations provided in Table 1.3 are used to solve for primary unknows using Stiffness Matrices. For all the Three Joints Direct stiffness matrices are written. The direct stiffness matrix, often referred to simply as the stiffness matrix, is a fundamental concept in structural mechanics and finite element analysis. It relates the forces applied to a structure to the displacements at specific points within that structure. The stiffness matrix is typically denoted by {K} and forms the basis of the stiffness method, which is a common approach to solving structural analysis problems.

Table 3.15: Forces and stiffnesses calculations for Main ring beam

Forces calculated

Forces/Stiffnesses	Formula used	Eq.No.
Self-weight w_7	$w_7 = 25 b_7 d_7$	(3.72)
Self-weight $SW7$	$SW7 = w_7 \times 2\pi r_2$	(3.73)
Total load at Main girder beam TL_{MGB}	$\mathrm{TL}_{MGB} = SW1 + SW2 + SW3 + SW4 + SW5 + SW6 + SW7 + capacity \times 10;$	(3.74)
Total load /m	$Total\ Load\ per\ meter = (\mathrm{TL}_{MGB}/(2\pi r_2)$	(3.75)
Net Horizontal force at Bottom support	$\mathrm{H_{NET}} = HF_{CDBottom}$- $HForce_{BD}$	(3.76)
Horizontal displacement	x_3	(3.77)
Rotation	ψ_3	(3.78)
Stiffnesses calculation		
Moment stiffness	$\frac{E b_7 {d_7}^3}{12 {r_2}^2}$	(3.79)
Corresponding stiffness	-	-
Thrust stiffness	$\frac{E b_7 d_7}{{r_2}^2}$	(3.80)

3.2.2.2. Defining stiffness matrix

For a given structure, the stiffness matrix {K} is a square matrix that relates the vector of nodal displacements {Δ} to the vector of applied forces {P} through the linear equation:

$$\{P\}=\{K\}\{\Delta\} \tag{3.81}$$

$$\{\Delta\} = \{K\}^{-1}(P) \tag{3.82}$$

- (P) is the force vector.
- {Δ} is the displacement vector.
- {K}is the stiffness matrix.

a) **Joint I-** Global stiffness matrix Constitutes of Stiffnesses of members joining at joint I that are Top dome, Top ring beam and wall.

- Global stiffness matrix for Joint I-

$$\{K_1\}_{2\times2}=\begin{bmatrix} MS1 + MS2 + MS3t & \mathrm{CorrTS1} + \mathrm{CorrTS3t} \\ \mathrm{CorrMS1} + \mathrm{CorrMS3} & TS1 + TS2 + TS3t \end{bmatrix} \tag{3.83}$$

$$\{K_1\}_{2\times2} = \begin{bmatrix} \frac{R_1 E t_1 (k_{td} + k_{td}^{-1})}{4 \times {\lambda_{td}}^3} + \frac{E b_2 {d_2}^3}{12 {r_1}^2} + 2\mu Z_{top} & \frac{E t_1}{2{\lambda_{td}}^2 k_{td} sin\theta_{td}} + 2\mu^2 Z_{top} \\ \frac{E t_1}{2{\lambda_{td}}^2 k_{td} sin\theta_{td}} + 2\mu^2 Z_{top} & \frac{E t_1}{\lambda_{td} R_1 k_{td} sin^2\theta_{td}} + \frac{E b_2 d_2}{{r_1}^2} + 4\mu^3 Z_{top} \end{bmatrix} \tag{3.84}$$

- Force Vector Equation at Joint I

$$\{P_1\}_{2\times1}=\begin{pmatrix} 0 - (MS1(-\psi_{td}) + CorrTS1(-x_{td}) + MS3t(-\psi_{wall})) \\ \frac{w_1 \times R_1 \cos\theta_{td}}{1+\cos\theta_{td}} - CorrMS1(-\psi_{td}) + TS1(-x_{td}) + CorrMS3t(-\psi_{wall}) \end{pmatrix} \tag{3.85}$$

- Displacement vector at Joint I

$$\{\Delta_1\}_{2\times1} = \{K_1\}_{2\times2}^{-1} \times \{P_1\}_{2\times1} \tag{3.86}$$

$$\begin{pmatrix} x_1 \\ \psi_1 \end{pmatrix} = \begin{bmatrix} MS1 + MS2 + MS3t & \mathrm{CorrTS1} + \mathrm{CorrTS3t} \\ \mathrm{CorrMS1} + \mathrm{CorrMS3} & TS1 + TS2 + TS3t \end{bmatrix}^{-1} \begin{pmatrix} {P_1}' \\ {P_1}'' \end{pmatrix} \tag{3.87}$$

b) **Joint II-** Global stiffness matrix Constitutes of Stiffnesses of members joining at joint II that are Wall, Middle ring beam and Conical dome

- Global stiffness matrix for Joint II-

$$\{K_2\}_{2\times2}=\begin{bmatrix} MS3b + MS4 + MS5t & \mathrm{CorrTS3b} + \mathrm{CorrTS5t} \\ \mathrm{CorrMS3b} + \mathrm{CorrMS5t} & \mathrm{TS3b} + \mathrm{TS4} + \mathrm{TS5t} \end{bmatrix} \tag{3.88}$$

$$\{K_2\}_{2\times 2} = \begin{bmatrix} 2\mu Z_b + \frac{Eb_4{d_4}^3}{12{r_1}^2} + \frac{Et_{5top}k_4}{l\tan^2\theta_{cd}(K)} & 2\mu^2 Z_b + \frac{Et_{5top}k_2}{l\tan^2\theta_{cd}(\mathrm{K})} \\ 2\mu^2 Z_b + \frac{Et_{5top}k_2}{l\tan^2\theta_{cd}(\mathrm{K})} & 4\mu^3 Z_b + \frac{Eb_4 d_4}{{r_1}^2} + \frac{Et_{5top}k_1}{l\tan^2\theta_{cd}(\mathrm{K})} \end{bmatrix} \tag{3.89}$$

- Force Vector Equation at Joint II

$$\{P_2\}_{2\times 1} = \begin{pmatrix} BM\ at\ MRB - (MS3b(-\psi_w) + CorrTS3b(-x_w) + MS5t(-\psi_{t5top}) \\ \mathrm{TL}_{MRB} + HF_{MRB} - (CorrMS3b(-\psi_w) + TS3b(-x_w) + CorrMS5t(-\psi_{cd1}) + TS5t(-x_{cd1})) \end{pmatrix} \tag{3.90}$$

- Displacement vector at Joint II

$$\{\Delta_2\}_{2\times 1} = \{K_2\}^{-1}_{2\times 2} \times \{P_2\}_{2\times 1} \tag{3.91}$$

$$\begin{pmatrix} x_2 \\ \psi_2 \end{pmatrix} = \begin{bmatrix} MS3b + MS4 + MS5t & \mathrm{CorrTS3b + CorrTS5t} \\ \mathrm{CorrMS3b + CorrMS5t} & \mathrm{TS3b + TS4 + TS5t} \end{bmatrix}^{-1} \begin{pmatrix} P_2{}' \\ P_2{}'' \end{pmatrix} \tag{3.92}$$

where $K = k_1k_4 - k_2k_3$ and $\psi_w = \psi_{wall}$

c) **Joint III-** Global stiffness matrix Constitutes of Stiffnesses of members joining joint III that are Conical dome, Bottom dome and Bottom beam

- Global stiffness matrix for Joint III-

$$\{K_3\}_{2\times 2} = \begin{bmatrix} MS5b + MS6 + MS7 & \mathrm{CorrTS5b + CorrTS6} \\ \mathrm{CorrMS5b + CorrMS6} & \mathrm{TS5b + TS6 + TS7} \end{bmatrix} \tag{3.93}$$

$$\{K_3\}_{2\times 2} = \begin{bmatrix} \frac{Et_{5b}k_4}{l\tan^2\theta_{cd}K'} + \frac{R_2Et_6(k_{bd} + k_{bd}^{-1})}{4\times{\lambda_{bd}}^3} + \frac{Eb_7{d_7}^3}{12{r_2}^2} & \frac{Et_{5b}k_2}{l\tan^2\theta_{cd}K'} + \frac{Et_6}{\lambda_{bd}R_2k_{bd}sin^2\theta_{bd}} \\ \frac{Et_6}{\lambda_{bd}R_2k_{bd}sin^2\theta_{bd}} + \frac{Et_{5top}k_2}{l\tan^2\theta_{cd}\mathrm{K}'} & \frac{Et_6}{\lambda_{bd}R_2k_{bd}sin^2\theta_{bd}} + \frac{Eb_7d_7}{{r_2}^2} + \frac{Et_{5top}k_1}{l\tan^2\theta_{cd}\mathrm{K}'} \end{bmatrix} \tag{3.94}$$

- Force Vector at Joint II

$$\{P_3\} = \begin{pmatrix} 0 - (MS5b(-\psi_{cd2}) + CorrTS5b(-x_{cd2}) + MS6(-\psi_{bd}) + CorrTS6(-x_{bd}) \\ \mathrm{HNET} - (CorrMS5b(-\psi_{cd2}) + TS5b(-x_{cd2}) + CorrMS6(-\psi_{bd}) + TS6(-x_{bd})) \end{pmatrix} \tag{3.95}$$

- Displacement vector at Joint III

$$\{\Delta_3\}_{2\times 1} = \{K_3\}^{-1}_{2\times 2} \times \{P_3\}_{2\times 1} \tag{3.96}$$

$$\begin{pmatrix} x_3 \\ \psi_3 \end{pmatrix} = \begin{bmatrix} MS5b + MS6 + MS7 & \mathrm{CorrTS5b} + \mathrm{CorrTS6} \\ \mathrm{CorrMS5b} + \mathrm{CorrMS6} & \mathrm{TS5b} + \mathrm{TS6} + \mathrm{TS7} \end{bmatrix}^{-1} \begin{pmatrix} P_3{}' \\ P_3{}'' \end{pmatrix} \quad (3.97)$$

3.2.2.3. Calculations of forces acting on members

Moments, Outward thrust and Hoop tensions are calculated after the calculation of net rotations and displacements.

Table 3.16: Moment calculations of tank body elements

Member	Moments	Eq.No.
Moment at TD	$MS1\ (\psi_1 - \psi_{td}) +\ CorrTS1\ (x_1 - x_{td})$	(3.98)
Moment at TRB	$\mathrm{MS2}(\psi_1)$	(3.99)
Moment at wall (Top)	$MS3t\ (\psi_1 - \psi_{wall}) + \mathrm{CorrTS3t} \times (x_1)$	(3.100)
Moment at wall (Bottom)	$MS3b\ (\psi_2 - \psi_{wall}) + CorrTS3b\ (x_2 - x_{wall})$	(3.101)
Balcony beam	$MS4\ (\psi_2)$	(3.102)
CD Top	$MS5t\ (\psi_2 - \psi_{cd1}) + CorrTS5t\ (x_2 - x_{cd1})$	(3.103)
CD Bottom	$MS5b\ (\psi_3 - \psi_{cd2}) + CorrTS5b\ (x_3 - x_{cd2})$	(3.104)
Bottom beam	$MS7\ (\psi_3)$	(3.105)
Bottom dome	$MS6\ (\psi_3 - \psi_{bd}) + CorrTS6\ (x_3 - x_{bd})$	(3.106)

Table 3.17: Outward thrust calculations of tank body Elements

Member	Outward thrust	Eq. No.
Top dome	$\mathrm{CorrMS1}\ (\psi_1 - \psi_{td}) + \mathrm{TS1}\ (x_1 - x_{td})$	(3.107)
Top ring beam	$\mathrm{TS2}\ (x_1)$	(3.108)
Wall (Top)	$CorrMS3t(\psi_1 - \psi_{wall}) + TS1\ (x_1)$	(3.109)
Wall (Bottom)	$CorrMS3b\ (\psi_2 - \psi_{wall}) + TS3b\ (x_2 - x_{wall})$	(3.110)
Balcony beam	$TS4\ (x_2)$	(3.111)
CD top	$CorrMS5t\ (\psi_2 - \psi_{cd1}) + TS5t\ (x_2 - x_{cd1})$	(3.112)
CD bottom	$CorrMS5b\ (\psi_3 - \psi_{cd2}) + TS5b\ (x_3 - x_{cd2})$	(3.113)
Bottom beam	$TS7\ (x_3)$	(3.114)
Bottom dome	$\mathrm{CorrMS6}\ (\psi_3 - \psi_{bd}) + \mathrm{TS6}\ (x_3 - x_{bd})$	(3.115)

Table 3.18: Calculation of the hoop tensions

Members	Hoop tension (in N)	Eq. No.
Top dome	$HTension1 = x_1 \times 1000 \times t_1 \times 1000/r_1/1000000;$	(3.116)
TRB	$HTension2 = b_2 d_2 x_1 * 1000000/1000/r_1;$	(3.117)
Wall (Top)	$HTension3t = t_{3\text{top}} x_1 * 1000000/1000000/r_1;$	(3.118)
Wall (Bottom)	$HTension3b = t_{3\text{bottom}} x_2 * 1000000/1000000/r_1;$	(3.119)
Balcony beam	$HTension4 = 1000000(t_4^2 + (1 - t_4)(\, t_4 + 0.2)\ 0.5)x_2/1000/r_1;$	(3.120)
CD Top	$HTension5t = t_{5top} \times 1000000 \times x_2/1000000/r_1;$	(3.121)
CD Bottom	$HTension5b = t_{5top} \times 1000000 \times x_3/1000000\ /r_2;$	(3.122)
Bottom beam	$HTension7 = b_7 d_7 x_3 \times 1000000/1000/r_2;$	(3.123)
Bottom dome	$HTension6 = x_3 \times t_3 \times 1000000/r_2/1000000;$	(3.124)

3.2.2.4. Calculation of reinforcement and checks under limit state of serviceability & application of heuristic optimization

Table 3.19: Calculation of reinforcement under Limit state of serviceability

Member	Reinforcement A_{st} under limit state of serviceability	Eq. No.
Top dome	$A_{st}H_1 = HTension1 \times 1000/f_y;$	(3.125)
Top ring beam	$A_{st}H_2 = HTension2/f_y;$	(3.126)
Wall (Top)	$A_{st}H_{3t} = HTension3t \times 1000/f_y;$	(3.127)
Wall (Bottom)	$A_{st}H_{3b} = HTension3b \times 100/f_y;$	(3.128)
Balcony beam	$A_{st}H_4 = HTension4/f_y;$	(3.129)
CD Top	$A_{st}H_{5t} = HTension5t \times 1000/f_y;$	(3.130)
CD Bottom	$A_{st}H_{5b} = HTension5b \times 1000/f_y;$	(3.131)
Bottom beam	$A_{st}H_7 = HTension7/f_y;$	(3.132)
Bottom dome	$A_{st}H_6 = HTension6 \times 1000/f_y;$	(3.133)

Table 3.20: Checks for f_{ct}

	Check for f_{ct}	
Top dome	$f_{ct1} = \frac{HTension1 \times 1000}{\mathrm{t}_1 \times 1000 \times 1000 + 14.6 \times A_{st}H_1}$	(3.134)
Top ring beam	$f_{ct2} = \frac{HTension2}{\mathrm{b}_2\mathrm{d}_2 \times 1000 \times 1000 + \mathrm{m} \times A_{st}H_2}$	(3.135)
Wall (Top)	$f_{ct3t} = \frac{HTension3t \times 1000}{\mathrm{t}_{3top} \times 1000000 + \mathrm{m} \times A_{st}H_{3t}}$	(3.136)
Wall (Bottom)	$f_{ct3b} = \frac{HTension3b \times 1000}{\mathrm{t}_{3bottom} \times 1000000 + \mathrm{m}A_{st}H_{3b}}$	(3.137)
Balcony beam	$f_{ct4} = \frac{HTension4}{\mathrm{b}_4\mathrm{d}_4 \times 1000 \times 1000 + \mathrm{m} \times A_{st}H_4}$	(3.138)
CD Top	$f_{ct5t} = \frac{HTension5t \times 1000}{\mathrm{t}_{5top} \times 1000000 + \mathrm{m} \times A_{st}H_{5t}}$	(3.139)
CD Bottom	$f_{ct5b} = \frac{HTension5b \times 1000}{\mathrm{t}_{5bottom} \times 1000000 + \mathrm{m}A_{st}H_{5b}}$	(3.140)
Bottom beam	$f_{ct7} = \frac{HTension7}{\mathrm{b}_7\mathrm{d}_7 \times 1000 \times 1000 + \mathrm{m} \times A_{st}H_7}$	(3.141)
Bottom dome	$f_{ct6} = \frac{HTension6 \times 1000}{\mathrm{t}_6 \times 1000000 + \mathrm{m} \times A_{st}H_6}$	(3.142)

If f_{ct} is below the Table 1.6 provided in Section 1.8.2 Then the loop will get terminate else will repeat with an increment of 5mm to the unsafe element. **Heuristic optimization** is employed. Top dome thickness and cross-Section of balcony beam is kept fixed. The whole Modules I and II will rerun until all the member found safe.

3.2.3. Module III- Design of members of tank

Once all the members are found safe, The design process of members starts. Upto this level, Cross Sections of the members are fixed, The reinforcement are to be calculated for Limit State of collapse and crack width calculation check. Reinforcement is calculated for hoop tension and Bending Momemts from the following formulas:

3.2.3.1. General Steps for the calculation of reinforcement

General Steps for the Calculation of reinforcement for hoop tensions, Bending and Shear forces for any member are given in Table 3.17.

Table 3.21: General Steps for calculation of reinforcement for any member

General steps for the calculation of reinforcement for any member		
Step I	Reinforcement for hoop tension	$A_{stH} = \frac{H_u}{f_y}$ (3.143)
Step II	Reinforcement for Moment A_{stM}	$A_{stM} = 0.5bd\frac{f_{ck}}{f_y}\left\{1 - \sqrt{1 - \frac{4.6M_u \times 10^6}{f_{ck}bd}}\right\}$ (3.144) Where b and d are cross-Sections of beams in mm; in the case of wall or conical domes cross-Sections are 1000 x t where t is thickness of elements.
Step III	Calculation of Number of bars	$NOs = \frac{A_{st}}{f_y}$ (3.145)
Step IV	Spacing calculations	$spacing = \frac{A_{\emptyset} \times 1000}{A_{st}}$ (3.146) where Φ is the Diameter of bar used
Step V	Calculations of shear reinforcement	Maximum shear stress is calculated τ_{max} Percentage reinforcement $p = \frac{A_{stH} \times 100}{b \times d}$ (3.147) Table 19 of IS 456:2000 is built-in code as a string function. τ_c is fetched from the string with the help of the do-while loop τ_{max} & τ_c are compared if $\tau_{max} < \tau_c$ shear reinforcement is not needed. Nominal Reinforcement provided. If $\tau_{max} > \tau_c$ shear reinforcement is needed. In this case, reinforcement is calculated for the difference of shear reinforcement $(\tau_{max} - \tau_c)$ $SF_{net} = (\tau_{max} - \tau_c)bd$ (3.148) $Spacing = \frac{A_{\emptyset} \times 4 \times 0.87 \times f_y}{(\tau_{max} - \tau_c) \times bd}$ (3.149)
Step VI	Checks	Spacing and reinforcement are checked for minimum and maximum criteria as specified by IS 456 and IS 3370. These checks are embedded in the code.

3.2.3.2. Crack width calculations

Crack width calculations are used in accordance with Annextures A & B of IS 3370:2021. The equations are explained in Section 18.2 of this study. Crack widths are checked for Hoop and bending both for all the water facing elements. General Steps for the Crack width calculations for any member are given in Table 3.18.

Table 3.22: Calculation of crack width for members

Crack width calculations for direct tension		
Parameter	Formula	Eq.No.
Stress at any level Fs	$F_S = \frac{HTension}{A_{st}}$	(3.150)
Strain at level of Stress e_1	$e_1 = \frac{F_S}{200000}$	(3.151)
stiffening effect of concrete in direct tension	$e_2 = \frac{2 \times (b \times d)}{3 \times A_{st} \times 200000}$	(3.152)
Average steel strain	$e_m = e_1 - e_2$	(3.153)
Crack width	$w_{cr} = 3a_{cr}\varepsilon_m$ $a_{cr} = \sqrt{C_{min}^2 + S_p^2}$ $S_p = 0.5\,(t - C_{min} \times 2)/(Nos - 1)$	(3.154-3.156)
Crack width calculations for flexure		
Stress at any level Fs	$F_S = \frac{M}{A_{st}(t - x_{n.a.})}$	(3.157)
Depth of neutral axis	$x_{n.a.} = \frac{0.87 f_y (A_{st})_{pro}}{0.36\, f_y b}$	(3.158)
Strain at level of Stress	$e_1 = \frac{(t - x_{n.a.}) \times F_S}{(t - x_{n.a.} - C_{min}) \times 200000}$	(3.159)
Stiffening effect of concrete in direct tension	$e_2 = \frac{(t - x_{n.a.})^2}{3(A_{st})_{pro} \times 200000(a' - x_{n.a.})}$	(3.160)
Crack width	$w_{cr} = \frac{3a_{cr}\varepsilon_m}{1 + 2(a_{cr} - C_{min})/(t - x_{n.a.})}$	(3.161)

3.2.3.3. Design of the top dome

Upto this level, Cross Sections of the members are fixed, The reinforcement are to be calculated for Limit State of collapse and crack width calculation check. As discussed under various above Sections 3.2.3.1-3.2.3.2. The top dome is to be checked for meridional and circumferential stresses.

Table 3.23: Design steps used in the design of Top dome

Step I – Calculations of ultimate forces and stresses		
Factored hoop tension	$H_u = 1.5 \times HTension1$	(3.162)
Factored hoop stresses	$H_{stress} = H_u/(t_1 \times 1000)$	(3.163)
Factored meridional thrust	$T_u = 1.5 \times T1$	(3.164)
Meridional stresses	$T_{stress} = T_u/(t_1 \times 1000)$	(3.165)
Checks	T_{stress} & $H_{stress} \leq 8N/mm^2$;	(3.166)
If found safe loop will proceed further else will terminate and will restart with increasing the thickness from Module I.		
Step II – Calculation of reinforcement		
Reinforcement calculation A_{st1}	Adopt nominal reinforcement of 0.28 percent of Cross Section of dome both ways	-
Spacing Calculation	Spacing is calculated with all the checks as IS 456:2000. Spacing should be less than three times the effective depth of Top dome (3× t_{eff}). 10 mm diameter bar is generally used Top dome $spacing = \frac{78.5 \times 1000}{A_{st1}}$	(3.167)
Step III- Estimation of Concrete and steel used		
Concrete (A1)	$A1 = 2\pi R_1 t_1 h_1$	(3.168)
Steel estimation Q1	$Q1 = 2R1 \times 0.28 \times s_w/100;$	(3.169)

The program will at last print the Final Cross Section of Top dome, Reinforcement provided, and quantity estimated.

3.2.3.4. Design of the top ring beam

Top ring beam reinforcement is calculated for hoop tension and Bending moment occurring at the Top beam.

Table 3.24: Design steps used in the design of Top ring beam

Step I – Calculations of ultimate forces and stresses		
Factored hoop tension	$H_u = 1.5 \times HTension2$	(3.170)
Factored bending moment	$M_u = 1.5 \times M2$	(3.171)
Reinforcement calculation for hoop tension & moment	A_{stH2}, A_{stM2} from formulas in Table 3.15	-
Check for nominal reinforcement	Checks are applied as per IS 3370 & IS 456	-
Calculation of the number of bars for main reinforcement & spacing for distribution reinforcement	As per Equation "Nos" and "spacing" are calculated as per Table 3.21	-
Crack width check	Crack width are checked as per Table 3.22.	
If crack width is found satisfactory the loop with continue for Estimation else Section will be redesigned (Steel is increased), if still found unsafe the thickness is increased and loop will restart from Module I.		
Estimation of Concrete and steel used		
Concrete (A2)	$A2 = 2\pi b_2 d_2 r_1$	(3.172)
Steel estimation (Main)	$Q21 = 2\pi r_1 \times Nos \times A_{\varphi main} s_w;$	(3.173)
Steel estimation (Distribution) Q22	$Q22 = 4(t_2 - 2 \times 0.04) s_w \frac{\pi(\varphi_d)^2}{4} \frac{2\pi r_1}{spacing}$	(3.174)
Steel Total Q2	Q2= Q21+Q22;	(3.175)

3.2.3.5. Design of the cylindrical wall

Cylindrical wall reinforcement is calculated for hoop tension and Bending moment occurring and Maximum reinforcement occurs at the bottom of the wall as Hoop tension varies with the height of water and reinforcement is gradually reduced to minimum at the top. Further the crack width is checked for the member. Here in this member the thicknesses of member at top and bottom can be different, diameter of the hoop reinforcement can is reduced at the top and vertical or distribution reinforcement also reduced at the top of the wall. This member is connected at two joints, top & middle joints, Hence the hoop tension and moments occurring is different at top and bottom of the wall. Reinforcement is calculated at every 1 m height using for loop for optimizing the material usage.

Table 3.25: Design steps used in the design of cylindrical wall

Calculations of ultimate forces and stresses		
Factored hoop tension at top	$H_u = 1.5 \times HTension3t$	(3.176)
Factored hoop tension at bottom	$H_u = 1.5 \times HTension3b$	(3.177)
Factored hoop tension is calculated at every 1 m height	$H_u = 1.5 \times HTension3b \times h/H$ where 'h' varies from 0 – H meters	(3.178)
Factored bending moment	$M_u = 1.5M3t, M_u = 1.5M3b$	(3.179)
Reinforcement calculation for hoop tension & moment	A_{stH3}, A_{stM3} from formulas in equation reinforcement is provided on both the face of wall. The diameter of the Hoop Reinforcement starts from 12 mm with an increment of 2 mm at every 1 m interval. Distribution reinforcement is also calculated.	-
Check for nominal reinforcement	Checks are applied as per IS 3370 & IS 456	-
Calculation of spacing for main reinforcement & distribution reinforcement	As per Equation "spacing" are calculated and check for minimum and maximum spacing is checked. (Table 3.15)	-
Cracked width check	Crack widths are checked for every 1 m interval and if found unsafe member is redesigned (Table 3.16)	-
If crack width is found satisfactory the loop with continue for Estimation else Section will be redesigned (Steel is increased), if still found unsafe the thickness is increased and loop will restart from Module II.		

Table 3.26: Estimation of quantities of cylindrical wall

Estimation of Concrete and steel used		
Concrete (A3)	$A3 = \frac{\pi}{2} H(2r_1 + t_{3bottom})(t_{3bottom} + t_{3top})$	(3.180)
Steel estimation (Main)	$Q31 = Q31 + 2\{2\pi r_1 \frac{\pi(\varphi_m)^2}{4} \frac{1000}{spacing} sw\}$ Q31 is added cumulative at each 1 m as φ_m and spacing can vary at each 1m interval with initial Q31 = 0;	(3.181)
Steel estimation (distribution)	$Q32 = Q32 + 2\left\{\frac{\pi(\varphi_d)^2}{4} 2\pi r_1 swH \frac{1000}{spacing}\right\}$ Q32 is added cumulative at each 1 m with initial Q32 = 0	(3.182)
Steel total Q3	Q3= Q31+Q32;	(3.183)

3.2.3.6. Design of the Middle ring beam

Balcony beam is designed for the bending moment occurring due to balcony projection. Balcony beam can have different sections at the ends. It acts like a cantilever beam.

Table 3.27: Design steps used in the design of Middle ring beam

Calculations of ultimate forces and stresses		
Factored hoop tension	$H_u = 1.5 \times HTension4$	(3.184)
Factored bending moment	$M_u = 1.5 \times BM\ at\ MRB$	(3.185)
Reinforcement calculation for hoop tension & moment	A_{stH4}, A_{stM4} from formulas in Equation	-
Check for nominal reinforcement	Checks are applied as per IS 3370, IS 456	-
Calculation of spacing for main reinforcement & distribution reinforcement	As per Equation “spacing” are calculated and check for minimum and maximum spacing is checked.	-

If crack width is found satisfactory the loop with continue for Estimation else Section will be redesigned (Steel is increased), if still found unsafe the thickness is increased and loop will restart from Module I.

Table 3.28: Estimation of concrete and steel for Middle ring beam

Estimation of concrete and steel used		
Concrete (A4)	$A4 = \pi H(2r_1 + b_4) \times t_4 \times b_4$	(3.186)
Steel estimation (Main) Q41	$Q41 = 2\pi r_1 \frac{\pi(\varphi_m)^2}{4} Nos \times \text{sw}$	(3.187)
Steel estimation (Distribution)	$Q42 = \frac{\pi(\varphi_d)^2}{4} \times b_4 \frac{2\pi r_1}{spacing} sw$	(3.188)
Steel total Q4	Q4= Q41+Q42;	(3.189)

3.2.3.7. Design of the conical dome

Conical dome can have uniform thickness throughout or can be variable ($t_{5\text{top}}, t_{5\text{bottom}}$). Conical here in this case Main reinforcement is different for Top and bottom of conical dome. This member is connected at two joints, Middle and Bottom joints, Hence the Hoop tension and moments occurring is different at top and bottom of the wall.

Based on the respective forces, the reinforcement area for hoop tension and bending moment is determined individually for the top and bottom portions using the formulas provided in Table 3.15. Distribution reinforcement is provided on both the inner and outer faces as per IS 456:2000. The spacing for main and distribution reinforcement is calculated and checked against the permissible limits given in Table 3.15. Crack width is verified according to the guidelines in Table 3.22 to ensure that serviceability criteria are met. Once all checks are satisfactorily completed, the final reinforcement detailing is carried out.

Table 3.29: Design steps used in the design of conical dome

Step I – Calculations of ultimate forces and stresses		
Factored hoop tension at top	$H_u = 1.5 \times HTension5t$	(3.190)
Factored hoop tension at Bottom	$H_u = 1.5 \times HTension5b$	(3.191)
Factored BM at Top	$M_u = 1.5 \times M3t$	(3.192)
Factored BM at bottom	$M_u = 1.5 \times M3b$ at top	(3.193)
Reinforcement calculation for hoop tension & moment	A_{stH5t}, A_{stH5b}, A_{stM5t} $andA_{stM5b}$ are calculated from formulas in Table 3.15. Distribution reinforcement is calculated as per IS 456:200. Main and distribution reinforcement is provided on both the face of dome	-
Check for nominal reinforcement	Checks are applied as per IS 3370 & IS 456	-
Calculation of spacing for main & distribution reinforcement	As per Equation “spacing” are calculated and check for minimum & maximum spacing is checked. (Table 3.15)	-
Cracked width check	Checked as per Table 3.22	-
If crack width is found satisfactory the loop with continue for Estimation else Section will be redesigned (Steel is increased), if still found unsafe the thickness is increased and loop will restart from Module I.		
Estimation of Concrete and steel used		
Concrete (A4)	$A5 = 0.5\pi l\left(r_1 + r_2 + t_{5top}\right)\left(t_{5top} + t_{5b}\right)$	(3.194)
Steel estimation (Main)	$Q51 = 2\pi(r_1 + r_1)l\frac{\pi(\varphi_m)^2}{4}\frac{1000}{spacing}sw$	(3.195)
Steel estimation (Distribution)	$Q52 = 2\pi(r_1 + r_1)l\frac{\pi(\varphi_{md})^2}{4}\frac{1000}{spacing_d}sw$	(3.196)
Steel Total Q5	Q5= Q51+Q52;	(3.197)

3.2.3.8. Design of the bottom dome

The Bottom dome is to be checked for meridional and circumferential stresses.

Table 3.30: Design steps used in the design of bottom dome.

Step I – Calculations of ultimate forces and stresses		
Factored hoop tension	$H_u = 1.5 \times HTension6$	(3.198)
Factored hoop stresses	$H_{stress} = H_u/(t_6 \times 1000)$	(3.199)
Factored meridional thrust	$T_u = 1.5 \times T6$	(3.200)
Meridional stresses	$T_{stress} = T_u/(t_6 \times 1000)$	(3.201)
Checks	T_{stress} & $H_{stress} \leq 8N/mm^2$;	(3.202)
Check for nominal reinforcement	Checks are applied as per IS 3370 & IS 456	-
Calculation of spacing for main reinforcement & distribution reinforcement	As per Equation "spacing" are calculated and check for minimum and maximum spacing is checked. (Table 3.15)	-
Cracked width check	Crack widths are checked as per Table 3.16	-
If crack width is found satisfactory the loop with continue for Estimation else Section will be redesigned (Steel is increased), if still found unsafe the thickness is increased and loop will restart from Module I.		
Estimation of Concrete and steel used		
Concrete (A6)	$A6 = 2\pi R_2 t_6 h_2$	(3.203)
Steel estimation Q6	$Q6 = 2R6 \times 0.28 \times s_w/100;$	(3.204)

3.2.3.9. Design of the Main girder beam

This is the Main ring beam of the superstructure. It transfers the whole load to the column and substructure. The diameter of the column must be equal to or greater than the width of the beam. Following equations are used for calculation:

Table 3.31: Step I- Design steps used in the design of Bottom beam

Step I – Calculations of ultimate forces and stresses		
Factored hoop tension	$H_u = 1.5 \times HTension7$	(3.205)
Factored B.M.	$M_u = 1.5 \times M7$	(3.206)
Total load at MRB	$\mathrm{TL}_{MGB} = SW1 + SW2 + SW3 + SW4 + SW5 + SW6 + SW7 + capacity \times 10;$	(3.207)
Total load per span	$\mathrm{TL}_{lps} = \mathrm{TL}_{MGB}/N$ where N is the Nos of columns	(3.208)
The angle at the center in radians	$\propto_{centre} = \pi/N$	(3.209)
Distance of critical Section	$d_{critical} = \frac{2\pi r_2}{N} - \frac{D_c\sqrt{\pi}}{2}$	(3.210)
Angle at the critical Section in radians	$\propto_{critical} = \frac{D_c\sqrt{\pi}}{2 \times 2 \times r_2}$	(3.211)
Bending moment at the critical Section	$BM_{cr} = 2\pi r_2 \mathrm{TL}_{lps}\left(\propto_{centre}\sin(\propto_{critical}) + \propto_{centre}\frac{\cos(\propto_{critical})}{tan(\propto_{centre})} - 1\right)$	(3.212)
Twisting Moment	$M_t = r_2^2\frac{2\pi r_2}{N}\left(+\propto_{centre}\cos(\propto_{critical}) - \propto_{centre}\frac{\sin(\propto_{critical})}{tan(\propto_{centre})} + \propto_{critical} - \propto_{centre}\right)$	(3.213)
BM by Continuity equations	$BM_{continuity} = M7 \times r_2 /1000;$	(3.214)
Equivalent Moment	$M_{EQ} = M_T\frac{1 + d_7/b_7}{1.7};$	(3.215)
Design Moment	M_{DM}=-$(BM_{cr} + BM_{continuity})$+ M_{EQ}	(3.216)
Eccentricity	$e = M_{DM} \times 1000000/Htension7$	(3.217)
Distance of Hoop force	$d_{hf} = e - (0.5 \times d_7 - 40);$	(3.218)
Depth of neutral axis	$la = d_7 - 50 - \varphi_d - 0.42 \times 0.48 \times d_7$; where 50 mm is the clear cover and $d_7\ is\ in\ mm$	(3.219)
Force of Tension	$F_T = Htension7 \times \frac{la + d_{hf}}{la};$	(3.220)
Force of Compression	$F_C = F_T - Htension7$	(3.221)
Reinforcement calculation for hoop tension & moment	A_{stH7}, A_{stM7} from formulas in Equation	-
Check for nominal reinforcement	Checks are applied as per IS 3370 & IS 456	-
Calculation for main reinforcement	As per Equation Numbers of bars for hoop tension and	-
Cracked width check	Crack widths are checked and if found unsafe member is redesigned	-

Table 3.32: Step II-IV- Design steps used in the design of Bottom beam

Step II- Calculation of forces and reinforcement at centre of beam		
BM at midspan	$BM_{mid} = 2\pi r_2 \mathrm{TL}_{lps}(\propto_{centre} /\sin(\propto_{centre}) - 1)$	(3.222)
Design moment	$M_{DM}= (BM_{mid} + BM_{continuity})$	(3.223)
Reinforcement calculations	Further Table 3.15 is used to calculate Reinforcement at the center of midspan	
Step III- Design for shear and twisting moment		
Critical section for shear	$d_{shearcr} = 0.5\ d_{critical} - d_7 - clcover$; Critical Section for shear is at a distance equal to effective depth from face of support	(3.224)
Shear forces	$SF_{MGB} = \mathrm{TL}_{lps} \times d_{shearcr}$	(3.225)
Angle for twisting moment	$\propto_{Twist}= \dfrac{\propto_{centre}\ d_{shearcr}}{r_2};$	(3.226)
Twisting moment	$M_{tcr} = r_2^2 \frac{2\pi r_2}{N}\left(\propto_{Twist} - \propto_{centre} + \propto_{centre} \cos(\propto_{Twist}) - \propto_{centre} \frac{\sin(\propto_{Twist})}{tan(\propto_{centre})}\right)$	(3.227)
Design shear force	$SF_{Design} = SF_{MGB} + 1.6M_{tcr}/b_7$	(3.228)
Maximum shear stress	$\tau_{max} = SF_{Design}/(b_7 \times d_7)$; Table 3.15 is used to calculate shear reinforcement. 12 mm bars are used for shear reinforcement	(3.229)
Step IV-Estimation of concrete and steel used		
Concrete (A7)	$A7 = (2\pi r_2) \times b_7 \times d_7$	(3.230)
Steel estimation at top of beam Q71	$Q71 = 2\pi r_2 \dfrac{\pi(\varphi_m)^2}{4} \times Nos \times \mathrm{sw}$ Where Nos is Calculated for F_T	(3.231)
Steel estimation (Bottom) Q72	$Q72 = 2\pi r_2 \times \dfrac{\pi(\varphi_m)^2}{4} \times Nos \times \mathrm{sw}$ Where Nos is Calculated for F_C	(3.232)
Steel estimation (Shear) Q73	$Q73 = \{(3d_7 - 200) \frac{\pi(\varphi_s)^2}{4} \times \mathrm{sw} \frac{2\pi r_2}{spacing}\}$	(3.233)
Steel total Q4	Q7= Q71+Q72+Q73;	(3.234)

With the design and Estimation of the tank, Module III is completed. Now the further modules will consider the design of staging and foundation and further the cost estimation.

3.2.4. Module IV: Staging configurations and lateral analysis of tanks

This chapter delves into the analysis of staging configurations for water tanks, emphasizing the importance of both structural support and stability under various loading conditions. Staging configurations are essential for maintaining the integrity and functionality of water tanks, ensuring they can safely contain and distribute water. The process begins with determining the total height of the staging, which is a critical parameter affecting the overall stability and load distribution. Several factors are considered in this calculation, including the height from ground level to the top of the tank, the depth of the base slab below ground level, and specific adjustments related to the foundation. Vertical load computations are performed to assess the weight contributions of different structural components. These include the self-weight of the columns, the braces, and additional elements such as stairs. Each component's weight is meticulously calculated to ensure accurate load distribution within the staging. The design of the Staircase is not considered in this study as cost of Stair Case will be same in case of HWT and CWT.

In addition to vertical loads, this module thoroughly analyzes lateral forces, particularly wind and seismic loads.

Wind load analysis involves evaluating the basic wind speed, terrain category, and the height and shape of the tank which is discussed in detail in Section 1.9.2. This analysis helps in determining the wind pressure acting on different parts of the tank and its staging, which is crucial for designing wind-resistant structures.

Seismic analysis is another critical aspect covered in this module. The seismic forces acting on the tank are determined by evaluating the tank's fundamental period in both full and empty conditions. Effective weight calculations and deflection assessments are conducted to understand the tank's behavior during seismic events. This analysis is vital for ensuring that the tank can withstand earthquakes without compromising its structural integrity. The module concludes by comparing the computed wind and seismic forces to identify the most critical load cases. This comparison helps in designing a robust staging configuration that can endure both everyday operational loads and extreme conditions, such as strong winds and seismic activities. Through detailed computations and rigorous analysis, this chapter provides a comprehensive framework for the design and lateral analysis of water tank staging, ensuring safety and stability throughout the tank's lifespan.

The design of staging involves determining the height and structural elements that support the tank. Design of staging is divided in 8 steps.

Table 3.33: Step I-II – Staging configuration design and vertical load calculations

Step I – Staging configuration design and Vertical load calculations		
Total height of staging	$H_{Total} = H_{staging} - d_7 + (D_f) - d_{10}$	(3.235)
Clear length of each panel above FGL	$C.l._{Panel} = (H_{staging} - d_7 - 4 * d_{10})/4;$	(3.236)
c-c distance of column	$c - c\ dis\ of\ column = N \times sin(\pi/N);$	(3.237)
Clear length of brace	$Cl_{brace} = N \times sin\left(\frac{\pi}{N}\right) - D_c$	(3.238)
Step II- Computation of vertical loads		
Self-weight of column	$\mathrm{SW8} = 25\frac{\pi \mathrm{D}_c^2}{4}H_{Total}$	(3.239)
Self-weight of brace	$\mathrm{SW9} = 25n_2\mathrm{b}_9\mathrm{d}_9Cl_{brace}$	(3.240)
Self-weight of stair	SW12=40-50 kN;	(3.241)
Total weight per column upto braces	$\mathrm{SWuptobrace} = \frac{\mathrm{TL}_{MGB}}{2\pi N} + \mathrm{SW8} + \mathrm{SW9} + \mathrm{SW12};$	(3.242)

Table 3.33 details the staging configuration design and vertical load calculations. **Step I** involves calculating the total height of the staging, the clear length of each panel, and the spacing between columns and braces. These parameters define the structural layout of the tank. **Step II** computes the vertical loads by assessing the self-weight of columns, braces, and stairs, along with the total weight per column up to the braces. These calculations are crucial for understanding the load distribution and ensuring the stability of the staging structure.

Table 3.34: Step III – Wind load calculation

Parameter	**Details**
Basic wind speed V_{bs}	Annex A of IS 875 contains the basic wind speeds of major Indian cities, which is one of the input parameters displayed at the user input interface. Users can refer to this table to determine the basic wind speed or input the value based on their own knowledge.
$\boldsymbol{k_1}$	The value of k_1 depends on the basic wind speed V_{bs}. The values are predefined in the wind analysis Section for important structures as follows: For • V_{bs} =33 m/s $-k_1$ =1.05 • V_{bs}=39 m/s$-$ k_1=1.06 • V_{bs}=44 or 47 m/s $-k_1$ =1.07 • V_{bs}=50 or 55 m/s $-k_1$ =1.08 This is done using if else loop
$\boldsymbol{k_2}$	Is calculated as per Section 1.9
$\boldsymbol{k_3}$	Values are inbuilt as per Section 1.9.2 of IS 875 Part 3. Usually, it is taken as 1.
$\boldsymbol{k_4}$	Users will be prompted to specify the region type at the input interface: • Enter '1' for coastal • Enter '1.3' for non-coastal
Calculation of V_z & P_z	• Calculation is done using Formulas mention in Section 1.9.2. • $V_z = V_b \times k_1 \times k_2 \times k_3 \times k_4$ • $P_z = 0.6 \times V_z^2$

Table 3.35: Step IV– Calculation of the wind pressure acting on the tank

Step IV- Calculation of the wind pressure acting on the tank-

Wind load on container	$\mathrm{WPattankbody} = 0.7\, P_z(\frac{2}{3}\mathrm{h}_1 + \mathrm{d}_2 + \mathrm{H} + \mathrm{d}_4) \times 2\mathrm{r}_1 + 0.7\, P_z(\mathrm{r}_1 + \mathrm{r}_2)(\mathrm{r}_1 - \mathrm{r}_2)\tan(\mathrm{A}_r);$	(3.243)
Wind load acting at 'x' m	$\mathrm{htankcg} = 0.5(H + \mathrm{h}_1 + \mathrm{d}_2 + (\mathrm{r}_1 - \mathrm{r}_2)\tan(\mathrm{A}_r))$	(3.244)
Effective number of column	$N_{Eff} = (N \times 0.5 + 1 + \frac{N \times 0.5 - 1}{2});$	(3.245)
Wind pressure at each panel except lower panel	$Wup = 0.7\, P_z \times N_{Eff} \times \mathrm{D}_c \times C.l._{Panel};$	(3.246)
Wind pressure at lower panel	$Wlp = 0.7\, P_z N_{Eff} \times \mathrm{D}_c(C.l._{Panel} - \mathrm{D}_f + \mathrm{d}_{10});$	(3.247)
Wind pressure at brace	$\mathrm{Wbrace} = P_z\, d_9(\mathrm{D}_c + 2\mathrm{r}_2);$	(3.248)
Shear force at panel I	$SF_{AT\ Panel\ 1} = \mathrm{WPattankbody} + Wup \times 0.5;$	(3.249)
Shear force at panel II	$SF_{AT\ Panel\ 2} = SF_{AT\ Panel\ 1} + Wup + \mathrm{Wbrace};$	(3.250)
Shear force at panel III	$SF_{AT\ Panel\ 3} = SF_{AT\ Panel\ 2} + Wup + \mathrm{Wbrace};$	(3.251)
Shear force at panel IV	$SF_{AT\ Panel\ 4} = SF_{AT\ Panel\ 3} + Wup + \mathrm{Wbrace};$	(3.252)
Shear force at panel V	$SF_{AT\ Panel\ 5} = SF_{AT\ Panel\ 4} + (Wup + Wlp) \times 0.5 + \mathrm{Wbrace};$	(3.253)

Table 3.36: Step V- Seismic analysis of tanks

Effective weight for full tank conditions EWT_1	$EWT_1 = N\{Loadperspan + \frac{SW8+SW9+SW12}{3}\}$	(3.254)
Effective weight for empty tank conditions	$EWT_2 = EWT_1 - q_u \times \gamma_w;$	(3.255)
Deflection for full tank condition Def_1	$Def_1 = \frac{(n_2+1)\{(C.l._{Panel}+b_9)^3+D_f{}^3\}EWT_1(1+\cos\frac{2\pi}{N})}{\{12\times5000\sqrt{f_{ck}}\times\frac{\pi\times D_c^2}{4}\times\frac{N}{64}\}}$ All dimensions in mm.	(3.256)
Deflection for empty tank condition Def_2	$Def_2 = Def_1 \frac{EWT_2}{EWT_1}$	(3.257)
Time Period in full tank T_{Full}	$T_{Full} = 2\pi \sqrt{\frac{Def1}{9810}}$	(3.258)
Time Period in empty tank T_{Empty}	$T_{Empty} = 2\pi \sqrt{\frac{Def2}{9810}}$	(3.259)
Damping	5%	-
Zone factor	Seismic Zone needs to be entered in input. As given in Section 3.2.4. is embedded in the code. With help of if else loop Zone factor is calculated by the code.	-
Importance factor	This is taken as 1.5	-
Response reduction factor	The response reduction factor (R) is set at 4 for SMRF buildings	-
Calculation of Sa/g	Soils are classified in Hard, Medium and Soft soils. Sa/g for Full & Empty tank are calculated as per Figure 2 IS 1893. Data's are embedded in the code using if else loop	-
Calculation of design horizontal earthquake acceleration coefficient $A_h$in Full and Empty tank	$A_{HFull} = \frac{Z \times 0.5 \times (S_a/g)_{Full}}{[^R/_I]}$ $A_{HEmpty} = \frac{Z \times 0.5 \times (S_a/g)_{Empty}}{[^R/_I]}$	(3.260) (3.261)
Base shear calculation V_B in full and empty tank	V_{BFull}or $SeismicForceFull = A_{HFull} \times EWT_1;$ $V_{BEmpty} = A_{HEmpty} \times EWT_2$	(3.262)

Table 3.37: Step VI-VIII for the analysis of staging

Step VI- Calculation of maximum lateral force and overturning moments for design of columns		
Check for maximum lateral force	$V_{BFull}\ and\ SF_{AT\ Panel\ 5}$ in Full tank and $V_{BEmpty}\ and\ SF_{AT\ Panel\ 5}$ in Empty tank	(3.263)
Overturning moment Calculation for wind pressure OTM_w	$OverTurningMomentwind = WPatTankbody(htankcg + Hstaging) + Wup\{(n \times Hstaging - 0.5C.l._{Panel} - 1.5C.l._{Panel} - d_9 - 2.5C.l._{Panel} - 2d_9 - 3.5C.l._{Panel} - 3d_9)\} + \mathrm{Wlp}(\mathrm{H}staging - 4.5C.l._{Panel} - 4d_9) + \mathrm{Wbrace}\{(\mathrm{n} \times \mathrm{H}staging - \mathrm{n} \times d_7 - C.l._{Panel}\,(\mathrm{n})(\mathrm{n}+1)/2 - d_9(0.5 + 1.5 + 2.5 + 3.5)\}$	(3.264)
Overturning moment Calculation for seismic forces	$OverTurningMomentSeismic = V_{BFull}(htankcg + Hstaging);$	(3.265)
Maximum thrust on leeward column	$MaxThrust = \frac{4 \times \mathrm{OverTurningMoment}}{(2r_2\mathrm{N})}$	(3.266)
Step VII- Calculation of the final design forces for full tank conditions		
Max lateral force on each column and overturning moment	If (V_{BFull}<=SFat5panel) Overturning Moment= O.M. Wind. Max Lateral Load= SFat5panel; Else if **(**V_{BFull}>SFat5panel) Overturning Moment= O.M. Seismic. Max Lateral Load =V_{BFull};	
Maximum thrust on leeward column	$MaxThrust = \frac{4 \times \mathrm{OverTurningMoment}}{(2r_2\mathrm{N})}$	(3.267)
Axial force on the column (Pu)	$if\ \frac{MaxThrust}{\mathrm{SWuptobrace}} \times 100 < 33.33$ $P_u = \mathrm{SWuptobrace};$ $else\ P_u = \mathrm{SWuptobrace} + MaxThrust;$	(3.268)
Step VIII- Calculation of the final design forces for empty tank conditions		
Max lateral forces and expect overturning moment etc	Parameters are calculated as per above equations for E.T for conditions	

This module covered the analysis of staging configurations for water tanks, including determining the total height of the staging, calculating vertical loads from structural components, and analyzing lateral forces like wind and seismic loads. It detailed the computation of wind pressures, shear forces, and effective weights under full and empty tank conditions. The module concluded by comparing wind and seismic forces to identify critical load cases. The next Section will focus on the design of columns and braces.

3.2.5. Module V-Staging design

This module covers the design process for columns and braces in the staging of a water tank. It begins with the calculation of maximum bending moments in the columns under full load conditions, followed by the determination of the necessary reinforcement based on various load and moment conditions. The design includes calculating axial load and bending moment ratios to determine the required number and size of reinforcement bars for the columns. Additionally, the bracing system is designed by calculating moments, determining reinforcement requirements, and ensuring adequate shear reinforcement. This module provides detailed computational procedures to ensure the structural integrity and safety of the staging system. This program segment automates the design of bracing for a structural framework. It computes essential parameters such as moments, steel reinforcement requirements, and shear forces. It determines the number and placement of reinforcement bars based on calculated moments and ensures compliance with minimum steel standards. Additionally, it computes the spacing and recommends the size of shear reinforcement stirrups to effectively withstand anticipated loads. This process ensures that the bracing design meets structural integrity and safety standards efficiently.

In this module, Column and braces are to be designed. For column design Axial Forces & Lateral forces were calculated in the previous module. In this module at first Bending moment are calculated for columns

Table 3.38: Complete design of columns

Step I- Design of the columns		
Column design- In this module, Column and braces are to be designed. For column design Axial Forces & Lateral forces were calculated in the previous module. In this module at first Bending moment are calculated for columns		
Max B.M. in one column in lowest panel in Full tank	$BM_{MaxinCol} = \frac{Maxlatralload(1+cos(\frac{2\pi}{N}))\times C.l._{Panel}}{2N}$	(3.269)
Calculation of Coefficient "a" and "b" for using sp charts	In this Code sp charts are used and are embedded in the code using if else loop in this Section $a = \frac{1.2 \times P_u \times 1000}{f_{ck}D_c^2 \times 1000000}$ $b = \frac{1.2 \times BM_{MaxinCol} \times 10^6}{f_{ck}D_c^3 \times 10^9}$ p_c is calculated from the sp charts. If the reinforcement exeeds the permissible limit of IS 456 :2000 clauses (i.e. 0.8- 6% and 4% as per clause number 26.5.3.1). **Section needs to redesign. Diameter of the column needs to be increased**	(3.270) (3.271)
Step II- Calculation of reinforcement		
Reinforcement calculation	$A_{stCol} = p_c \times \frac{\pi D_c^2}{400} \times 10^6$ $N_8 = \frac{A_{stCol}}{\frac{\pi \varphi_m^2}{4}}$	(3.272-3.273)
Number of bars N8	Can be even/odd but we want even numbers of bars hence following algorithm is used $N_8 = {}^{N_8}/_{2}\,; N_8 = 2 \times (intN_8 + 1)$ φ_m is kept at a minimum of 12 mm and max 20 mm	(3.274-3.275)
Distribution reinforcement	Dist. reinforcement is calculated as per IS 456:2000	
Step III- Estimation of concrete and steel used in column		
Concrete (A8)	$A8 = \frac{\pi D_c^2}{4} \times N \times H_{Total}$	(3.276)
Steel estimation Main Q81	$Q81 = \frac{\pi(\varphi_m)^2}{4\times 10^6} \times H_{Total} \times N_8 \times sw;$	(3.277)
Steel estimation Distribution	$Q82 = 2\pi sw(D_c - clearcover)\frac{\pi(\varphi_d)^2}{4} \times \frac{H_{Total}}{0.3}$	(3.278)
Steel total Q8	Q8=N×(Q81+Q82);	(3.279)

Table 3.39: Complete design of braces

	Braces design	
Moments in braces	Braces are designed for the moment occurring due to wind forces. This has been already discussed in Section 3.2.4. $BMbrace_1 = \frac{1+cos(2\pi/N)}{N}(SF_{AT\ Panel\ 4} + SF_{AT\ Panel\ 5})0.5(C.l._{Panel} + d_9);$ $BMbrace_2 = \frac{0.5 \times BMbrace_1 \times C.l._{Panel}}{4Cos\frac{\pi}{N}}$ $Mbrace_F = 1.2 \times 0.5 \times (BMbrace_1 + BMbrace_2);$	(3.280-3.281)
	Calculation of reinforcement for braces	
Reinforcement calculation for moment	A_{stM9} are calculated from formulas in Equation. Distribution reinforcement and shear reinforcement are calculated as the formula Main reinforcement is provided on both faces of beam. beam is to be designed as Singly Reinforcement beam. If the Section fails in bearing moments beam Cross Sections are increased	
Check for nominal reinforcement and deflections	Checks are applied as per IS 3370 & IS 456	
Distribution reinforcement	Distribution reinforcement is calculated as per IS 456:2000	
	Estimation of concrete and steel used	
Concrete (A9)	$A9 = 4 \times N \times d_9 \times b_9 \times Cl_{brace};$	(3.282)
Steel estimation Main Q91	$Q91 = 2 \times \frac{\pi(\varphi_m)^2}{4 \times 10^6} \times Nos \times Cl_{brace} \times 4N \times sw;$	(3.283)
Steel estimation Distribution	$Q92 = 4N \times 4(d_9 - cc)\frac{\pi(\varphi_d)^2}{4}\frac{Cl_{brace}}{spacing} \times \mathrm{sw}$	(3.284)
Steel Total Q9	Q9=N×(Q91+Q92);	(3.285)

This module automates the design process for columns and bracing, ensuring structural integrity by calculating optimal reinforcement and dimensions based on load conditions. It provides precise specifications for column diameters, reinforcement bars, and bracing configurations, thereby ensuring the structural elements meet safety standards and performance requirements effectively.

3.2.6. Module VI-Foundation design

This module covers the design process for Annular raft footing of a water tank. design has already been covered in Section 1.10. Module VI, focused on foundation design, is dedicated to the meticulous planning and calculation of structural elements crucial for supporting the building's integrity. This Section emphasizes the critical role of foundation beams and raft foundations in distributing loads effectively and ensuring stability. By analyzing various parameters such as load distribution, overturning moments, and soil pressure, the module aims to design foundation elements that can withstand both static and dynamic forces. Through comprehensive design iterations and considerations, the objective is to optimize dimensions, reinforcement layouts, and material specifications to ensure the foundation's resilience and longevity. This approach ensures that the structural integrity and safety standards are met, providing a stable base for the entire structure. Analysis and design of foundation is divided in 8 steps and are shown in Tabular form given below:

Table 3.40: Step I-II of Analysis and design of foundation

Step I- Forces calculation for design of foundation beam	
Max negative B.M. calculations at the support of Fb	$MaxBM = \frac{(SWuptobrace+MaxThrust)\times M_{DM})}{(TL_{MGB}/N)}$ (3.286)
Check for the depth of beam (d_{eff10})	$d_{eff10} = \sqrt{\frac{MaxBMatSuppofFB)\times 10^6}{f_{ck}\times b_9\times 1000}}$ (3.287) If d_{eff10} found safe then loop will proceed further else will increase the depth of foundation beam. After d_{eff10} found, safe loop will proceed to reinforcement
Step II- Calculation of reinforcement	
Reinforcement calculation for moment, distribution and shear	A_{stM10} are calculated from formulas in Equation distribution and shear reinforcement are calculated as the formula Table 3.15. Main reinforcement is provided on both faces of beam. Beam is to be designed as singly reinforcement beam. If the Section fails in bearing moments beam Cross Sections are increased
Check for nominal reinforcement	Checks are applied as per IS 3370 & IS 456

Table 3.41: Step III-IV of Analysis and design of foundation

Step III- Estimation of concrete and steel used		
Concrete	$A10 = 2\pi r_2 d_{10} b_{10};$	(3.288)
Steel Main	$Q101 = 2\pi r_2 \times \frac{\pi(\varphi_m)^2}{4\times10^6} \times Nos \times sw;$	(3.289)
Steel Distribution	$Q102 = 2(d_{10} + b_{10}) \frac{2\pi r_2}{spacing} \frac{\pi(\varphi_d)^2}{4\times10^6} sw;$	(3.290)
Steel total	$Q10 = Q101 + Q102;$	(3.291)
Step IV: Basic dimensioning of Raft foundation		
Total load from structure	$TotalLoad = SWuptobrace/N;.$	(3.292)
Foundation load	Load is assumed to be 10 % of Total load from structure	
Total load	$TLwithFoundationLoad = 1.1 \times TotalLoadFromAbove;$	(3.293)
Raft area	$RaftArea = \frac{TLwithFoundationLoad}{Bc}$	(3.294)
Width of the Raft	$b_{raft} = \frac{RaftArea}{2\pi r_2}$	(3.295)
Inner diameter raft	$D_i = 2r_2 - Widthofraft + 0.1;$	(3.296)
Inner radius of raft	$R_i = 0.5 \times D_i$	(3.297)
Outer diameter raft	$D_o = 2r_2 + Widthofraft;$	(3.298)
Outer radius raft	$R_o = 0.5 \times D_o$	(3.299)
CCR calculation	It is important to distribute pressure uniformly, CCR is assumed to be r_2. Then CCR is calculated using do while loop with using R_o & R_i decreasing $R_i$to 0.001 at every iteration. Once the CCR =r_2 Loop terminates giving the final values of R_o & R_i	
Area of raft provided	$A_{raftpro} = \pi(R_0^2 - R_i^2)$	(3.300)
Check for area of raft	$if\ A_{raftpro}$>=$A_{raftreq}$, then loop will proceed forward else will have to redesign the area of raft.	-

Table 3.42: Step V of analysis and design of foundation

Step V: Calculations of soil pressures		
Upward pressure when tank is full. MaxUP	$\text{MaxUP}_{FT} = \frac{\text{TLwithFoundationLoad}}{A_{raftpro}}$	(3.301)
Over turning moment	Overturning Moment from seismic and wind forces are calculated using the similar formulas as used in Module V.	
Max pressure on soil due to lateral forces at outer egde	$MaxP_o = \frac{OverTurningMoment \times Ro;}{(\pi(R_0^4 - R_i^4) \times 0.25)}$	(3.302)
Max downward pressure, when tank is full (MaxDP)	$(\text{MaxDP})_{FT} = MaxP_o + \text{MaxUP};$	(3.303)
Upward pressure when tank is empty MaxUP_{ET}	$\text{MaxUP}_{ET} = \frac{\text{Total load} - q_u \times 10}{A_{raftpro}}$	(3.304)
Max downward pressure at empty tank MaxDP_{ET}	$\text{MaxDP}_{ET} = \text{MaxUP}_{ET} + MaxP_o$	(3.305)
Check	• $MaxDownPresFullTank <= 1.25Bc$ • $MaxDownPresEmptyTank < Bc$	
If the equations don't satisfy the Section needs to be redesigned		
Min downward pressure, when tank is empty	$\text{MaxDP}_{ET} = \text{MaxUP}_{ET} - MaxP_o$	(3.306)
Maximum upthrust at outer edge, $MaxUPth$	$MaxUPth = \text{MaxUP}_{FT} + MaxP_o$	(3.307)
Radius at inner face of beam	$RadatinnerfaceofBeam = CCR - 0.5d_{10};$	(3.308)
Radius at outer face of beam	$RadatouterfaceofBeam = CCR + 0.5d_{10};$	(3.309)
B.M. at the inner face of beam, BM_i	$BM = \frac{MaxUPth(2R_i + R_{iFB})(R_{iFB} - R_i)^2}{6 \times R_{iFB}}$	(3.310)
B.M. at the outer face of beam, BM_o	$BM = \frac{MaxUPth(2R_i + R_{0FB})(R_{oFB} - R_i)^2}{6 \times R_{oFB}}$	(3.311)
Max of BM_i & BM_0	Maximum BM is checked and is called as $BM_{MaxatRAft}$	

Table 3.43: Step VI-VIII of analysis and design of foundation

Step VI: Calculation of depth of raft at the face of the beam		
Effective depth of raft	$d_{effRaft} = \sqrt{\frac{(BM_{MaxatRAft} \times 10^6}{0.138 \times f_{ck} \times 10^3}}$;	(3.312-3.313)
Depth of raft d_{11}	$d_{11} = \frac{d_{effRaft}}{1000} + 0.07 + 0.1;$ Where 70 mm is cover and 0.1 is $\frac{\emptyset_m}{2}$ Provide Overall Depth "d_{11}" at face of beam & reducing to 0.15 m at free edge	
Step VII: Calculation of reinforcement		
Reinforcement calculation for moment	A_{stM11} are calculated from formulas in Equation Distribution reinforcement and shear reinforcement are calculated as the formula Table 13.5	
Check for nominal reinforcement	Checks are applied as per IS 3370 & IS 456	
Distribution reinforcement	Distribution reinforcement is calculated as per IS 456:2000	
Step VIII: Estimation of concrete and steel used in raft		
Concrete (A11)	$A11 = \{\pi(R_o^2 - R_i^2)(d_{11} + 0.15) \times 0.5\}$	(3.314)
Steel estimation Main Q111	$Q111 = \pi \times sw(R_o - R_i)\frac{\pi(\varphi_m)^2}{4 \times 10^6} \times Nos;$	(3.315)
Steel estimation Distribution Q111	$Q112 = 2\pi(R_o - R_i)4N\frac{\pi(\varphi_d)^2}{4}\frac{1000}{spacing}\text{sw}$	(3.316)
Steel Total Q11	Q11= $Q111 + Q112;$	(3.317)

Module VI provides a detailed guide for designing the annular raft footing of a water tank, ensuring structural integrity and stability. By thoroughly analyzing load distribution, bending moments, soil pressures, and reinforcement needs, the design process adheres to IS 3370 and IS 456 standards. This iterative approach optimizes dimensions, reinforcement layouts, and material specifications to create a durable and resilient foundation. The focus on uniform pressure distribution and effective load management ensures a stable base for the water tank, contributing to the structure's long-term safety and reliability.

3.2.7. Module VII-Estimation and costing

Module VII delves into the crucial phase of evaluating various quantities and performing cost estimations for the foundation design of the water tank. This module systematically calculates the volumes of concrete and steel required, assesses the shuttering area, and aggregates these values to determine the overall material requirements. Additionally, it provides a detailed cost analysis by calculating the expenses associated with concrete, steel, and shuttering. This thorough evaluation ensures precise budgeting and resource management, laying the groundwork for an efficient and cost-effective construction process.

Table 3.44: Summation of quantities and cost calculations

Summation of quantities and cost calculations		
Summation of the quantities	Total concrete used: A=A1+A2+A3+A4+A5+A6+A7+A8+A9 +A10+A11 Total steel used: Q=Q1+Q2+Q3+Q4+Q5+Q6+Q7+Q8+Q9 +Q10+Q11	(3.318-3.319)
Cost calculation	Rate is taken from CPWD DAR 2021 Rate per cum of M 30 concrete used =Rs10447.7 (clause 5.33.2.2) Rate per kg of Steel used =Rs 89.65. Rate per ton= 89650 Rs/ton (clause 5.22.4)	
Summation of the cost	Cost of Total Structure= $10447.7 \times A + 89650 \times Q$	(3.320)
For analysis purposes	Cost of tank elements, Staging, Foundation are also calculated separately	-

Module VII on Estimation and costing for the foundation design of water tanks is crucial for accurately assessing material requirements and budgeting effectively. It systematically calculates volumes of concrete and steel and aggregates these quantities to determine overall material requirements. The module utilizes rates from CPWD DAR 2021 to compute costs for concrete and steel, ensuring precise cost analysis. By breaking down costs for tank elements, staging, and foundation separately, it facilitates detailed budget allocation, optimizing resource management for a streamlined construction process.

In this way, Software program is developed for the design Complete analysis, design and Estimation of Intze water tanks.

3.2.8. Application of software program for CWT I and CWT II Intze tanks

Two different conventional Intze software programs are further developed as per the clause 4.4.3.1. and 4.4.3.3. of IS 3370:2021. Major changes in both the programs are shown in table below:

Table 3.45: Major differences in software programs of CWT I and CWT II

Sr.No	Type	Key Difference
1	Conventional type I Intze water tank	• Ultimate steel stresses are kept as 130 N/mm^2 for the elements of the tank body only. steel stresses are kept as ultimate stresses for staging and foundation • Crack width calculations are not mandatory but are performed for study purposes only
2	Conventional type II Intze water tank	• Ultimate steel stresses are kept as 435 N/mm^2 for the elements of the tank body only. Steel stresses are kept as ultimate stresses for staging and foundation • Crack width calculations are mandatory.

For coding E. Balagurusamy (2017), “Computing fundamentals and C programming” is referred.

3.3. Hybrid design – Detailed concept note on design of hybrid Intze tanks and Software development

The key difference between CWT and HWTs lies in the use of IS 456:2000 (Reaffirmed 2021) for the RCC design of the tank body (the water-carrying container) and use of ferrocement lining as an Integral part of tank body. IS 456: 2000 is a general code for all RCC structures. The minimum dimensions prescribed by IS 3370:2021 are larger compared to IS 456:2000. This difference is due to IS 3370:2021 being specifically for aqueous retaining structures, where larger member sizes are used to control crack width and reduce leakage. The stringent criteria for minimum dimensions, stresses, and exposure in IS 3370:2021 result in material underutilization and significant material wastage, leading to increased construction costs.

In this study, IS 456:2000 is used for designing the RCC tank body to carry Structural Loads in HWTs while maintaining crack width limitations as per IS 456:2000 (which are the same as those in IS 3370:2021 for water-retaining structures, i.e., 0.2 mm). Additionally, ferrocement lining is applied to further reduce crack width, ensuring no leakage from the tank body. According to ACI standards, the maximum permissible crack width for water-retaining structures with ferrocement is 0.05 mm.

Thus, HWTs can bear all loads through the RCC, and the ferrocement lining makes the tank almost impermeable.

This chapter deals with

1. Design of RCC water tank elements (Container) for HWTs,
2. Design of the ferrocement lining,
3. Design of Staging and foundation
4. Software Development for HWTs.

At first this chapter will deal with the design concept of HWT and then then Software Development process.

3.3.1. Design of RCC water tank elements (container) for hybrid tanks

The design of the RCC tanks is done using continuity analysis method and limit state design. In HWTs, RCC is supposed to bear all the structural loads, designed in accordance to IS 456:2000 (Reaffirmed in 2021). The analysis and design are done as per Conventional tank only but the minimum exposure criteria, minimum dimension and minimum stress criteria is followed as per IS 456:2000.

Table 3.46: Minimum cross section and thicknesses of members for HWTs

Sr.No.	Member	Minimum dimension
1	Top dome thickness t_1	0.08 m
2	Top ring beam (b_2 x d_2)	0.2 x 0.2 m
3	Cylindrical wall at top and bottom (t_{3top} & $t_{3bottom}$)	0.1 m at top and 0.1 at bottom initially.
4	Middle ring beam (b_4 x d_4)	1.0 x 0.12 m
5	Conical dome at top and at bottom ((t_{5top} & $t_{5bottom}$)	1.5 x t_{3top} at top and 0.15 m at bottom
6	Bottom dome	0.12 m
7	Main girder beam (b_7 x d_7)	0.4 x 0.5 m
8	Column ($D_{c)}$	0.4 m
9	Braces (b_9 x d_9)	0.2 x 0.5 m
10	Foundation beam (b_{10} x d_{10})	0.4 x 0.5 m
11	Raft area	As per calculations

Table 3.47: Material stress criteria for HWTs

Sr.No.	Material	Stresses	Eq. No.
1	Tensile strength of concrete	$0.7\sqrt{f_{ck}}$	(3.321)
2	Steel stresses	Ultimate stresses	-

3.3.2. Design of ferrocement lining

For the first time, ferrocement is being utilized as a lining material for water tanks, integrated into the tank's design and construction from the outset. This innovative use of ferrocement aims to leverage its unique properties to improve the durability, impermeability, and overall performance of water storage systems. It is important to note that the ferrocement lining-

- Functions like reinforced plaster.
- Serves as a non-structural component of the tank.
- Impervious medium between water and RCC.
- Depends on the RCC outer lining to carry all structural loads.

Purpose of providing ferrocement as liner- The main purpose of providing Ferrocement lining is to impart impermeability to the structure or water proofing of the tanks. It will act as an impermeable membrane between RCC and water. Ferrocement has lesser crack width of the order 0.0022 mm which will not permit any kind of leakage from the structure. Ferrocement lining is to be designed for strain compatibility criteria at the interface of RCC & ferrocement lining. Ferrocement lining will act like reinforced plaster. The primary purpose of the lining is to provide impermeability to the structure design has been done as per the text available and mentioned in the references using American concrete institute standards. General requirements are mentioned in section 1.14. Maximum allowable strain limits for Concrete as well as Ferrocement is 0.0035. design of the water retaining structure mandates the maximum crack width to be less than 0.2 mm for RCC. Hence the allowable strain carrying capacity has been regulated to 0.001583. As lining is a non-structural member, Hence it is assumed that lining is designed for imparting impermeability only. The thickness of the lining is designed for strain compatibility and reinforcement is designed to keep crack width is well within the limits. As lining is to be designed for no loading criteria, hence bending theory is applied to calculate the thickness of lining and further the reinforcement is calculated to balance the strain at the interface and water facing face of the lining and then the crack width is checked to assure impermeability.

Design mythology of ferrocement lining consists of following parts-

1. Strain Calculation in RCC tank body,
2. Determination of thickness of lining and mesh reinforcement
3. Check for the Crack width.

3.3.2.1. Strain calculation in RCC tank body

Crack width calculations are used in accordance to Annextures A & B of IS 3370:2021. The equations are explained in Section 1.8.2 of this study.

Table 3.48: Strain calculation in RCC tank body

Crack width calculations for direct tension and maximum allowable strain in RCC		
Strain at level of Stress e_1	$e_1 = F_S/20000$	(3.322)
stiffening effect of concrete in direct tension	$e_2 = \frac{2(b\times d)}{3\times A_{st}\times 200000}$	(3.323)
e_m	$e_m = e_1 - e_2$	(3.324)
Crack width	$w_{cr} = 3a_{cr}\varepsilon_m$ and $w_{cr} < 0.2\ mm$	(3.325)
Max permissible value of w_{cr}	0.19 mm	
Strain for maximum crack width	$\varepsilon_m = \frac{0.19}{3a_{cr}}$; $\varepsilon_{maxallowable} = 0.001583$; Where minimum value of a_{cr} is 40 mm;	(3.326-3.327)
Crack width calculations for flexure		
Strain at level of Stress e_1	$e_1 = \frac{(t-x_{n.a.})\times F_S}{(t-x_{n.a.}-C_{min})\times 200000}$	(3.328)
stiffening effect of concrete in direct tension	$e_2 = \frac{(t-x_{n.a.})^2}{3\times(A_{st})_{pro}\times 200000\times(a'-x_{n.a.})}$	(3.329)
e_m for flexure	$e_m = e_1 - e_2$	(3.330)
Crack width	$w_{cr} = \frac{3\times a_{cr}\times\varepsilon_m}{1+\frac{2(a_{cr}-C_{min})}{t-x_{n.a.}}}$	(3.331)
Strain for maximum crack width	For maximum w_{cr}, $(a_{cr} - C_{min}) = 0$; $\varepsilon_m = \frac{0.19}{3\times a_{cr}}$; $\varepsilon_{maxallowable} = 0.001583$;	(3.332-3.333)
Permissible strain in concrete	0.0035.	-

Crack width are checked for Hoop and bending both for all the water facing elements and are available in Section 1.8.2. Maximum Strain occurring in the water tanks are noted down for Hoop and Bending moments. Strain in the RCC wall or other element will always be in the permissible limits of 0.0035 as the maximum value if strain for the maximum allowable crack width is 0.001583. Hence the lining is to be designed for a maximum strain of 0.001583.

3.3.2.2. Basic of ferrocement lining design and assumptions

The design of ferrocement lining is focused on ensuring strain compatibility at the interface between the RCC (RCC) and the ferrocement lining. The safety of the lining is determined by its strain limits, with the maximum strain at the RCC interface calculated to be 0.001583. Given that the permissible maximum strain for both ferrocement and concrete is 0.0035, the design is within safe limits. Optimizing the lining thickness and the reinforcement mesh is crucial for an effective design, ensuring the most optimized Section. The composite tensile member is assumed to be made from a brittle matrix (primarily cement-based) reinforced with steel. The matrix is considered linear elastic up to failure with no post-failure load resistance. The matrix strain to failure is significantly lower than that of the reinforcement, making the reinforcement's contribution crucial after matrix cracking. The reinforcement is modeled as elastic perfectly plastic, similar to steel with a distinct yield plateau. The model applies to other brittle matrices and reinforcements, such as carbon or glass fibers.

Assumptions:

- The load is applied to the reinforcement and transferred to the matrix, simulating a through matrix crack at each composite end.
- Bond stress at the reinforcement-matrix interface is constant and slip-independent.
- Matrix cracking occurs when matrix stress reaches its tensile strength.
- Composite stress is calculated by dividing the external tensile load by the composite's cross-Sectional area.
- Beyond a certain distance from the free end, strains in the matrix and reinforcement are equal, leading to $\varepsilon m = \varepsilon r = \varepsilon c$

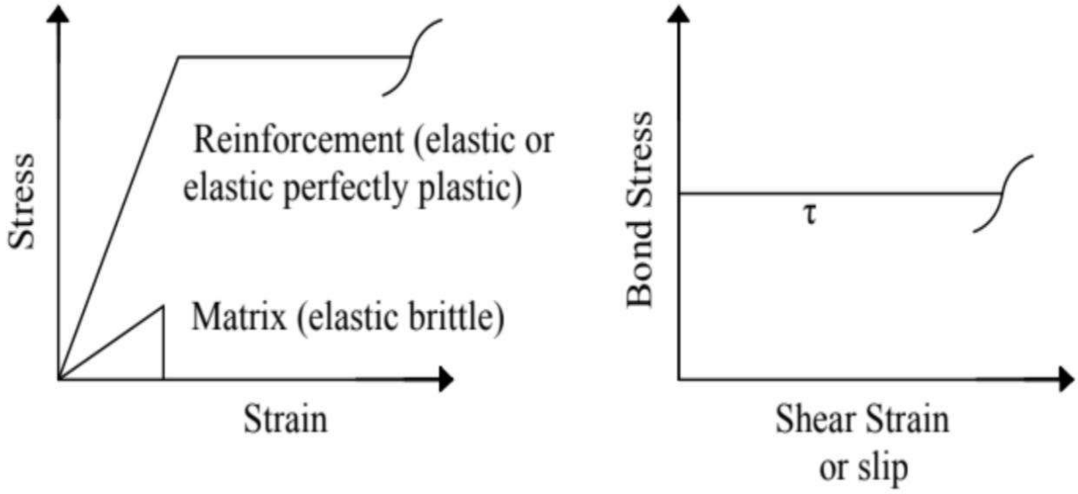

Fig. 3.1: Stress Strain Diagram of ferrocement

3.3.2.3. Basic mechanisms & modeling for uncracked tensile member

Consider an axial load N applied to the reinforcement. For the initial analysis, assume no cracking occurs, and the stresses in the matrix remain below its tensile strength. The equilibrium of forces and stress distribution within the member is analyzed, demonstrating that stress varies linearly along the member length until stabilization occurs where matrix and reinforcement strains equalize.

Modulus of elasticity

$$E_c = E_m V_m + E_r V_{rL} \tag{3.334}$$

Modular ratio

$$n_f = \frac{E_r}{E_m} \tag{3.335}$$

Transformed area

$$A_{tr} = A_m + n \sum A_{ri} \tag{3.336}$$

Stress in the ferrocement Lining

$$f_c = f_m V_m + f_r V_{rL} \tag{3.337}$$

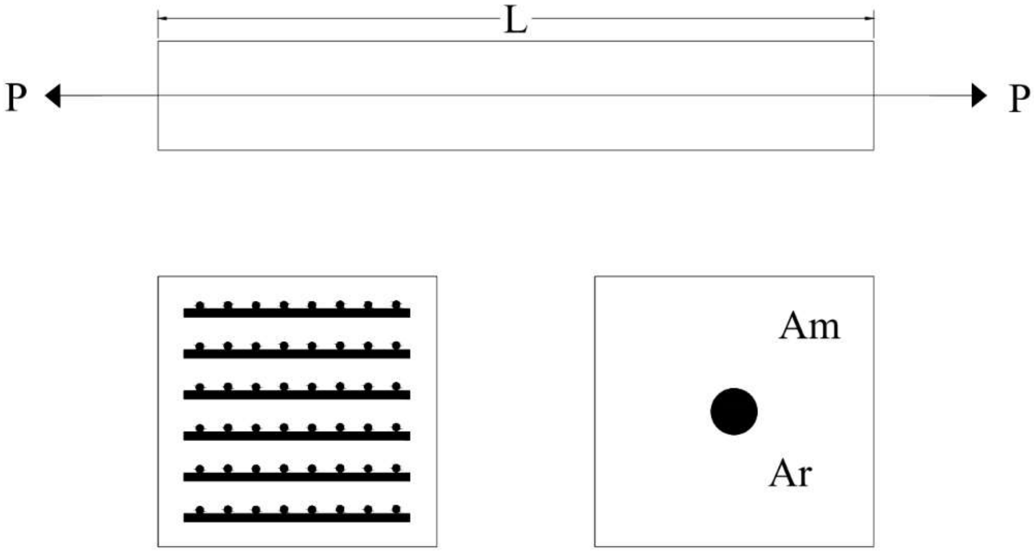

Fig. 3.2: Cross Section of ferrocement beam

3.3.2.4. Crack spacing and width in ferrocement

Crack spacing and crack width are crucial aspects in the analysis and design of ferrocement linings, particularly for water-retaining structures. These parameters help in understanding the behavior of the material under stress and ensuring the integrity and durability of the structure.

- **Minimum crack spacing:** The shortest distance between two consecutive cracks, derived based on the matrix's tensile strength and the bond stress between the matrix and the reinforcement.

  Minimum crack spacing $$\Delta L_{min} = \frac{A_m \sigma_{mu}}{p_f \tau} \quad (3.338)$$

- **Maximum crack spacing**: Twice the minimum crack spacing, representing the longest possible distance between two consecutive cracks.

  Maximum Crack spacing $$\Delta L_{max} = 2\Delta L_{min} \quad (3.339)$$

- **Average crack spacing**: Provides a realistic estimation of the distance between consecutive cracks, considering factors such as the tensile strength of the matrix, the bond stress, and the reinforcement's geometry.

  $$\Delta L_{av} = \lambda_c \frac{A_m \sigma_{mu}}{p\tau} \quad (3.340)$$

- **Average crack width**: A measure of the typical width of cracks in the ferrocement, considering reinforcement, shrinkage strain, and tensile strain.

$$W_{av} = \left(\Delta L_{av}\left\{\frac{N_u}{A_r\,E_r} - \frac{p\tau}{4A_r\,E_r}\Delta L_{av}\right\} + \varepsilon_{SH}\Delta L_{av} - \Delta L_{av}\left\{\frac{p\tau}{4A_r\,E_r}\Delta L_{av}\right\}\right) \quad (3.341)$$

3.3.2.5. Theory of bending in ferrocement linings

The theory of bending in ferrocement linings involves understanding the behavior of this composite material under flexural loads. This includes determining the distribution of stresses and strains across the Section, the location of the neutral axis, and the balance between tensile and compressive forces.

Table 3.49: Theory of bending in ferrocement linings

Theory of bending equations in ferrocement linings		
Calculation of neutral axis	$b_f \times c \times c \times 0.5 + \sum(n-1)A_{ri}\,(c - d_i) = \sum(n) \times A_{rj}\,(d_j - c)$	(3.342)
Compression force in concrete stress block	$C_c = \{0.85 \times f'_c \times b \times \beta_1 \times c\}$	(3.343)
Compression force in concrete stress block due to reinforcement	$C_{ri} = \{(\varepsilon_{ri} \times E_r - 0.85 \times f'_c) \times A_{ri}\}$	(3.344)
Stain at any point on the section	$\varepsilon_{ri} = \frac{d_i - c}{c}\varepsilon_{mu}$ & $\varepsilon_{ri} \leq 450;$	(3.345-3.346)
Tensile forces at each layer of reinforcement	$T_{ri} = A_{ri} \times 450$	(3.347)
Balancing tension & compressive forces in the lining.	$C_c + C_{ri} = \sum T_{ri}$	(3.348)

3.3.2.6. Secondary check for crack width for ferrocement linings

To ensure durability and prevent leakage, especially for water-retaining structures, a secondary check for crack width is performed and are shown in Table 3.30

Table 3.50: Secondary Check for Crack width for ferrocement linings

Crack width analysis for hoop tension for ferrocement lining		
Hoop tension which ferrocement plate can bear	$HT_{fl} = 1000b_f f_{Ferrocement}$	(3.349)
Reinforcement area required	$Ast_f = HT_{fl}/f_{tensile}$	(3.350)
Deciding the number of bars and mesh numbers	$Nos = \ Ast_f/\left(\frac{\pi\emptyset^2}{4}\right)$ $Mesh\ Number\ M_n = Around\ 80\ bars\ per\ mesh$	(3.351)
Maximum spacing	$maximum\ spacing \leq \frac{1000}{t_f}$	(3.352)
Volume fraction Check for the crack width	$V_r = \frac{2A_{ri}}{1000\ t}$	(3.353)
Specific surface area reinforcement	$S_r = 4V_r$	(3.354)
Service stress in reinforcement	$\sigma_r = (\varepsilon_m \times E_s)\ \&\ \sigma_r \leq 450;$	(3.355)
Estimated stress at crack stabilization	$\sigma = 345 \times (1 + S_r)\&$ $\sigma_r\ should\ be\ less\ than\ \sigma$	(3.356)
Crack width check	$W_{max} = {}^{3500}/_{E_r}$ $W_{max}\ should\ be\ less\ than\ 0.05\ for\ water\ retaining\ structures$	(3.357)
Further Steps: $\sigma_r \leq \ \sigma$ loop reduces the number of mesh one and repeat the entire process until optimum is reached.		

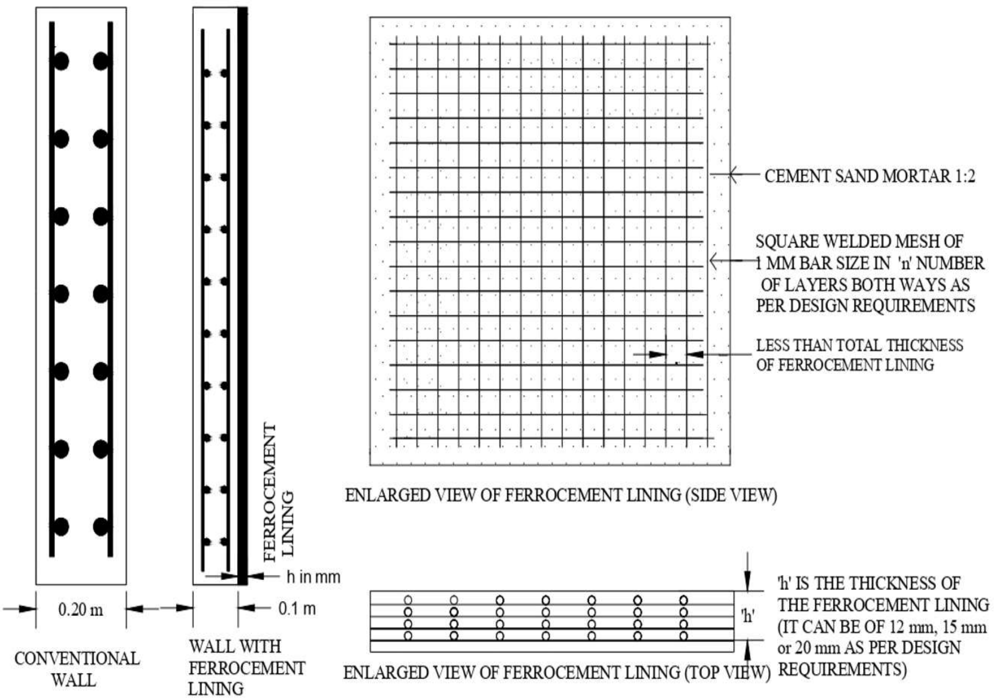

Fig. 3.3: Typical drawing of the cylindrical wall in HWT and CWTs with detailed lining diagram

3.3.3. Design of staging and foundation

The analysis and design process are same as that for Conventional tank as described in the Section 3.2.4- 3.2.6. Although the tank body of the HWT is Light weight as compared to Conventional ones. A lighter tank body will result in lighter staging and foundation. Thus results in the reduction of the cost.

3.3.4. Software development of hybrid tanks

3.3.4.1. Major changes in Hybrid software program

The software development process for HWTs is largely similar to that for CWT, with a few key differences. CWT have seven modules, while HWTs have eight. Below are the major changes and details for each module:

Table 3.51 : Major changes in hybrid vs conventional software program

Modules	**Major differences**
Module I	Minimum exposure criteria, minimum dimension and minimum stress criteria is followed as per IS 456:2000. Initial dimensions are given in Table 4.1.
Module II: Analysis of RCC tank body	Checks and design stresses are taken as per IS 456:2000. Material stresses are provided in Table. The maximum crack width for the RCC is the same in CWT and HWTs. (i.e. 0.2 mm as per IS 456:2000 in HWT and IS 3370:2021 for CWT)
Module III: Design of members of tank	The specific surface area of the entire tank body is calculated. Checks and design stresses are taken as per IS 456:2000. Material stresses are provided in Table. The maximum Crack width for the RCC is the same in CWT and HWTs. (i.e. 0.2 mm as per IS 456:2000 in HWT & IS 3370:2021 for CWT). Reinforcement in wall is given on one face only both ways.
Module IV: Staging configurations and lateral analysis of tanks, Module V-Staging design, Module VI-Foundation design	The modules for staging and foundation design are identical for both HWT and CWT due to adherence to consistent standards and methodologies. Both tank types of follow IS 456:2000 for RCC structures, ensuring structural integrity and durability. Seismic and wind force calculations are performed in both empty and fully tank conditions as per IS 1893:2016 Parts I and II and IITK-GSDMA guidelines for seismic loads, and IS 875: Part 3 for wind loads. Using the Response Spectrum Method, equivalent weight, deflection, response time, and base shear are calculated. Wind calculations consider terrain category and wind speed, as detailed in Table 2. Annular raft footings, preferred for their stability, are designed according to IS 11089:1984. This uniform approach guarantees safety and performance across both tank types.
Module VII-Estimation and costing	This module has been replaced by a new module for HWT design. Module is named as **Module VII-Design of Ferrocement lining**
Module VIII-Estimation and costing of HWTs	Module VIII for Estimation and costing is made which includes the Estimation and costing of ferrocement lining. Rest the Module is same as per Convention Module VII.

3.3.4.2. Details of software module for hybrid Intze tanks

a) **Module I, Module II & Module III-** The changes in Modules 1, 2, and 3 have been detailed in the previous table and discussed comprehensively in Section 3.1. Additionally, Module III for HWTs includes a new feature for calculating the specific surface area of the water tank container. In this module, alongside the conventional calculations for the quantity of concrete and steel after designing each member, the surface area of each member is also calculated. Equations of calculation of surface area are as follows-

Table 3.52: Surface area equations for ferrocement lining calculations

Member	Surface area equations	Eq.No.
Top dome	$SA_1 = \pi R_1 h_1$	(3.358)
Top ring beam	$SA_2 = \pi r_1 t_2$	(3.359)
Cylindrical wall	$SA_3 = 2\pi(2r_1 + t_{3bottom})H$	(3.360)
Middle Ring beam	$SA_4 = \pi r_1 t_4;$	(3.361)
Conical dome	$SA_5 = \pi(r_1 + r_2 + t_{5top})L_c;$	(3.362)
Bottom dome	$SA_6 = \pi R_2 h_2;$	(3.363)
Bottom Ring beam	$SA_7 = \pi r_2(b_7 + d_7);$	(3.364)

Although majorly water contact members are cylindrical wall, conical dome, bottom dome and top dome (ferrocement cover is to be provided allover inside top dome). The rest of the T.R.B, M.R.B and B.R.B area in contact of water is negligible.

b) **Module IV, Module V & Module VI-** These modules as discussed in Table are same as conventional

c) **Module VII – Design of ferrocement lining-** This module is an addition to the Hybrid design and is not present in CWT, as detailed in Section 3.2. The following steps are utilized in this module:

- **Step 1: Fetching strain values-** Start by retrieving strain values from Module III for both hoop and bending. Calculate the maximum strain, typically occurring at the bottom of the cylindrical wall, to ensure uniformity in the lining.

- **Step 2: Strain threshold check-** Verify if the maximum strain exceeds the limit of 0.035.
- **Step 3: Initial lining configuration-** Initially, implement a 12 mm thick lining with 2 mm cover on both sides and three layers of reinforcement. Ensure the spacing between bars is less than the lining thickness, using approximately 85 bars in the first iteration.
- **Step 4: Basic calculations and steel stress check-** Perform fundamental calculations as per the specified Section requirements. Check steel stresses against strain to ensure they remain below 450 N/mm^2, adhering to ACI standards. Calculate crack spacing and width in the ferrocement. If deemed safe, proceed to the bending theory check. If unsafe, increase the lining thickness to 15 mm with six layers of reinforcement.
- **Step 5: Bending analysis-** Analyze the Section for bending per designated criteria. If the tension zone predominates, reduce reinforcement to five bars and re-evaluate from Step 1. If compression dominates, increase reinforcement bars or meshes and repeat Steps 1-5 until achieving a balanced Section.
- **Step 6: Hoop tension and secondary crack width assessment -** Conduct a secondary assessment of crack width under hoop tension. Although the ferrocement lining is not load-bearing, assess its capacity to withstand hoop tension. Adjust reinforcement incrementally, reducing mesh numbers until reaching the proposed configuration from Step 5. Verify crack width at each adjustment. If necessary, increase lining thickness to 15 mm and repeat Steps 1-6. Finalize with a maximum 20 mm lining, using three to six layers of reinforcement as needed.

This systematic approach ensures thorough evaluation and optimization of ferrocement lining design, maintaining compliance with safety standards and structural integrity throughout the iterative process.

Generally, as previously discussed, the strain values for ferrocement are initially calculated manually and then verified using software. The thickness of the lining depends on the strain levels are given. These thickness and reinforcement configurations are initially determined through manual calculations and then entered into the software. These Calculations are given in **Annexure A**. The software subsequently conducts its own calculations to verify and validate this data. These guidelines ensure that the ferrocement lining meets the necessary strength and durability requirements based on the calculated strain levels.

d) **Module VIII – Estimation and costing-** This Module is same as conventional Module No VII with addition of the calculation of cost for ferrocement lining. Ferrocement lining cost for cement mortar and steel bars. The cost estimation and rate analysis for ferrocement lining were carried out by manually analyzing rates as per CPWD DAR 2021. Since rates for 1:2 cement mortar was not available, a rate analysis was done based on available 1:3 mortar rates, adjusting material costs while keeping labor costs the same. Conversion factors were applied for different plaster thicknesses (10 mm, 12 mm, 15 mm, and 20 mm) to arrive at final rates per square meter. Additionally, the cost of HB steel wires, recommended for ferrocement, was taken from market rates (Rs 35–50 per kg) and verified with local centers.

Table 3.53: Estimation of quantities and cost estimations of ferrocement lining

Estimation of quantities		
Total quantity of cement mortar	SA= $SA1 + SA2 + SA3 + SA4 + SA5 + SA6 + SA7;$	(3.365)
Total quantity of steel	$Q_f = Number\ of\ Mesh \times Number\ of\ Bars \times (\frac{\pi\varphi_f^2}{4}) \times SA$ where $\varphi_f is\ 0.001m\ (1\ mm)$	(3.366)

Table 3.54: Cost estimations and rate analysis of ferrocement lining

Cost estimations and rate analysis	
Rates calculation for plastering	1:2 cement mortar is used with OPC 43 grade or PPC. OPC 43 grade is recommended. 1: 2 cement sand mortar rate was not available in CPWD DAR 2021. Hence rate analysis is done to calculate rates of 12 mm, 15 mm and 20 mm using cement sand mortar
Mortar rate	1:2 cm mortar rate: Rs. 5613.46 per cum (clause 3.7 CPWD DAR 2021)
Rate analysis steps	Rate analysis is performed manually to determine the costs for 10 mm, 12 mm, 15 mm, and 20 mm Cement Sand Plaster. While the rate for 1:3 Cement sand mortar is provided, the rate for 1:2 Cement Stone Dust is also given. Consequently, rate analysis is conducted according to DAR 2021 standards. The 1:3 Cement sand mortar is replaced with 1:2 Cement Mortar. For item number 1.3.9.1, the labor and other costs remain unchanged, with only the material rate being updated. The conversion factors from cubic meters to square meters for the different plaster thicknesses are as follows: • For 10 mm plaster, the conversion factor is 0.12. • For 12 mm plaster, the conversion factor is 0.144. • For 15 mm plaster, the conversion factor is 0.172. • For 20 mm plaster, the conversion factor is 0.244. These conversion factors are used to calculate the required materials and associated costs for different plaster thicknesses.
Final rates of cement mortar	10 mm thick plaster- Rs 233.9 per sqm; 12 mm thick plaster- Rs 249.4 per sqm; 15 mm thick plaster- Rs 281.5 per sqm; 20 mm thick plaster- Rs 319.0 per sqm
Final rates of steel	HB bars are recommended to be used. Rate of HB wires is not given in DAR 2021 CPWD. However Rs 35-50 per Kg is the Market Rate cross and is verified by TMM Building Centre and Ferrocement Factory Tilothu Rohtas Bihar

In conclusion, the design and development of HWTs, which integrate RCC tank bodies with ferrocement linings, represent a significant advancement in water storage solutions. By utilizing RCC for structural loads and ferrocement for impermeability, these HWTs achieve superior performance, reduced leakage, and cost-effectiveness compared to conventional RCC tanks. The detailed design methodology encompasses four primary steps: designing the RCC water tank elements, ferrocement lining, staging, and foundation, all while adhering to relevant IS codes and standards. The introduction of specialized software for HWT design further enhances accuracy and efficiency, ensuring optimal material use and structural integrity. Ultimately, HWTs offer a robust and reliable solution for water storage, meeting the critical demands of impermeability and structural performance.

3.4. Development of the software for conventional circular water tanks

The fundamentals of conventional circular RCC water tanks were introduced in Chapter I, Section 1.8 of this study. This chapter focuses on the development of software tailored for the design and analysis of these tanks. Similar to the software program developed for conventional RCC Intze tanks, the software for Conventional circular tanks is composed of seven modules.

3.4.1. Software modules for the conventional circular water tanks

The software for conventional circular water tanks includes seven modules, mirroring the structure of the software for conventional Intze tanks. The forces, loadings, analysis methods, and design elements are consistent between circular tanks and Intze tanks. However, key distinctions between the two types of tanks impact their structural design and analysis.

Key distinctions between Intze and circular water tanks-

- The primary differences between Intze water tanks and circular water tanks lie in their structural components and how these components are interconnected. Intze water tanks feature seven main elements: the Top dome, Top ring beam , Cylindrical wall, Middle Ring beam, Conical dome, Bottom dome, and Bottom Ring beam. These elements are interconnected by three joints. In contrast, circular water tanks consist of six elements (Top dome, Top ring beam , Cylindrical wall, Middle Ring beam, Bottom dome, and Bottom Ring beam) and only two joints (Top and Bottom). This design includes all the components found in Intze tanks except for the Conical dome, simplifying the design and analysis processes for circular tanks.
- As a result of these differences, the radius of the Bottom ring beam in circular water tanks is the same as that of the Top ring beam . In Intze tanks, however, the radius of the Bottom ring beam is typically about 0.75 times that of the Top ring beam .

- This distinction requires circular water tanks to have more columns and a larger raft area compared to Intze water tanks.
- Due to these structural requirements, the cost of staging and foundation is higher for circular tanks compared to Intze tanks.

Modules are discussed in detail as below-

3.4.2. Module I: Input and dimensioning of tanks

The initial step involves determining the tank dimensions, for which the primary input is the desired capacity of the tank. This is essential as it drives the overall design process. Additional crucial inputs include the column height, which defines the vertical dimensions of the tank, and the bearing capacity of the soil, which assesses the soil's ability to support the tank. The depth of the foundation is also important to ensure stability. The seismic zone of the location, which affects the tank's design to withstand earthquakes, along with the wind velocity and terrain topography, are vital factors that influence the structural integrity and suitability of the tank.

3.4.2.1. Input required

Input required are as follows-

Table 3.55: Input required for design of circular tanks

Sr.No	Input required
1	Capacity of tanks in kL
2.	Staging height
3.	Bearing capacity of soil and foundation depth (As per soil report)
4.	Seismic zone
5.	Wind speed and terrain type along region type coastal /noncoastal
6	H/D ratio (Software is inbuilt with optimal values for 100-1000 kL capacity tanks, but user can input as per own)
7	Optimal number of columns (Software is inbuilt with optimal values for 100-1000 kL capacity tanks, but user can input as per own)
8	The characteristic strength of concrete for tank body elements (f_{cktank}) and for Staging and foundation ($f_{ckstaging}$) is kept as 30 N/mm^2. User can change these parameters.
9	The ultimate strength of Steel for tank body elements (f_{ytank}) and for Staging and foundation (f_y) is kept as 500 N/mm^2. User can change these parameters. For CWT I f_{ytank} is kept as 130 N/mm^2 and for CWT II f_{ytank} is kept as 435 N/mm^2 (0.87 times f_y)

3.4.2.2. Calculations of member dimensions for the desired capacity of tanks

Capacity is the basic of data required to design the tank. It is required to calculate the dimensions of the tanks from basic mensuration formula. In this module, various mensuration formulas are applied to calculate the dimensions and other essential parameters of the tank. Here is a detailed description of the formulas used in Table 3.2. Loop will terminate when the required capacity is achieved. And the dimensions are finalized.

Table 3.56: Calculation of the dimensions for conventional circular water tanks

Dimension	Formula or starting value	Eq. No.
Radius of cylindrical wall	1 m (At the start of loop with increment of 0.1m)	-
H/D ratio: c_1	User input	-
Height of the top dome	$h_1 = (2r_1 + 0.2)/7;$	(3.367)
Radius of bottom girder beam wall	$r_2 = r_1;$	-
Height of bottom dome	$h_2 = (2r_1 + 0.2)/5;$	(3.368)
Radius of the top dome	$R_1 = \{(r_1 + 0.1)^2 + {h_1}^2\}/h_1;$	(3.369)
Radius of bottom dome	$R_2 = \{(r_1 + 0.1)^2 + {h_2}^2\}/h_2;$	(3.370)
Volume calculation= Volume of cylinder - Volume of bottom dome	$\pi {r_1}^2 H - \pi h_2^2 \frac{3R_2 - h_2}{3};$	(3.371)
Free board	0.15 m	-
Height of cylindrical portion	$H = 2r_1c_1 + 0.15;$	(3.372)

Once the specified capacity is reached, the iterative loop terminates, and the final dimensions are calculated based on the last iteration. These final dimensions incorporate the optimized parameters determined throughout the iterative process, ensuring that the tank design meets the desired capacity requirement.

3.4.2.3. Member properties for water tank design-

In this step, minimum cross-Sectional properties are assigned to all structural elements of the water tank, such as the top dome, top ring beam, cylindrical walls at the top and bottom, balcony beam, bottom dome, main girder beam, column diameter, bracing size, and foundation beam size, in accordance with IS 3370:2021. Additionally, concrete and steel grades are specified to meet the required standards. Basic dimensions are given below in Table.

Table 3.57: Minimum dimensions/ Thicknesses of the conventional circular tanks

Member	Minimum dimension
Top dome thickness t_1	0.12 m
Top ring beam (b_2 x d_2)	0.2 x 0.2 m
Cylindrical wall at top and bottom (t_{3top} & $t_{3bottom}$)	0.2 m at top and 0.2 at bottom initially.
Middle ring beam (b_4 x d_4)	1.0 x 0.12 m
Bottom dome t_6	0.12 m
Main girder beam (b_7 x d_7)	0.4 x 0.5 m
Column ($D_{c)}$	0.4 m
Braces (b_9 x d_9)	0.2 x 0.5 m
Foundation beam (b_{10} x d_{10})	0.4 x 0.5 m
Raft area	As per calculations

Nomenclature given to the dimensions of different elements of tanks are kept same as that of Intze tank for reducing the complexity of nomenclature. Here in this case all the parameters regarding Conical dome are kept as zero.

3.4.3. Module II: Analysis of RCC tank body

Steps involved are as follows-

3.4.3.1. Determination of stiffnesses and member forces

- **Top dome, Top ring beam , cylindrical wall, balcony/middle ring beam and bottom dome analysis-** Similar as that of Conventional Intze tanks.
- **Balcony/ middle ring beam-** Force analysis is different in case of Balcony beam rest the Stiffness calculations; Calculation of displacement & Rotations are similar as that of Intze tanks.

Table 3.58: Force calculation for the middle ring beam

Force calculations		
Forces	Formulas	Eq.No.
Self-weight (kN/m^2)	$w_6 = 25 \times t_6$	3.373
Total weight (kN)	$SW6 = w_6 \times 2\pi h_2 R_2$	3.374
Water load at BD WL_{BD}	$WL_{BD} = \gamma_w\{\pi r_2{}^2(H) - \pi h_2^2 \frac{3R_2-h_2}{3}\}$	3.375
Total load at bottom dome per sqm	$\mathrm{TL}_{BD} = \frac{SW6 + WL_{BD}}{2\pi R_2 h_2}$	3.376
Horizontal thrust at bottom support	$HForce_{BD} = \frac{\{SW6 + WL_{BD}\}R_2 cos\theta_2}{(1 + cos\theta_2)\{2\pi h_2 R_2\}}$	3.377

- **Main ring beam or Main girder beam-**

Table 3.59: Force calculation for the main ring beam

Forces calculations		
Forces	Formulas	Eq. No.
Self-weight $SW7$	$SW7 = 25 b_7 d_7 w_7 \times 2\pi r_1$	(3.378)
Total load at Main girder beam TL_{MGB}	$\mathrm{TL}_{MGB} = SW1 + SW2 + SW3 + SW4 + SW6 + SW7 + 10qu;$	(3.379)
Total load /m	$Total\ Load\ per\ meter = \frac{\mathrm{TL}_{MGB}}{2\pi r_1}$	(3.380)
Net Horizontal force at Bottom support	$\mathrm{H_{NET}} = -\ HForce_{BD}$	(3.381)

Equations provided in Table 1.4 are used to solve for primary unknows using Stiffness Matrices. For all the Two Joints Direct stiffness matrices are written. The direct stiffness matrix, often referred to simply as the stiffness matrix, is a fundamental concept in structural mechanics and finite element analysis. It relates the forces applied to a structure to the displacements at specific points within that structure. The stiffness matrix is typically denoted by {K} and forms the basis of the stiffness method, which is a common approach to solving structural analysis problems.

3.4.3.2. Defining stiffness matrix for circular tanks

Here in case of circular tanks there are only two joints with top joint having three members and bottom joints having four members.

a) **Joint I- Top Joint-** Global stiffness matrix Constitutes of stiffnesses of members joining at joint I that are top dome, top ring beam and wall.

- Global stiffness matrix at Joint I

$$\{K_1\}_{2\times 2}=\begin{bmatrix} MS1+MS2+MS3t & \mathrm{CorrTS1}+\mathrm{CorrTS3t} \\ \mathrm{CorrMS1}+\mathrm{CorrMS3} & TS1+TS2+TS3t \end{bmatrix} \tag{3.382}$$

$$\{K_1\}_{2\times 2}=$$

$$\begin{bmatrix} \frac{R_1Et_1(k_{td}+k_{td}^{-1})}{4\times\lambda_{td}{}^3}+\frac{Eb_1d_1{}^3}{12r_1{}^2}+2\mu Z_{top} & \frac{Et_1}{2\lambda_{td}{}^2k_{td}sin\theta_{td}}+2\mu^2Z_{top} \\ \frac{Et_1}{2\lambda_{td}{}^2k_{td}sin\theta_{td}}+2\mu^2Z_{top} & \frac{Et_1}{\lambda_{td}R_1k_{td}sin^2\theta_{td}}+\frac{Eb_1d_1}{r_1{}^2}+4\mu^3Z_{top} \end{bmatrix} \tag{3.383}$$

- Force vector at Joint I

$$\{P_1\}_{2\times 1}=\begin{pmatrix} 0-(MS1(-\psi_{td})+CorrTS1(-x_{td})+MS3t(-\psi_{wall}) \\ H1-(CorrMS1(-\psi_{td})+TS1(-x_{td})+CorrMS3t(-\psi_{wall})) \end{pmatrix} \tag{3.384}$$

- Displacement vector at Joint I

$$\{\Delta_1\}_{2\times 1}=\{K_1\}_{2\times 2}^{-1}\quad\times\ \{P_1\}_{2\times 1} \tag{3.385}$$

$$\begin{pmatrix} x_1 \\ \psi_1 \end{pmatrix}=\begin{bmatrix} MS1+MS2+MS3t & \mathrm{CorrTS1}+\mathrm{CorrTS3t} \\ \mathrm{CorrMS1}+\mathrm{CorrMS3} & TS1+TS2+TS3t \end{bmatrix}^{-1}\begin{pmatrix} P_1{}' \\ P_1{}'' \end{pmatrix} \tag{3.386}$$

b) **Joint II- Bottom joint-** This joint has four members- Wall (connected at bottom), bottom dome, middle ring beam & bottom ring beam.

- Global stiffness matrix at Joint II

$$\{K_2\}_{2\times 2}=$$

$$\begin{bmatrix} MS3b + MS4 + MS6 + MS7 & \text{CorrTS3b} + \text{CorrTS6} \\ \text{CorrMS3b} + \text{CorrMS6} & \text{TS3b} + \text{TS4} + \text{TS6} + \text{TS7} \end{bmatrix} \tag{3.387}$$

$$\{K_2\}_{2\times2} =$$

$$\begin{bmatrix} 2\mu Z_b + \frac{Eb_4 {d_4}^3}{12{r_1}^2} + \frac{R_2 Et_6\left(k_{bd} + k_{bd}^{-1}\right)}{4\times{\lambda_{bd}}^3} + \frac{Eb_7 {d_7}^3}{12{r_7}^2} & 2\mu^2 Z_b + \frac{Et_6}{2{\lambda_{bd}}^2 k_{bd} sin\theta_{bd}} \\ 2\mu^2 Z_b + \frac{Et_6}{2{\lambda_{bd}}^2 k_{bd} sin\theta_{bd}} & 4\mu^3 Z_b + \frac{Et_6}{\lambda_{bd} R_2 k_{bd} sin^2\theta_{bd}} + \frac{Eb_4 d_4}{{r_1}^2} + \frac{Eb_7 d_7}{{r_1}^2} \end{bmatrix} \tag{3.388}$$

- Force vector at Joint II

$$\{P_2\}_{2\times1} =$$

$$\begin{pmatrix} BMat\ MRB - (MS3b(-\psi_w) + CorrTS3b(-x_w) + MS6(-\psi_{bd}) + CorrTS6(-x_{bd}) \\ \text{HNET} + HFatMRB - (CorrMS3b(-\psi_w) + TS3b(-x_w) + CorrMS6(-\psi_{bd}) + TS6(-x_{bd})) \end{pmatrix} \tag{3.389}$$

- Displacement vector at Joint II

$$\{\Delta_2\}_{2\times1} = \{K_2\}_{2\times2}^{-1} \times \{P_2\}_{2\times1} \tag{3.390}$$

$$\begin{pmatrix} x_2 \\ \psi_2 \end{pmatrix} = \begin{bmatrix} MS3b + MS4 + MS6 + MS7 & \text{CorrTS3b} + \text{CorrTS6} \\ \text{CorrMS3b} + \text{CorrMS6} & \text{TS3b} + \text{TS4} + \text{TS6} + \text{TS7} \end{bmatrix}^{-1} \begin{pmatrix} {P_2}' \\ {P_2}'' \end{pmatrix} \tag{3.391}$$

3.4.3.3. Calculation of forces acting on the members

These forces include Hoop tension, outward thrust and bending moments and are shown below in Tables.

Table 3.60: Moment calculations

Members	Moments	Eq.No.
Top dome	$MS1\ (\psi_1 - \psi_{td}) + CorrTS1\ (x_1 - x_{td})$	(3.392)
Top ring beam	$\text{MS2}(\psi_1)$	(3.393)
Wall (Top)	$MS3t\ (\psi_1 - \psi_{wall}) + \text{CorrTS3t} \times (x_1)$	(3.394)
Wall (Bottom)	$MS3b\ (\psi_2 - \psi_{wall}) + CorrTS3b\ (x_2 - x_{wall})$	(3.395)
Balcony beam	$MS4\ (\psi_2)$	(3.396)
Bottom beam	$MS7\ (\psi_2)$	(3.397)
Bottom dome	$MS6\ (\psi_2 - \psi_{bd}) + CorrTS6\ (x_2 - x_{bd})$	(3.398)

Table 3.61: Outward thrusts calculations

Members	Outward thrusts	Eq.No.
Top dome	CorrMS1 $(\psi_1 - \psi_{td})$ +TS1 $(x_1 - x_{td})$	(3.399)
Top ring beam	TS2 (x_1)	(3.400)
Wall (top)	$CorrMS3t(\psi_1 - \psi_{wall}) + TS1\ (x_1)$	(3.401)
Wall (bottom)	$CorrMS3b\ (\psi_2 - \psi_{wall}) + TS3b\ (x_2 - x_{wall})$	(3.402)
Balcony beam	$TS4\ (x_2)$	(3.403)
Bottom beam	$TS7\ (x_2)$	(3.404)
Bottom dome	CorrMS6 $(\psi_2 - \psi_{bd})$ +TS6 $(x_2 - x_{bd})$	(3.405)

Table 3.62: Hoop tensions calculations

Members	Hoop tension	Eq.No.
Top dome	$HTension1 = x_1 \times 1000 \times t_1 \times 1000/r_1/1000000;$	(3.406)
Top ring beam	$HTension2 = b_2 d_2 x_1 \times 1000000/1000/r_1;$	(3.407)
Wall (Top)	$HTension3t = t_{3\text{top}} x_1 \times 1000000/1000000/r_1;$	(3.408)
Wall (Bottom)	$HTension3b = t_{3\text{bottom}} x_2 * 1000000/1000000/r_1;$	(3.409)
Balcony beam	$HTension4 = 1000000(t_4^2 + (1 - t_4)(\ t_4 + 0.2) \times 0.5)x_2/1000/r_1;$	(3.410)
Bottom beam	$HTension7 = b_7 d_7 x_3 \times 1000000/1000/r_1;$	(3.411)
Bottom dome	$HTension6 = x_3 \times t_3 \times 1000000/r_1/1000000;$	(3.412)

3.4.3.4. Calculations of reinforcement and checks under serviceability and application of heuristic optimization

This Subsection is the same as that of Intze tank except that of Conical dome.

3.4.4. Module III- Design of members of tank

The design of Top dome, Top ring beam , cylindrical wall, middle ring beam, Bottom dome and Bottom ring beam is same as that of design of Intze tanks

3.4.5. Module IV-VII

Rest of the Modules 4-7 are same as that of Conventional Intze water tanks. Although Circular tanks require more number of column than that of Intze tanks but the analysis and design of staging and foundation is same.

Module IV	Staging configurations and lateral analysis of tanks
Module V	Staging design
Module VI	Foundation design
Module VII	Estimation and costing

3.4.6. Application of software program for CWT I and CWT II circular water tanks-

Two different conventional circular software programs are further developed as per the clause 4.4.3.1. and 4.4.3.3. of IS 3370:2021. Major changes in both the programs are –

Table 3.63: Major differences in software program of Circular CWT I and II

Sr.No	Type	Key difference
1	Conventional type I Circular water tank	• Ultimate steel stresses are kept as 130 N/mm^2 for the elements of the tank body only. steel stresses are kept as ultimate stresses for staging and foundation • Crack width calculations are not mandatory but are performed for study purposes only
2	Conventional type II Circular water tank	• Ultimate steel stresses are kept as 435 N/mm^2 for the elements of the tank body only. Steel stresses are kept as ultimate stresses for staging and foundation • Crack width calculations are mandatory.

In this way software program for the Circular tank is developed. In this way, the software program for circular tanks has been developed. In conclusion, the development of the software for conventional circular water tanks has been meticulously structured, mirroring the modular approach used for conventional RCC Intze tanks. By retaining the same nomenclature for dimensions and elements, the complexity is reduced, and familiarity is maintained, facilitating easier adoption and application of the software. The key distinctions between Intze and circular tanks, particularly in their structural components and interconnections, have been carefully addressed to ensure accurate design and analysis. The software consists of seven modules, covering input and dimensioning, analysis of RCC tank body, design of members, staging configurations and lateral analysis, staging design, foundation design, and estimation and costing. Each module follows a detailed process to ensure the final tank design is both structurally sound and cost-effective, taking into account the specific requirements and challenges of circular tank design. The iterative calculation process ensures optimized dimensions and material selection, while the consistent methodology across modules guarantees reliability and precision. In this way, the software program for circular water tanks has been successfully developed, providing a comprehensive tool for engineers and designers in the field.

3.5. Design details and development of the software for hybrid circular water tanks

Design of Hybrid Circular tanks is having Four steps-

- Design of RCC water tank elements (Container)
- Design of ferrocement lining
- Design of staging
- Design of foundation

Design methodology of RCC tank body, staging and foundation is the same as that of conventional Circular tanks.

Design considerations of HWTs and major shifts from CWT.

3.5.1. Design of RCC water tank elements (container)

The design of the RCC tanks is done using Continuity analysis method and limit state design. In HWTs, RCC is supposed to bear all the structural loads, designed in accordance to IS 456:2000. Analysis and design of RCC tank body uses continuity analysis approach and limit state method of design. The analysis and design are done as per conventional tank only but the minimum exposure criteria, minimum dimension and minimum stress criteria is followed as per IS 456:2000.

Table 3.64: Calculation of the dimensions for conventional Intze water tanks

Member	**Minimum dimension**
Top dome thickness t_1	0.08 m
Top ring beam (b_2 x d_2)	0.2 x 0.2 m
Cylindrical wall at top and bottom (t_{3top} & $t_{3bottom}$)	0.1 m at top and bottom initially.
Middle ring beam (b_4 x d_4)	1.0 x 0.12 m
bottom dome	0.12 m
Main girder beam (b_7 x d_7)	0.4 x 0.5 m
Column ($D_{c)}$	0.4 m
Braces (b_9 x d_9)	0.2 x 0.5 m
Foundation beam (b_{10} x d_{10})	0.4 x 0.5 m
Raft area	As per calculations

3.5.2. Design of ferrocement lining

This section is same as per design of ferrocement lining for hybrid Intze water tanks.

3.5.3. Major differences in the software program of conventional and hybrid circular tanks

The software development process for HWTs is largely similar to that for CWT, with a few key differences. CWT have seven modules, while HWTs have eight. Below are the major changes and details for each module are shown in Table.

The major differences between the software programs for conventional and hybrid circular tanks mainly occur in Modules I to III and Modules VII–VIII. Modules IV to VI, covering staging configuration, staging design, and foundation design, remain the same for both types. In the case of hybrid tanks, the estimation and costing modules are updated: Module VII is replaced with a ferrocement lining design module, and Module VIII handles the estimation and costing, including the ferrocement lining, differing from the conventional approach.

Table 3.65: Major differences in the software program of conventional & hybrid Circular tanks

Modules	Major differences
Module I:	Input & dimensioning module: Minimum exposure criteria, minimum dimension and minimum stress criteria is followed as per IS 456:2000. Initial dimensions are given in Table 4.1.
Module II:	Analysis of RCC tank body & module: Checks and design stresses are taken as per IS 456:2000. Material stresses are provided in Table. The maximum Crack width for the RCC is the same in CWT and HWTs. (i.e. 0.2 mm as per IS 456:2000 in Hybrid & IS 3370:2021 for CWT)
Module III:	Design of members of tank module: The specific surface area of the entire tank body is calculated. Checks and design stresses are taken as per IS 456:2000. Material stresses are provided in Table. The maximum Crack width for the RCC is the same in CWT and HWTs. (i.e. 0.2 mm as per IS 456:2000 in Hybrid & IS 3370:2021 for CWT)
Module IV	Staging configurations and Lateral analysis of tanks module: same as per Conventional Circular tanks
Module V	Staging design module: Same as per Conventional Circular tanks
Module VI	Foundation design module: Same as per Conventional Circular tanks
Module VII	The Estimation and costing module of CWT has been replaced by a new module for HWT design. Module is named as **Module VII-Design of ferrocement lining**
Module VIII	Estimation and costing of HWTs module: Module VIII for Estimation and costing is made which includes the Estimation and costing of ferrocement lining. Rest the Module is same as per Convention Module VII.

3.5.4. Details of the software modules

a) **Module I, Module II & Module III-** The changes in Modules I, II, and III have been detailed in the previous table and discussed comprehensively in Section 3.2 of this study. Additionally, Module III for HWTs includes a new feature for calculating the specific surface area of the water tank container. In this module, alongside the conventional calculations for the quantity of concrete and steel after designing each member, the surface area of each member is also calculated. Equations of calculation of Surface area are as follows-

Table 3.66: Calculation of the dimensions for Hybrid circular water tanks

Member	Surface area equations	
Top dome	$SA_1 = 2\pi R_1 h_1;$	(3.414)
Top ring beam	$SA_2 = 2\pi r_1 t_2;$	(3.415)
Cylindrical wall	$SA_3 = 2\pi(2r_1 + t_{3bottom})H;$	(3.416)
Middle ring beam	$SA_4 = 2\pi r_1 t_4;$	(3.417)
Bottom dome	$SA_6 = 2\pi R_2 h_2;$	(3.418)
Bottom ring beam	$SA_7 = 2\pi r_1(b_7 + d_7);$	(3.419)

b) **Module IV, Module V & Module VI-** These modules as discussed in Table are same as conventional

c) **Module VII – Design of ferrocement lining-** This module is an addition to the Hybrid design and is not present in CWT, as detailed in Section 3.3.2.

d) **Module VIII – Estimation and costing-** This Module is same as Conventional Module No 7 with addition of the calculation of cost for ferrocement lining. Ferrocement lining cost for Cement mortar and steel bars.

Table 3.67: Calculation of the estimation and cost calculations for Hybrid circular tanks

Estimation of quantities		
Total quantity of cement mortar used	SA= $SA1 + SA2 + SA3 + SA4 + SA6 + SA7$;	(3.420)
Total quantity of Steel	$Q_f = Number\ of\ Mesh \times$ $Number\ of\ Bars\ (\frac{\pi\varphi_f^2}{4}) \times SA$ $\varphi_f is\ 0.001m$	(3.421)
Rate analysis		
Rates calculation for plastering	1:2 Cement Mortar is used with OPC 43 grade or PPC. OPC 43 Grade is Recommended. 1: 2 Cement Sand mortar rate was not available in CPWD DAR 2021. Hence Rate analysis is done to calculate the Rate of 12 mm, 15 mm and 20 mm using Cement Sand mortar	
1:2 Cement sand mortar Rate in cum	Rs. 5613.46 per cum (clause 3.7 of CPWD DAR 2021)	
Final rates of cement mortar	10 mm thick plaster- Rs 233.9 per sqm 12 mm thick plaster- Rs 249.4 per sqm 15 mm thick plaster- Rs 281.5 per sqm 20 mm thick plaster- Rs 319.0 per sqm	
Final rates of steel	HB bars are recommended to be used. Rate of HB wires is not given in DAR 2021 CPWD. However Steel Rate per kg of Steel as used for Steel is used =Rs 89.65; (clause 5.22.4)	

The development of software for the design of hybrid circular water tanks represents a significant advancement in the field of water storage infrastructure. By integrating both RCC and ferrocement technologies, these HWTs combine the structural robustness of traditional RCC tanks with the enhanced durability and crack resistance offered by ferrocement linings. The design process retains many conventional methodologies but incorporates additional considerations to address the unique properties and benefits of HWTs. The eight-module software framework provides a comprehensive approach, covering all aspects from structural analysis and design to detailed cost estimation. This holistic approach ensures the design of efficient, cost-effective, and long-lasting water tanks that meet contemporary standards as per IS 456:2000 and IS 3370:2021. The inclusion of ferrocement linings and their detailed costing within the software underscores a forward-thinking approach, making this tool invaluable for modern water tank design and construction projects.

3.6. Validation of software program

Before advancing to the software development phase, a preliminary manual pilot test was conducted to evaluate the outcomes of both conventional and hybrid designs. Over 50 manual designs were carried out for tanks varying in capacity from 100 to 1000 kL, with increments of 50 kL. These designs accounted for different staging heights, seismic zones, and wind speeds. Specifically, staging designs were completed for all four seismic zones and at least four basic wind speeds (39, 44, 47, and 50 m/s) for each tank capacity. The foundation was also scrutinized for each scenario, considering diverse soil bearing capacities ranging from 80 to 250 kN/m^2.

After obtaining positive results from these manual designs, the development of the software was initiated. The results from the manual designs were then cross-verified using the newly developed software program. Additionally, standard problems from reference texts were examined using the software to ensure validation.

Detailed manual design calculations for Hybrid Intze tanks of 600 kL and Hybrid Circular water tanks of 200 kL are provided in **Annexure B and C**, along with the corresponding software outputs for validation attached in **Annexure D and E**. Furthermore, the calculations for ferrocement lining and the required lining thickness for different strain values are included in **Annexure A.**