JETIR.ORG ISSN: 2349-5162 | ESTD Year : 2014 | Monthly Issue
JOURNAL OF EMERGING TECHNOLOGIES AND INNOVATIVE RESEARCH (JETIR)
An International Scholarly Open Access, Peer-reviewed, Refereed Journal

ANALYSIS AND DESIGN OF INTZE TYPE OVERHEAD WATER TANK UNDER THE HYDROASTATIC PRESSURE BY USING SOFTWARE

Analyzing, Designing & Comparison between Software based calculations and manual calculations of Intze type overhead tank under hydrostatic pressure

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Abstract: This study has been undertaken to investigate the determinants of behavior & design of Intze type overhead tank under different cases of hydrostatic pressure i.e. full and empty condition applied on it and also comparing this conditions between manual calculations and software based calculation. This study also relates the accuracy of manual calculation and software based calculation. Water tanks are important public utility and industrial structure. The design and construction methods in reinforced concrete are influenced by the prevailing construction practices, the physical property of the material and the climatic conditions. Before taking up the design, the designer should first decide the most suitable type of staging of tanks and correct estimation of loads including statically equilibrium of structure particularly in regards to overturning of overhanging members shall be made. The design should be based on the worst possible combination of loads, moments and shears arising from vertical loads and horizontal loads acting in any direction when the tank is full as well as empty. In this research by performing the analysis of Intze tank, what is deflection shape due to hydrostatic pressure then stresses, etc. which are analyzed. This study also relates which calculation gives most economical results.

***Index Terms* - Water, hydrostatic pressure, types of tank, Intze water tank, reviews, analysis, design criteria as per IS code.**

I. INTRODUCTION

Overhead tanks and reservoirs are liquid storage containers. These containers are generally used for storing water for irrigation works, human consumption, fire, manufacturing units, rainwater harvesting, and for many purposes. The main purpose of design of tanks are economical, strength, service life, to provide safe portable drinking water after storing for a long time and it also resist special conditions like wind and earthquakes. Water tanks are generally constructed with reinforced concrete or steel and design is based on IS code. Design of tanks depends on the position of tank i.e, above or below, at the ground level. The overhead tanks are generally constructed at certain height from the ground level using columns and braces, for direct distribution of water by gravity. In any case, the underground tanks are rest underneath the ground level. In the construction of concrete structure for the storage of water and other liquids the imperviousness of concrete is most essential. The permeability of any uniform and thoroughly compacted concrete of given mix proportions is mainly dependent on water cement ratio. The increase in water cement ratio results in increase in the permeability. The decrease in water cement ratio will therefore be desirable to decrease the permeability, but very much reduced water cement ratio may cause compaction difficulties and prove to be harmful also. Design of liquid retaining structure has to be based on the avoidance of cracking in the concrete having regard to its tensile strength. Cracks can be prevented by avoiding the use of thick timber shuttering which prevent the easy escape of heat of hydration from the concrete mass. The risk of cracking can also be minimized by reducing the restraints on free expansion or contraction of the structure. The main reason for life loss is collapse of structures It is said that natural calamities itself never kills people; it is badly constructed structure that kill. Hence it is important to analyse the structure properly for different natural calamities like earthquake, cyclones, floods and typhoons etc. and also should be well studied and calculated under the hydrostatic pressure applied on it.

II. WATER TANK IN GENERAL AND TYPES OF WATER TANK

2.1 Types of Tank:

In recent years, there has been much emphasis on water supply projects all over the world, which are very essential for the social and industrial development of the country. Water tanks can be of different capacity depending upon the requirement of consumption. Based on the location the water tanks are classified into three ways:

Types of Tank

Based on heads

1. Tank resting on ground
2. Under ground water tank.
3. Elevated or Overhead water tank

Based on shape of tank

1. Circular tanks
2. Rectangular tanks
3. Intze tanks
4. Spherical tanks

2.2 Usage of water tanks:

A reinforcement concrete tank is a very useful structure which is meant for the storage of water, for swimming bath, sewage sedimentation and for such similar purposes. Reinforced concrete overhead water tanks are used to store and supply safe drinking water.

2.3 Overhead water tank or E.S.R (Elevated Storage Reservoir)

Overhead water tanks of various shapes can be used as service reservoirs, as a balancing tank in water supply schemes and for replenishing the tanks for various purposes. For an efficient water distribution system, overhead water tanks or elevated storage reservoirs are one of the most important components. The basic purpose of elevated water tanks is to secure constant water supply with sufficient flow to wide area by gravity. The height of the elevated tank depends on the area to be covered for the water supply. Wider the area to be served higher will be the required elevation of the tank.

2.4 Intze Water Tank

Intze water tanks are constructed to minimize the project cost because lower dome in this construction resists the horizontal thrust. This type tank is simplest form as compare to the circular tank.

A German hydraulic engineer is given the name tank as Intze. The water tower built in accordance with the Intze precept has a brick shaft on which the water tank sits. the bottom of the tank is fixed with a hoop anchor (Ring Anker) manufactured from iron or metal, so that handiest vertical, no longer horizontal, forces are transmitted to the tower. In this project Intze tank is designed with working stress method and elements of Intze tank is designed with limit state method.

It can be divided into two types based on support:

a) Column rested water tank

b) Shaft rested water tank

2.5 Components and Structural elements of Intze water tank

The various structural elements of an Intze type tank comprises of the following:

1. Top spherical dome
2. Top ring beam
3. Circular side walls
4. Bottom ring beam
5. Conical dome
6. Bottom spherical dome
7. Bottom circular girder
8. Foundations
9. Tower with columns and braces

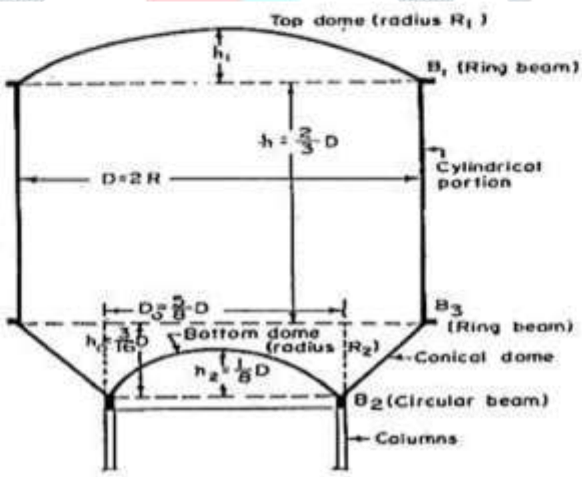


Figure-1: Typical section showing components of Intze type Water Tank

III. DESIGN CRITERIA OF INTZE TANK

The design of the tank as per IS code will involve the following:

1) **The top dome:** At top, usually 100 mm to 150 mm thick slab with reinforcement along the meridians and latitudes. The rise for dome is usually l/5th of the span.

2) **Ring beam supporting the dome:** The ring beam is necessary to resist the horizontal component of the thrust of the dome. The ring beam will be designed for the hoop tension induced.
3) **Cylindrical walls:** This should be designed for hoop tension caused due to horizontal water pressure.

4) **Ring beam at the junction of the cylindrical walls and the conical wall:** This ring beam is provided to resist the horizontal component of the reaction of the conical wall on the cylindrical wall. The ring beam will be designed for the induced hoop tension.

5) **Conical slab:** This will be designed for hoop tension due to water pressure. The slab will also be designed as a slab spanning between the ring beam at top and the ring girder at bottom.

6) **Floor of the tank:** The floor may be circular or domed. This slab is supported on the ring girder.

7) **The ring girder:** This will be designed to supportthe tank and its contents. The girder will be supported on columns and should be designed for resulting bending moment and Torsion.

8) **Columns:** These are to be designed for the total load transferred to them. The columns will bebraced at intervals and have to be designed for wind pressure or seismic loads whichever govern.

9) **Foundations:** A combined footing is usuallyprovided for all supporting columns. When this is done, it is usual to make the foundation consisting of a ring girder and a circular slab.

IV. DESIGN & ANALYSIS OF INTZE TANK BY MANUAL CALCULATION

4.1 DIMENSION OF TANK

Steel = Fe 415 Concrete grade = M30

Diameter of tank (D) = 15 m

Diameter of lower Ring Beam (D0) = 15 × 0.6 = 9m

Rise of top dome (h1) = 2.5m

Rise of bottom dome (h2) = 1.5m

Height of conical dome (h0) = 2m

Height of cylindrical portion = $\frac{\pi}{4}$ x D^2 x h + $\frac{\pi}{12}$ x h0 ($D^2 + D0^2 + D \times D0$) – $\frac{\pi}{3}$ x $h_2^2 \times (3R_2 - h_2)$

Where, $R_2 = \dfrac{(\frac{D}{2})^2+h^2}{2 \text{ x } h} = \dfrac{(\frac{15}{2})^2+1.5^2}{2 \text{ x } 1.5}$ = 7.25

$900 = (\frac{\pi}{4} \text{ x } 15^2 \text{ x } h) + \frac{\pi}{12} \text{ x } 2 \text{ x } (15^2 + 9^2 + 15 \times 9) - \frac{\pi}{3} \text{ x } 1.5^2 \times (3 \text{ x } 7.25 - 1.5)$

h = 4.05 m

Say, h = 4.50 m

As per calculated dimensions capacity of tank = 900 Cum = 900 x 100 = 9, 00,000 liters= **90 KL**

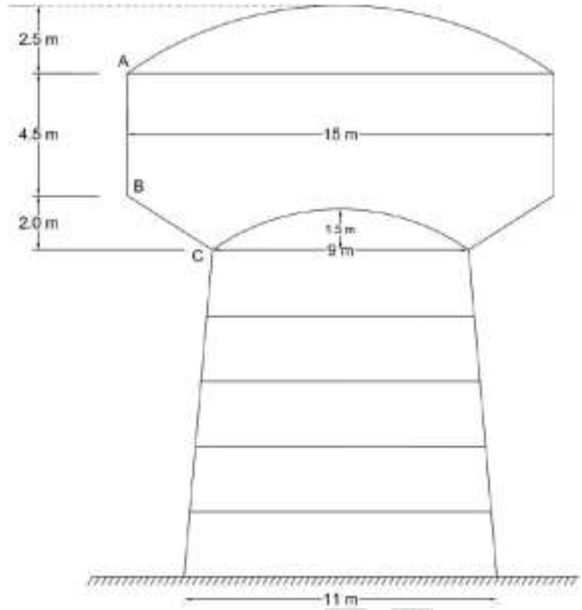
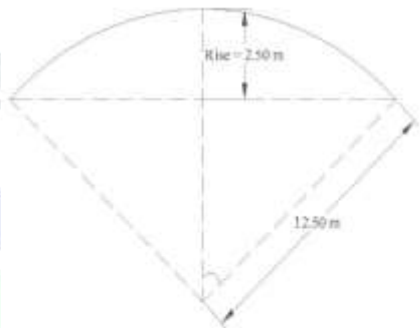


Figure 4.1 : Dimension of water tank

4.2 DESIGN OF TOP DOME

Provide a thickness of 150 mm for the roof dome

Let 2θ be the angle subtended by the dome of its center



$\therefore \text{Sin } \theta = \dfrac{d}{2R_1}$, Where, $R_1 = \dfrac{(\frac{15}{2})^2 + 2.5^2}{2\ X\ 2.5} = 12.5$ m

$\theta = \text{Sin}^{-1}\ (\dfrac{15}{2 \text{ x } 12.5})$

$\theta = 36.87^0$

Figure 4.2: Details rise and angle of top dome

4.2.1 LOADS

a) Dead load = 0.15 × Density of concrete ………………….. (As per IS Code 856 part-1)

Dead load = 0.15 × 25000 ……………….....……T (Density of Concrete = 25kN/m)

= 3750 N/m2

b) Live load of dome = 0.75 – 0.52 y2 ……………….... (From Table 2, IS: 875 part – 2)

$y = \frac{h}{D} = \frac{2.5}{15} = 0.75 - 0.52 \times (\frac{2.5}{15})^2$ = 0.735 KN/m2 = 735 N/m2

Total load (w) = 3750 + 735 = 4485 N/m2

4.2.2 HOOP STRESS AT THE LEVEL OF SPRINGING

$$f = \frac{w\,R1}{t}\left(\text{Cos}\ \theta - \frac{1}{1+\text{Cos}\ \theta}\right) = \frac{4485 \times 12.5}{0.25}\left(\text{Cos}\ 36.87° - \frac{1}{1+\text{Cos}\ 36.87°}\right) = 91360.5\ \text{N/m}^2$$

$= 0.091\ \text{N/mm}^2$

4.2.3 HOOP STRESS AT THE CROWN

i.e., at $\theta = 0°$

$$f = \frac{w\,R1}{t}\left(1 - \frac{1}{1+1}\right) = \frac{4485 \times 12.5}{0.25} \times 0.5 = 186875\ \text{N/m}^2 = 0.18\ \text{N/mm}^2$$

Meridional thrust at the level of the springing, per meter run:

$$T_1 = \frac{wR}{1+\text{Cos}\theta} = \frac{4485 \times 12.5}{1+\text{Cos}\ 36.87°} = 31145\ \text{N/m}$$

∴ Meridional Stress = $\frac{31145}{150 \times 1000} = 0.21\ \text{N/mm}^2$

These stresses are very small. Provide nominal reinforcement

∴ Provide nominal reinforcement (0.3%)

∴ Ast = $\frac{0.30}{100} \times 1000 \times 150 = 450$ mm2

∴ Provide 8 mm ϕ @ 110 mm c/c

4.3 RING BEAM AT TOP

Horizontal component of T1 = T1 Cos θ = 31145 × Cos 36.870 = 24915 N

Hoop tension in the ring beam =24915 × $\frac{15}{2}$ = 186862 N

∴ Area of steel required for hoop tension = $\frac{186862}{150}$ = 1245 mm2

Provide 6 bars 18 mm diameter (1526 mm²)

4.3.1 SIZE OF THE RING BEAM

Let the area of the ring beam section = A mm²

Equivalent concrete area = A + (m-1) Ast

$$\sigma_{ct} = \frac{T}{A+(m-1)Ast} \Rightarrow A + (m-1)Ast = A + (13.33 - 1) \times 1526 = A + 18815\ \text{mm2}$$

Limiting tensile stress on the equivalent concrete area to 1. 2 N/mm2

$\sigma_{ct} = \frac{T}{A+(m-1)Ast} = \frac{186862}{A+18815} = 1.2$ ➡ A = 136903 mm2

Provide 350 mm × 400 mm section

Shear reinforcement

Provide 8 φ – 2 legged stirrups IS : 456 – 2000 , Cl – 26 .5.1.6

1) 0.7 D + 0.75 × 400 = 300; (2) 300 mm , and (3) $S_v = \frac{0.87 \text{ x } f_y \text{ x } Asv}{0.4 \text{ x } b}$

Asv = 2 x $\frac{\pi}{4}$ x 8² = 100 mm²

∴ Sv $= \frac{0.87 \text{ x } 415 \text{ x } 100}{0.4 \text{ x } 350}$ = 257.89 mm

Provide, Sv = 250 mm

∴ Provide 8 φ – 2 legged vertical stirrups @ 250 mm C/C

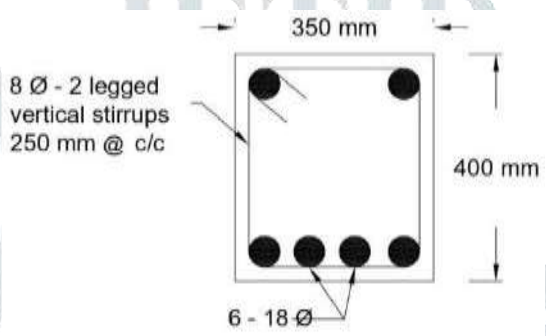


Figure 4.3: Details of top ring beam reinforcement

4.4 CYLINDRICAL WALL

Pressure intensity at the bottom of cylindrical wall = 4 × 10000 = 40000 N/m2

Consider bottom strip of the wall as 1 m

Hoop tension = $\frac{Pd}{2}$ = 40,000 x $\frac{15}{2}$ = 30, 00,000 N

Ast = $\frac{3000000}{150}$ = 2000 mm²

Spacing of 12 mm diameter bars

Spacing = $\frac{\frac{\pi}{4} x \ 12^2 \ x \ 1000}{2000}$ = 56 mm

Provide 12 mm φ bars @ 110 mm centers near each face

Thickness of wall, Ct = $\frac{T}{1000t+(m-1)Ast}$ N/mm² <1.2

T = $\frac{Yw \ x \ h \ x \ D}{2} = \frac{10 \ x \ 4.5 \ x \ 15}{2}$ =337.5KN=337500N

∴ $A_{st} = \frac{\pi}{4}$ x 12² x 110 = 1028 mm²

$= \frac{337500}{1000 \ x \ t \ (13.33-1) \ x \ 1028}$ < 1.2 N/mm² , ∴ 268.57 < t , ∴ Provide t = 300 mm

B

Thickness of the wall may be kept as 300 mm.

Distributions steel = $\frac{0.3}{100}$ x (300 x 1000) = 900mm^2

Specing of 8mm Ø bars = $\frac{50 \, x \, 1000}{900} = 55$mm

Provide 8mm Ø bars @ 100 mm centers near each face.

Check for compressive stress at the bottom of the cylindrical wall.

Vertical component of $T_1 = V_1 = T_1 \sin\theta$ ➡ $31145 \times \sin 36.87^0$ ➡ 18687 N/m

Weight of the wall = 0.3 × 4.5 × 25000 = 33750 N/m.

Weight of ring beam = 0.35 × 0.4 ×25000 = 3500 N/m.

V_2 = Total vertical load per ′m` = 18687 + 33750 + 3500 = 55937 N/m.

Compressive stress = $\frac{55937}{300 \, x \, 1000}$ = 0.18N/mm^2

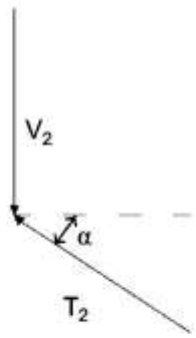


Figure 4.4: Showing moments & loads acting on point B bottom of cylindrical wall

This stress being low profile nominal vertical stress at 0.3 % gross area

∴ Vertical steel = $\frac{0.3}{100}$ (300 x 1000) = 900 mm^2

Spacing of 8 mm φ bars = $\frac{\frac{\pi}{4} \, x \, 8^2 \, x \, 1000}{900}$

Provide of 8 mm φ bars @ 100 mm c/c near each face

4.5 RING BEAM AT BOTTOM (B) :-

Let T_2 be the thrust per meter run exerted by the conical slab at the junction B.

Resolving vertically at B, $T_2 \sin\alpha = V_2$ = 55937 N/m.

$\tan \alpha = \frac{2}{3} = 0.67$, α = 33.69°, T_2 = 55937 N/m

Resolving horizontally at B, $H_2 = T_2 \cos\alpha = V_2 \cot\alpha = \frac{55937}{0.67} = 83488$ N/m

This horizontal load H_2 will produce a hoop tension in ring beam B

Hoop tension due to $H_2 = H_2 \, x \, \frac{d}{2} = 83488 \, x \, \frac{15}{2} = 626160$ N

Let the ring beam be 1000mm deep.

Water pressure on the ring beam = $1000 \times 4.5 \times \frac{900}{1000}$ = 40500 N/m

Hoop tension due to water pressure = $40500 \times \frac{15}{2}$ = 303750 N

∴ Total hoop tension = 626160 + 303750 = 929910 N

∴ Steel for hoop tension = $\frac{929910}{150}$ = 6199 mm^2

Provide 24 mm φ bars, number of bars required = 6199/ (π/4 x 24^2) = 13.70 bars

Provide 14 bars of 24 mm diameter.

Let the area of the ring beam section be A.

Equivalent concrete area = A+(m-1) × Ast

= A+ (13.33-1) × 6333➡ A+ 78085

Limiting the tensile stress on the equivalent concrete area to 1.2 N/mm^2

$= \frac{929910}{A + 78085} = 1.2$ ➡ A = 696840mm^2

Provide, 750 mm × 950 mm section

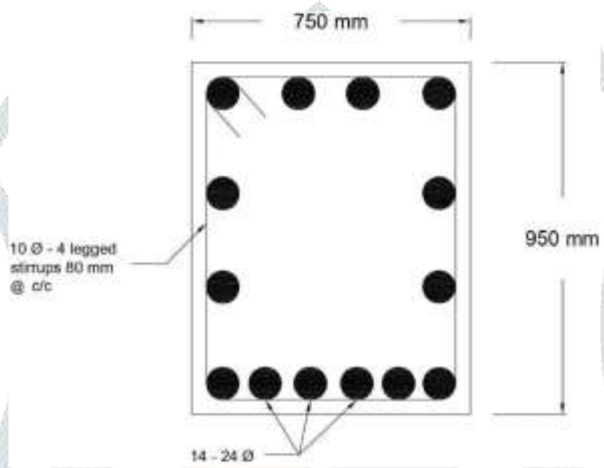


Figure 4.5: Reinforcement details of bottom ring beam

4.6 DESIGN OF CONICAL SLAB

The Conical slab should be designed for

a) Hoop tension
b) Bending as it spans on a sloping slab from the ring beam at 'B' at the ring girder at 'C'.

4.6.1 DESIGN FOR HOOP TENSION

The hoop tension on the conical slab is given by = $\frac{W_{w+}W_s}{2\,\pi} + \frac{W_w}{2\,\pi}\tan\alpha$

Where,

W_w = weight of water resting on the conical slab.

W_s = weight of the conical slab.

α = inclination of the conical slab with the horizontal.

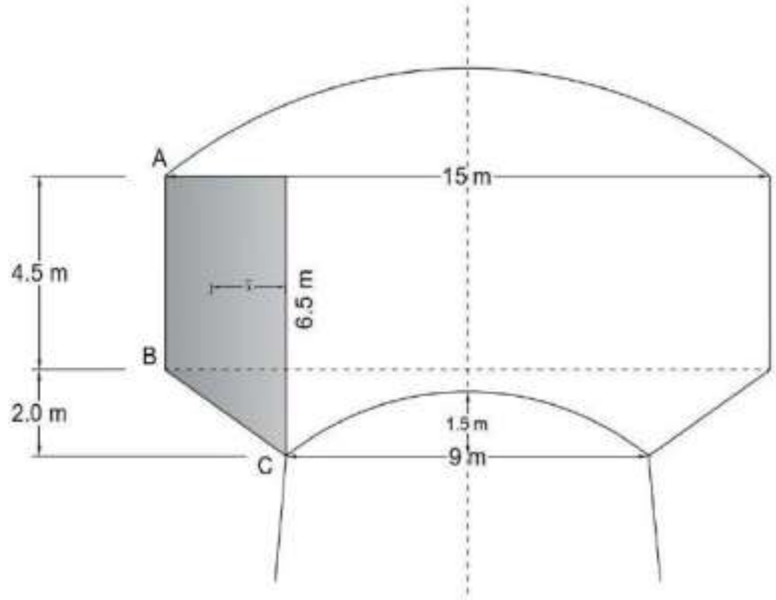


Figure 4.6: Design of Dome Section

Area of water section standing on the conical slab = $\frac{3}{2}$ (4.5 + 6.5) = 16.5 m^2

$\bar{x} = \frac{(4.5 \text{ x } 3)+(\frac{3 \text{ x } 2}{3})}{16.5}$ = 0.94 m

∴ Weight of water resting on the conical slab , $W_w = \gamma \times A \times 2\pi \times (\frac{9}{2} + \bar{x})$

$= 1000 \text{ x } 16.5 \text{ x } 2\pi \text{ x } (\frac{9}{2} + 0.94) = 5639787$ N

Length of sloping slab = $\sqrt{3^2 + 2^2}$ = 3.6055 m

Thickness of sloping slab = 300 mm.

∴ Weight of the conical slab Ws = 3.6055 × 0.3 × 25000 × 2π × ($\frac{15}{2} + \frac{9}{2}$) / 2 = 1019431 N

∴ Hoop tension = $\frac{5639787+1019431}{2\pi} + \frac{5639787}{2\pi} \text{ x } \frac{2}{3} = 1658247$ N

∴ Hoop steel on the entire section = $\frac{1658247}{150}$ = 11054 mm²

Provide 35 bars of 20 mm diameter

These bars may be distributed near both the faces of conical slab.

4.6.2 DESIGN FOR BENDING MOMENT

Load per meter width of the conical slab = $\frac{W_{w+} W_s}{2 \pi \text{ x mean radian}} = \frac{5639787+1019431}{2 \pi \text{ x } 6}$ = 176641 N

Maximum bending moment = $\frac{wl}{8} = \frac{176641 \text{ x } 3}{8}$ = 66240 Nm.

Axial compression $T_2 = \frac{V_2}{\sin \alpha} = \frac{55937}{\sin 33.69^\circ}$ = 100842 N

Providing 20 mm diameter bar at clear covers of spacing 25 mm

Effective depth = 300−25− 10 = 265 mm.

Distance between centre of section and centre of steel

$\overline{x} = d - \frac{t}{2} = 265 - 150 \quad = 115$ mm

Resultant bending moment = M + T2 x = = 66240n×103 + (100842 × 115) = 77836830 Nmm

$A_{st} = \frac{M}{\sigma_{st} \, x \, j \, x \, d} = \frac{77836830}{190 \, x \, 0.86 \, x \, 265} = 1797 \text{ mm}^2$

Spacing of 20 mm dia bar = $\frac{\frac{\pi}{4} x \, 20^2 x \, 1000}{1797} = 174$ mm

provide 20 mm φ bars @ 170 mm c/c

4.7 THE BOTTOM DOME

Span of the dome = 9 m, Rise of the dome = 1.5 m,

Let R be the radius of dome

$R = \frac{\left(\frac{D}{2}\right)^2 + h^2}{2 \, x \, h} = \frac{\left(\frac{9}{2}\right)^2 + 1.5^2}{2 \, x \, 1.5} = 7.5$ m

Let 2θ be the angle subtended by the dome

$\text{Sin } \theta = \frac{D/2}{R} = 0.6; \;\; \theta = 36.86^{\circ}$

Thickness of dome = 300 mm

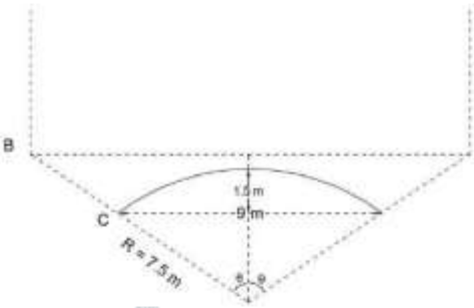


Figure 4.7: Details of bottom dome

4.7 .1 LOADS

Dead load = 25000 x 0.3 = 7500 N/m²

Weight of water resting on the dome (when tank is full)

$= \gamma_w \left[\frac{\pi}{4} x \, d^2 x \, h - \frac{\pi \, x \, h_c}{3}\right] (3R - h_c) = 1000 \left[\frac{\pi}{4} x \, 9^2 x \, 6.5 - \frac{\pi \, x \, 1.5}{3}\right] (3 \, x \, 7.5 - 1.5) = 3805254$ N

Area of dome surface = 2 π × R × h = 2 × π × 7.5 × 1.25 = 70.68 m²

Load intensity due to weight of water = $\frac{3805254}{70.68} = 53838 \text{ N/m}^2$

Total load intensity = 53838 + 7500 = 661338 N/m^2.

Meridional thrust = $\frac{WR}{1+COS\theta} = \frac{61338 \times 7.5}{1+\cos 36.86^{\circ}} = 255560$ N/m

Meridional compressive stress = $\frac{255560}{300 \times 1000} = 0.85 \text{ N/mm}^2$

Hoop stress = $\frac{WR}{t}\left(\cos\theta - \frac{1}{1+\cos\theta}\right) = \frac{61338 \times 7.5}{0.3}\left(\cos 36.86^{\circ} - \frac{1}{1+\cos 36.86^{\circ}}\right) = 0.375 \text{ N/mm}^2$

Hoop stress at the crown , i.e at θ = 0°

Maximum hoop stress = $\frac{WR}{t}\left(\cos\theta - \frac{1}{1+\cos\theta}\right) = \frac{61338 \times 7.5}{0.3}\left(1 - \frac{1}{2}\right) = 0.767 \text{ N/mm}^2$

These stresses are low and hence provide nominal 0.3% steel.

∴ Provide 8 mm bars @ 100 mm spacing

4.8 CIRCULAR GIRDER

The total load acting on the circular girder will be from the following loads: -

Weight of water, w1 = weight of water on (conical slab + dome)

= 5639787 + 3805254

= 9445041 N

Weight of top dome and side wall, w2 = v2× 2π× D/2

= 55397 × 2π× 15/2 = 2610522 N

Weight of ring beam at B, w3 = 0.75 × 0.95 × 25000 × 2π × 4.5 = 503636 N

Weight of conical wall, w4 = 1019431 N

Weight of lower dome, w5 = 25000 × 0.3 × 70.68 = 530100 N

Total load, W = w1 + w2 + w3 + w4 + w5 = 14108730 N

Providing 8 no.s of columns: -

Max. –ve bending moment = 0.0083Wr = 0.0083 × 14108730 × 4.5 = 526961 Nm

Max. +ve bending moment = 0.00416Wr = 0.00416 × 14108730 × 4.5 = 264115 Nm

Max. torsion = 0.0006Wr = 0.0006 × 14108730 × 4.5 = 38093 Nm

Shear force at support = $\frac{\mathrm{W}}{2\times\text{Number of columns}} = \frac{14108730}{2\times 8}$ = 881796 N

4.8 .1 DESING AT SUPPORT SECTION

Equating the moment of resistance to the B.M at support

0.913 bd2 = Mmax

= 0.913 × 750 × d2 = 526961 × 1000

∴ d = 880 mm Overall depth of beam = 905 mm

Actual effective depth = 880 mm [(905-25) →25 mm = clear cover]

Equivalent shear force = Se = S × 1.6$\times \frac{T}{b}$ ➡ = 881796 + 1.6 × $\frac{38093 \text{ x } 10^3}{750}$ = 963061N

Equivalent nominal shear stress ➡ $\tau_{vc} = \frac{S_e}{bd} = \frac{963061}{750 \text{ x } 880}$ = 1.46 N/mm²

Maximum shear stress $\tau_{max} > \tau_v$

τ_{max} = 2.2 N/mm² (for M30 concrete)

∴ Hence safe.

Provide longitudinal and transverse reinforcement according to - B – 6.4 (IS: 456 – 2000)

LONGITUDINAL REINFORCEMENT

Me = M + M1

$M_1 = \frac{T\,(1+\frac{D}{b})}{1.7} = \frac{38093 \text{ x } 10^3\,(1+\frac{905}{750})}{1.7}$ = 49446207 Nmm

Ast = $\frac{M}{\sigma st \text{ x } j \text{ x } d} = \frac{576907207}{230 \text{ x } 0.9 \text{ x } 880}$ = 3164 mm²

∴ Provide 12 bars of 20 mm Ø (3770 mm²)

TRANSVERSE REINFORCEMENT

Asv = $\frac{T \times S_V}{b_1 \times d_1 \times \sigma_{SV}} + \frac{V \times S_V}{2S \times d_1 \times \sigma_{SV}}$

Distance between centers of corner bars parallel to the width; b1 = 750 - 2 × 40 = 670 mm

Distance between centers of corner bars parallel to the depth; d1 = 905 - 2 × 40 = 825 mm

Area section of stirrups ➡ Asv = $[\frac{38093 \times 10^3}{670 \times 825 \times 230} + \frac{963061}{2.5 \times 825 \times 230}] \times S_V$

∴ Provide 4 legged 10 mm stirrups.

Ast = $4 \times \frac{\pi}{4} \times 10^2$ = 316 mm²

S_V = 136 mm; Provide, S_V = 100 mm

Transverse reinforcement shall not be less than ➡ $\frac{(\tau_{vc} \times \tau_c) \times b \times S_V}{\sigma_{SV}}$ ➡ $\frac{1.46 - 0.3 \times 750}{230} \times S_V = 315$

S_V = 83 mm

Provide 80 mm spacing

Steel for sagging moment

Ast = $\frac{M}{\sigma st \times j \times d} = \frac{264115 \times 10^3}{230 \times 0.90 \times 880}$ = 1451.55 mm²

Provide 6 bars of 18 mm diameter ➡ Ast= 1527 mm2

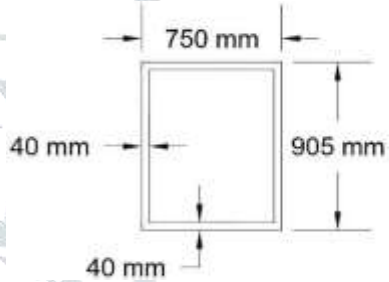


Figure 4.8: Section of Beam

HOOP STRESS

Tc = Thrust exerted by the conical slab on the girder.

Tc sinα × 2πr = Ww+ Ws+ weight of cylindrical wall and upper dome. Tc sinα ×2πr = 5639787 + 1019431 + 2610522

Tc sin33.690 × 2π 4.5 = 9269740 = 591041 N

Horizontal component of Tc= 591041 ×cos 33.69°, H1 = 491777 N

Horizontal component due to dome T'= 255560 × cos36.860 ➡ H2 = 204475 N

∴ Net Horizontal force =H1 – H2 = 287302 N.

∴ Hoop force = 287302 × 4.5 = 1292859 N

Hoop compressive stress = $\frac{1292859}{750 \times 905}$ = 1.90 N/mm²

4.9 COLUMNS

Columns should be designed for direct loads coming upon them and for Bending moments caused by wind load.

Vertical load on one column at top = $\frac{14108730}{8}$ = 1763592 N

Let α be the inclination of the column with the vertical.

Tan α = $\frac{1}{12}$, α = 4.76°; sin α = 0.83 , cos α = 0.99

Actual length of column = 1 sec α = 12.04 m.

Providing 500 mm diameter column

Weight of column = $\frac{\pi}{4}$ x 0.5^2 x 12.04 x 25000 = 59102

Total vertical load = 1763592 + 59102 N = 1822694 N

∴ Corresponding axis load = $\frac{1822694}{0.995}$ = 1822694 N

Weight of water in tank = 9445041

Weight of water transmitted to one column = $\frac{9445041}{8}$ = 1180631 N

Vertical load on one column when the tank is empty = 1822694 – 1180631 = 642064 N

∴ Corresponding axial load = $\frac{642064}{0.996}$ = 644643 N

Ignoring wind load effect if the steel requirement is Asc

Then, C (A – Asc) + Asc = 1830014 ➔ 8 ($\frac{\pi}{4}$ x Asc) + 190 x Asc = 1830014

Hence, Asc = 1424 mm^2

Minimum Requirement of steel = 0.8% = $\frac{0.8}{100}$ x ($\frac{\pi}{4}$ x 500^2) = 1571 mm^2

Provide 7 bars of 20mm diameter = 2199 mm2

(More steel has been subjected since the column is subjected to bending moment caused by wind load)

4.10 STAGING AND GRAVITY LOAD

Height of column = 12.04m

No. of column = 8, Dia. of column = 500 mm

No. of braces = 4, Size of braces = 300 × 500 mm

4.10.1 GRAVITY LOADS

a) from container

When full = 14108730 N

When empty = 4663689 N

b) Weight of column = 8 x $\frac{\pi}{4}$ x 0.5^2 x 12.04 x 25 x 1000 = 472809.7 N or 472.80 kN …………… (i)

c) Weight of braces

Clear length of braces between two column = 2 x $\frac{10.5}{2}$ x sin 22.5˚ - 0.5 = 3.518 m

Weight of braces = 3.518 × 0.3 × 0.5 × 25 ×4 × 8 = 422160 N = 422.16kN …………………….. (ii)

Total weight of staging = 472809.7 + 422160 = 894969.7 N = 894.96kN …………….. (iii = ii + i)

4.11 LATERAL FORCES

4.11.1 SEISMIC FORCES

Consider 1.5 percent longitudinal steel in column

Equivalent area of column = $\frac{\pi}{4}$ x 500² x (1 + (13.11-1) x 0.015) = 232665 mm²

Equivalent area of column = $\sqrt{\frac{232665}{\frac{\pi}{4}}}$ = 544.3 mm, Say 550 mm

$I_r = \frac{\pi}{64} \times 5502 = 4.4918 \times 10^9$ mm⁴

$E = 500\sqrt{30} = 2.7386 \times 10^4$ MPa

Stiffness of the staging

$$K = \frac{n(12EI)}{\sum hi^2} = \frac{8\times12\times2.7386\times10^2\times4.4918\times10^9}{4\times3.01^3\times10^9} = 1.0826\times10^5 \text{ N/mm}$$

Case – I : When tank is full

$$W = 14108730 + \frac{894969.7}{3} = 14407054 \text{ N}$$

$$T_f = 2\pi\sqrt{\frac{Wf}{gk}} = 2\pi\sqrt{\frac{14407054}{9810\times1.0826\times10^5}} = 0.732 \text{ sec}$$

From fig- 2 of IS: 1893 – 1984, for Tf = 0.732 sec and 5 % damping (assumed)

$\frac{S_a}{y} = 0.13$

β = 1, from table – 3 of IS: 1893 – 1984

I = 1.5, from table – 4 of IS: 1893 – 1984

F0 = 0.4, from table – 2 of IS: 1893 – 1984

$\alpha h = \beta \, I \, F_0 \frac{S_a}{y} = 0.096$

Seismic force = 0.081 × 14407054 = 1166977.37 N

Case – 2: When tank is empty

$$W_f = 4663689 + \frac{894969.7}{3} = 4962012 \text{ N}$$

$$T_t = 2\pi\sqrt{\frac{Wf}{gk}} = 2\pi\sqrt{\frac{4962012}{9810\times1.0826\times10^5}} = 0.430 \text{ sec}$$

From fig- 2 of IS: 1893 – 1984, for Tf = 0.430 sec and 5 % damping (assumed)

$\frac{S_a}{y}$ = 0.16

$\alpha_h = \beta I F_{\circ} \frac{S_a}{y}$ = 0.09

Seismic force = 0.096 × 4962012 = 4763254 N

Shear force per column = $\frac{1166971.3}{8}$ = 145871.4N

Maximum bending moment for the column

145871.4× $\frac{3.01}{2}$ = 219536.4 Nm

WIND FORCES

Terrain category – 2

Type of structure – class A

Basic wind speed of North – East Indian,Vb = 50 m/sec

Design wind speed, Vz = Vb K1 K2 K3

Rise coefficient K1 = 0.9; from table -1 of IS: 875 -1987 part – 3

Terrain height factor K2 = 1, 124; fr0m table -2 of IS: 875 -1987 part – 3

Topography factor, K3 = 1; for slope < 30, as per cl.5..3.1 of IS : 875 -1987 part – 3

Design speed,

Vz = 50 × 0.9 × 1.124 ×1

= 50.58 m/sec

Design wind pressure, Pz = 0.6 V = 1535 N/m^2

Reduction factor = 0.7

Now, wind force on the top dome and cylindrical walls

$(4.5 + \frac{2.5}{2}) \times 15.6 \times 1535 \times 0.7 = 96383N$

Acting at 16.9 m above the base.

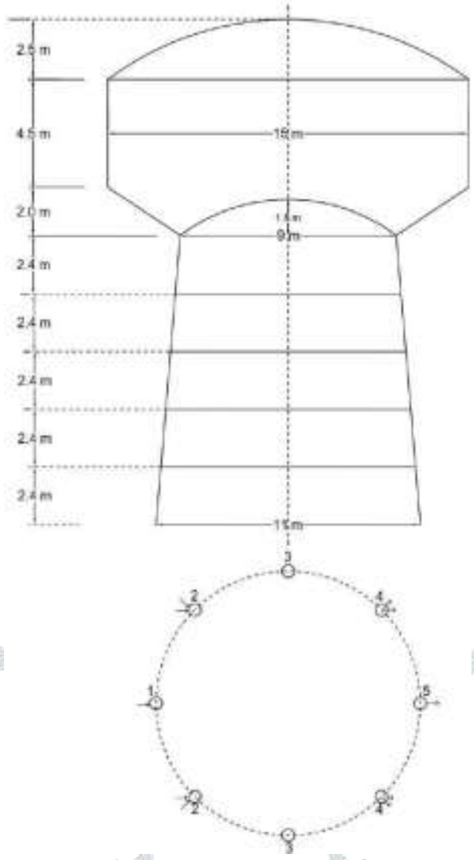


Fig no. 4.9: Analysis of wind force in the column

Wind force on the circular wall = $(\frac{15.6+10.5}{2}) \times 2 \times 1 \times 1535 \times 0.7$ = 28045N

Acting at 13.04 m above the base

Wind force on circular girder = 0.905 x 10.5x 1500x 0.7= 9978N

Acting at 12.04m above the base

Wind force on column and braces

= 0.7 × (5 × 0.5 × 12.04 + 7 × 10.50) × 1535 = 71831 N

Acting at 6.02 m above the base

Total moment of wind pressure about the base

= (96383 × 16.9) + (28045 × 13.04) + (9978 × 12.04) + (71831 × 6.02) = 2547137 Nm

Vertical load on any column due to wind load = $\frac{M.x}{\sum x^2}$ ➡ $\sum x^2 = 2r^2 + 4\,(r\sin(\frac{\pi}{4}))^2 = 2\times5.5^2 + 4\,(5.5\sin 45°)$

Where r = radius column circle = 5.5 m ➡ $\sum x^2$ = 121 m2

Maximum wind load force in most leeward and the most windward side = $\frac{2547137\times5.5}{121}$ = 115779N

Maximum wind load force in column marked 5 = $\frac{2547137 \times 5.5}{125}$ x $\frac{5.5}{\sqrt{2}}$ = 81868 N

Consider the windward column 1

Vertical load due to load and wind load = 1822694 + 115779 = 1938473 N

Corresponding axial load = $\frac{1938473}{0.996}$ = 1946258 N

Since the column are inclined the horizontal component of the axial force caused by wind action reduce the horizontal shear in column.

Horizontal component of axial force caused wind action

= 2 × 115779 × 0.0996 + 4 × 81868 × 0.0996 × $\frac{1}{\sqrt{2}}$ = 46127 N

∴ Actual horizontal force at the base

= (96383 + 28045 + 9978 + 71831) – 46127 = 160110 N

∴ Horizontal shear per column = $\frac{160110}{8}$ = 20013.75

∴ Maximum bending moment for the column = 20013.75 x $\frac{3.01}{2}$ = 30121 Nm

The effect of lateral forces due to seismic load is greater than those due to wind load; therefore seismic forces govern the design of staging.

4.12 ANALYSIS OF THE COLUMN SECTION

Radius of column circle = 0.25m

Axial force in column due to gravity load tank full = 15003700 N

Overturning moment when tank is full = 145871.4 × 16.07 = 2344154 Nm

Maximum axial force on the remotest column staging,

When tank is full = $\frac{150037000}{8} \pm \frac{2344154}{\sum(\text{X x X})}$ x R

Where , $\sum x^2$ = 2 x R² + 4 (R sin($\frac{\pi}{4}$))^2= 121 ➡ $\frac{150037000}{8} \pm \frac{2344154}{121}$ x 5.5 = 1982015 N

Maximum axial force = $\frac{1982015}{0.996}$ = 1989975 N

Provide 9 bars of 20 mm diameter, and an effective cover of 50 mm.

Area of steel, Ast = 9 × 314 = 2828 mm²

Equivalent concrete area = $\frac{\pi}{4}$ x 500² + (13.33 -1) x 2828 = 231219 mm²

Polar moment of inertia of equivalent concrete section = (πd4/64) + Ast (m-1) r2(n/2)

= (π× 5004)/64 + 2828× (13-33-1) × (250-50)2× (9/2)

= 9.34 × 109 mm4

Therefore, Equivalent moment of inertia about a diameter = 9.34× 109 mm4

Direct stress in concrete = $\frac{1989975}{231219}$ = 8.61 N/mm²

Bending stress in concrete = $\frac{219536.4}{9.34 \times 1000000000} \times 1000 \times 250 = 5.87$ N/mm²

Factored designed load, Pu = 1.5 × 1982015 = 2973022.5 N

Mu = 1.5 × 219536.4 = 329305 Nm

Dimensionless parameter for seismic forces,

$(Pu/fck.D^2) = \frac{2973022.50}{30 \times 50 \times 500} = 0.369$

$(Mu/fck.D^3) = \frac{329305}{30 \times 500 \times 500 \times 500} = 0.088$

From SP: 16, chart-55,

P/fck = 0.12 × 0.4 = 0.048

P = 0.048 × 30 = 1.44 %

Requirement of steel = $\frac{1.44}{100} \times \frac{\pi}{4} \times 500^2 = 2828$

Provide 9 bars of 20 mm diameter at an effective cover of 50 mm.

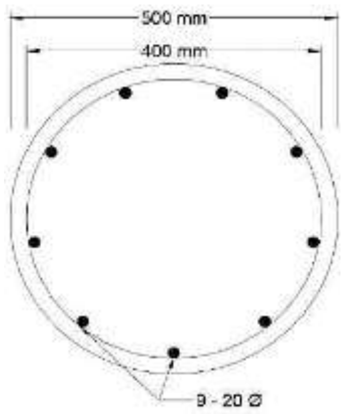


Figure 4.9: Details of column section

4.13 ANALYSIS OF THE COLUMN SECTION

Consider ore braces, say the base BC

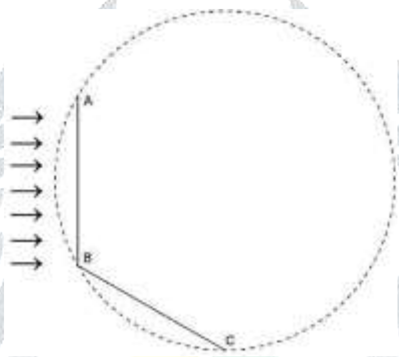


Fig no. 4.10: Wind pressure acting on brace AB

For the condition of maximum B.M. for the brace BC, seismic should act normal to an adjoining brace AB.

Moment in brace BC = moment for the column × (sec 45̊) = 219536.4 × √2 = 310471 Nm

Providing (300 ×500) mm section and designing as doubly reinforcement beam with equal steel at top and bottom,

Asc = Ast = $\frac{310471 \times 1000}{220 \times 420} = 3660$ mm²

Provide 6 bars of 20mm dia. at top and equal amount of steel at bottom.

Shear force for brace = $\frac{\text{B.M.for brace}}{0.5 \times \text{span of brace}} = \frac{310471}{0.5 \times 3.518} = 176504$ N

Therefore, Nominal shear stress, tv = S/bd = $\frac{176504}{300 \times 460} = 1.27$ N/mm² < 1.8 N/mm²

Hence the section is adequate with shear reinforcement.

Therefore, provide nominal stirrups; say 4-legged 8 mm φ stirrups @ 160 mm c/c.

4.14 DESIGN OF FOUNDATION

Total load on the columns when the tank is full = 1830014 × 8 = 14640112 N

Approximate weight of foundation (10% of column load) = 1464011.2 N Total loads = 16104123 N

Safe bearing capacity = 112.815 KN/m² ……………….. (Assumed)

Area of foundation = $\frac{16104123}{112815}$ = 142.75 m

Let us provide 16m outer dia. and 8m inner dia. for raft foundation.

Therefore, area of footing = $\frac{\pi}{4}(16^2 - 8^2)$ = 150.80 m²

Therefore, net upward pressure intensity = $\frac{14640112}{150.80}$ = 97085 N/m² = 97.085kN/m² < 112.815kN/m

4.14.1 DESIGN OF CIRCULAR GIRDER

Maximum negative B.M. at centre = 0.00416 × W × r = 0.00416 × 14640112 × 5.5 = 334966.5 Nm

Maximum positive B.M. at support = 0.0083 × W × r = 0.0083 × 14640112 × 5.5 = 668321.5 Nm

Maximum torsion = 0.0006 × W × r = 0.0006 × 14640112 × 5.5 = 4831.2 Nm

Maximum S.F. at support = $\frac{14640112}{2 \times 8}$ = 915007 N

4.14.2 DESIGN AT SUPPORT SECTION

Moment of resistance = Maximum bending moment at support

0.913 bd2 = 668321.5 ➡ bd2 = 732006 × 1000

So, b = 750 mm and d = 1000 mm. ➡ Effective depth = 1000 – 60 = 940 mm.

Effective Shear Force, $S_e = S + 1.6\frac{T}{b}$ = 915007 + 1.6$\times \frac{48312\times1000}{750}$ = 1018072.6N

Equivalent nominal shear stress, $q_{ve} = \frac{S_e}{bd} = \frac{1018072.6}{750\times940}$ =1.44N/mm^2

For M30 concrete, tc max = 2.2 N/m2 ……………………… (from IS 456:2000 table-24)

LONGITUDINAL REINFORCEMENT

Equivalent B.M. = Me1 = M + Mt

$M_t = T\times\frac{[1+\frac{D}{d}}{1.7} = \frac{48312000}{1.7}$ x $[1+\frac{1000}{750}]$= 66310588 Nm

Me_1 = 668321500 + 66310588 = 734632088 Nm

$A_{st} = \frac{734632088}{230\times0.9\times940}$ = 3775mm^2

So, provide 14 bars of 20 mm Ø (4400 mm^2)

Transverse Reinforcement

Distance between centers of corner bars parallel to the width = b1 = 750 – (2×40) = 670 mm

Distance between centers of corner bars parallel to the depth = d1 = 1000 – (2×40) = 920 mm

Area of section of stirrups = $A_W = \frac{TP}{b_1 \times d_1 \times tw} + \frac{SP}{2.5 \times d_1 \times tw}$

Providing 4-legged 10 mm Ø stirrups, Aw = 4 × 79 = 316 mm^2

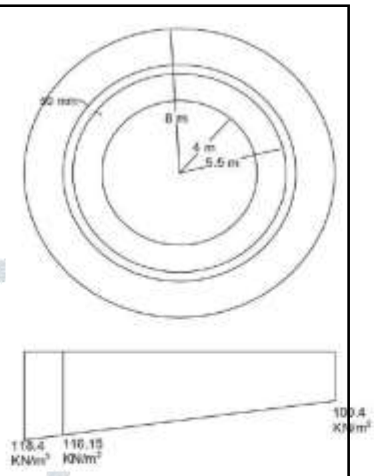


Fig no. 4.11: B.M acting on the support section

$316 = \left[\frac{48312000}{670 \times 920 \times 230} + \frac{915007}{2.5 \times 920 \times 230}\right] \times P$

316 = [0.34+1.73] P

P = 152.6mm. Say120 mm c/c

Steel for hagging moment = $\frac{334966.5 \times 1000}{230 \times 0.9 \times 940} = 1722$mm2

Provide 6 bars of 20 mm Ø (1885 mm2).

4.14.3 DESIGN OF BOTTOM SLAB

Cantilever projection beyond the face of the beam = $\frac{1.5-0.5}{2}$ = 0.5m

Maximum bending moment for 1m wide strip = 174.225 x $\frac{0.5 \times 0.5}{2}$ = 21778.125Nm

Equating the M.R. to the beam 0.913 × 1000 × d2 = 21778125 ➡ d = 154mm

Let us provide d = 200 mm.

So, effective depth = 200 – 40 = 160 mm ………………. (40 mm clear cover)

$A_{st} = \frac{21778125}{230 \, x \, 0.90 \, x \, 160}$ = 658mm^2

Spacing of 12 mm diameter bars = $\frac{113 \times 1000}{658}$ =172 mm …………………Say 150 mm c/c

Distribution of steel = $\frac{0.12}{100}$ 200×1000 =240mm^2 ; Provide 8 mm Ø bars @ 200 m centres.

Check for sliding

Total load on the foundation when the tank is empty = 14640112 – 94450 = 5195071 N

Horizontal force on the base = 1166971.3 N

Assuming co-efficient of friction of 0.5.

So, factor of safety against sliding = $\frac{0.5\times519571}{1166971.3}$ = 2.23 > 2.0 ………….. Hence, OK.

Check for overturning

Factor of safety against overturning = $\frac{5195071\times8}{1166971.3\times16.07}$ = 2.22 > 2.0 ………..Hence, OK.

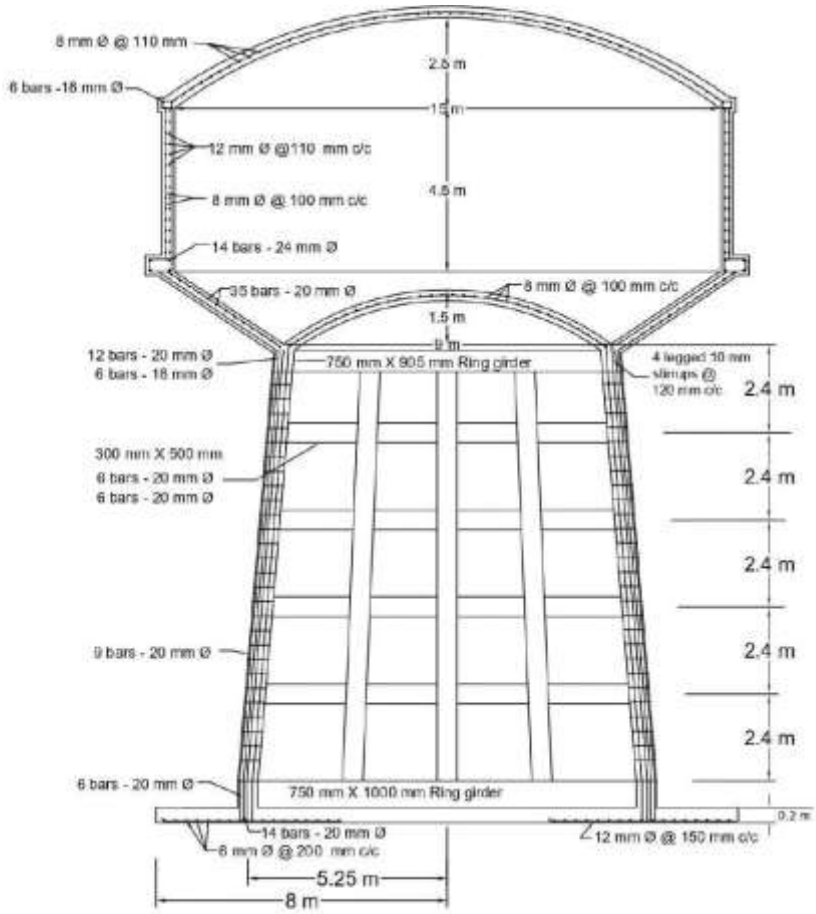


Figure 4.12: Details of reinforcement in Intze tank

V DESIGN OF INTZE TANK USING STADD PRO SOFTWARE

STADD.Pro is one of the most widely used structural analysis and design software products worldwide developed by Bentley

5.1 DESIGN PROCEDURE

Open STAAD.pro

1) Click on new project > add file name > Select 'space' > Length (in m), Force (in KN) > Select add beam option and click on finish.

2) **For elevated frame** → Go to > Geometry > Run structure wizard > select Surface/plate model > cylindrical surface > Close it to transfer to modelling Length: 12m, Division along length: 5 Nos., Start radius: 11.0m, Division along periphery: 8(column) End radius: 9.0m.

3) Using Add beam selecting top node and bottom node. Repeat along periphery for required number of columns.

4) Copy all vertical members using ctrl + C and paste aside using ctrl + V.

5) Add intermediate nodes along length to add required number of beams in horizontal direction. Connect all node in a plane to form a circular beam.

6) Repeat the same process at top to get circular girder.

7) **For bottom dome** → Geometry > Run structure wizard > select surface/plate model > Spherical cube >

Select spherical cap (Bottom dome). Close it to transfer to modelling, Diameter of sphere: 15, Base Diameter: 9. (Rise of Bottom dome = 1.50m)

8) Shift the obtained conical dome to top of bottom ring beam, Measure distance using 'display node to node distance' tool. Select all plates > Right click mouse >Move > add (-) sign to above distance to rest on top beam.

9) **For Conical Slab** → Geometry > Run structure wizard > select surface/plate model > cylindrical surface Length: 3.6055m; Division along length: 1; Start radius: 9m; Division along periphery: 8(column) End radius: 15m.

10) Shift the obtained conical slab to top beam measure distance using 'display node to node distance' tool. Select all plates > Right click mouse > Move > add (-) sign to above distance to rest on top beam.

11) **For Cylindrical wall** → Geometry > Run structure wizard > select surface/plate model > cylindrical surface Length: 4.5m; Division along length: 1; Start radius: 15m; Division along periphery: 8; End radius: 15m.

12) Shift the obtained cylindrical wall to top of conical slab measure distance using 'display node to node distance' tool. Select all plates > Right click mouse > Move > add (-) sign to above distance to rest on top beam.

13) **For top dome** → Geometry > Run structure wizard > select surface/plate model > Spherical cube > Selects spherical cap (Bottom dome). Close it to transfer to modelling, Diameter of sphere: 25, Base Diameter: 15. (Rise of Bottom dome = 2.50m)

14) **Support** → General > Support > create > Fixed > Add > Assign of selected nodes

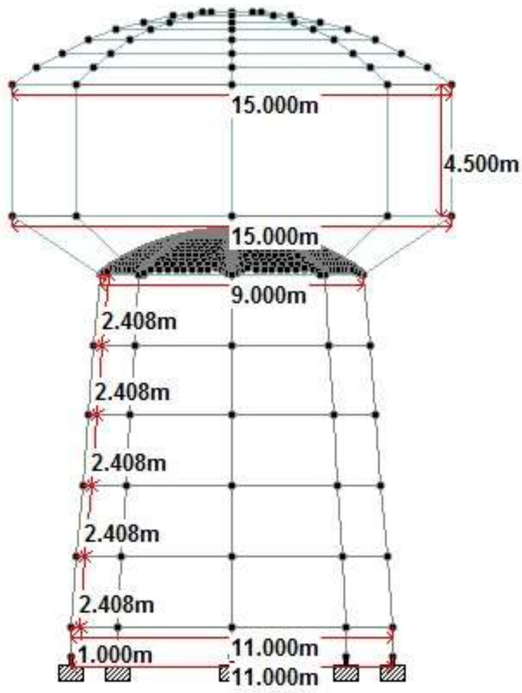


Figure 5.1: Intze water tank model

5.2 GEOMTRY OF STRUCTURE

Go to → GENERAL > Property > define > define the physical properties of all components: -

- Thickness of top dome = 0.15 m
- Thickness of Cylindrical wall = 0.30m
- Bottom ring beam : Depth = 0.95m ; Width = 0.75m
- Thickness of conical slab = 0.30m
- Thickness of bottom slab = 0.30m
- Circular Girder = 0.90m x 0.75m
- Diameter of circular column = 0.50m
- Support section braces: Depth = 0.30m ; Width = 0.050m
- Bottom circular girder: Depth = 1.0m; width = 0.75m.

Assign all the properties component wise.

Go to > view 3D > view 3D structure

Figure 5.2: Intze tank 3D model

5.3 LOAD APPLICATION

Go to → GENERAL > Load & defination > define > define the physical properties of all components: -

1) Application of dead load:

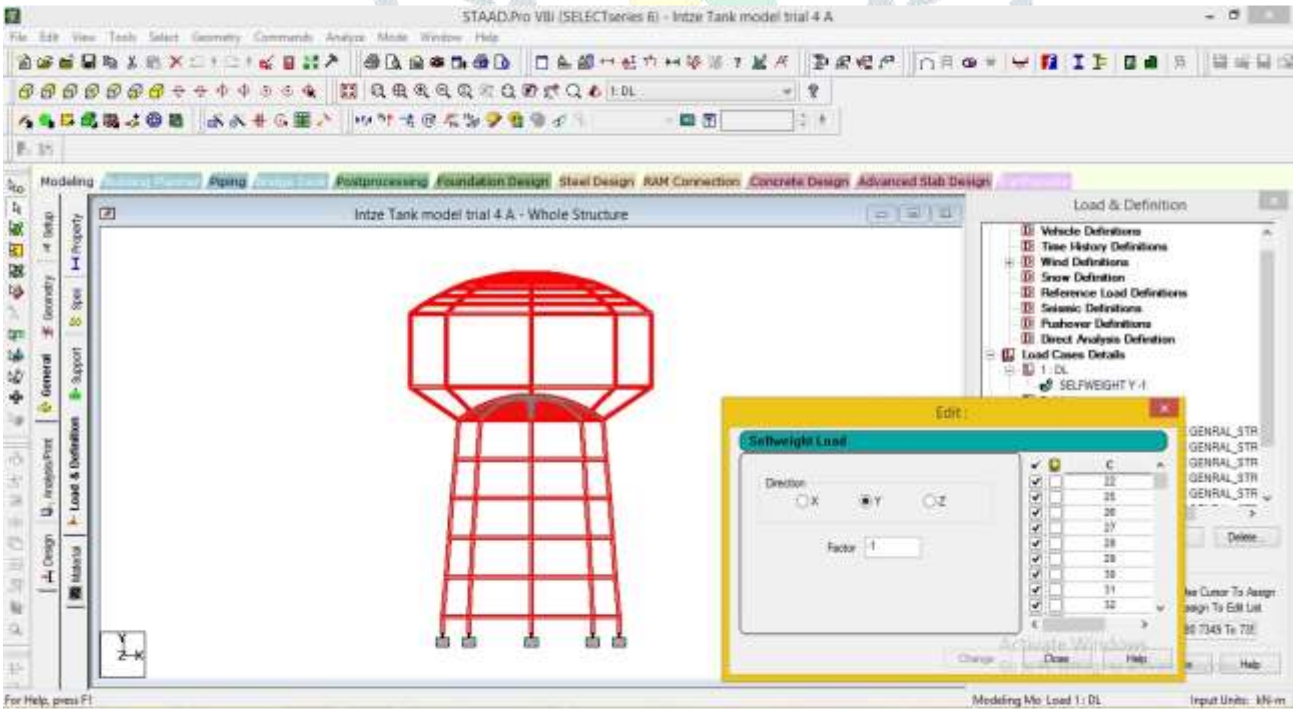


Figure 5.3: Application of Dead load

2) Application of live load i.e. hydrostatic pressure (When tank is full):

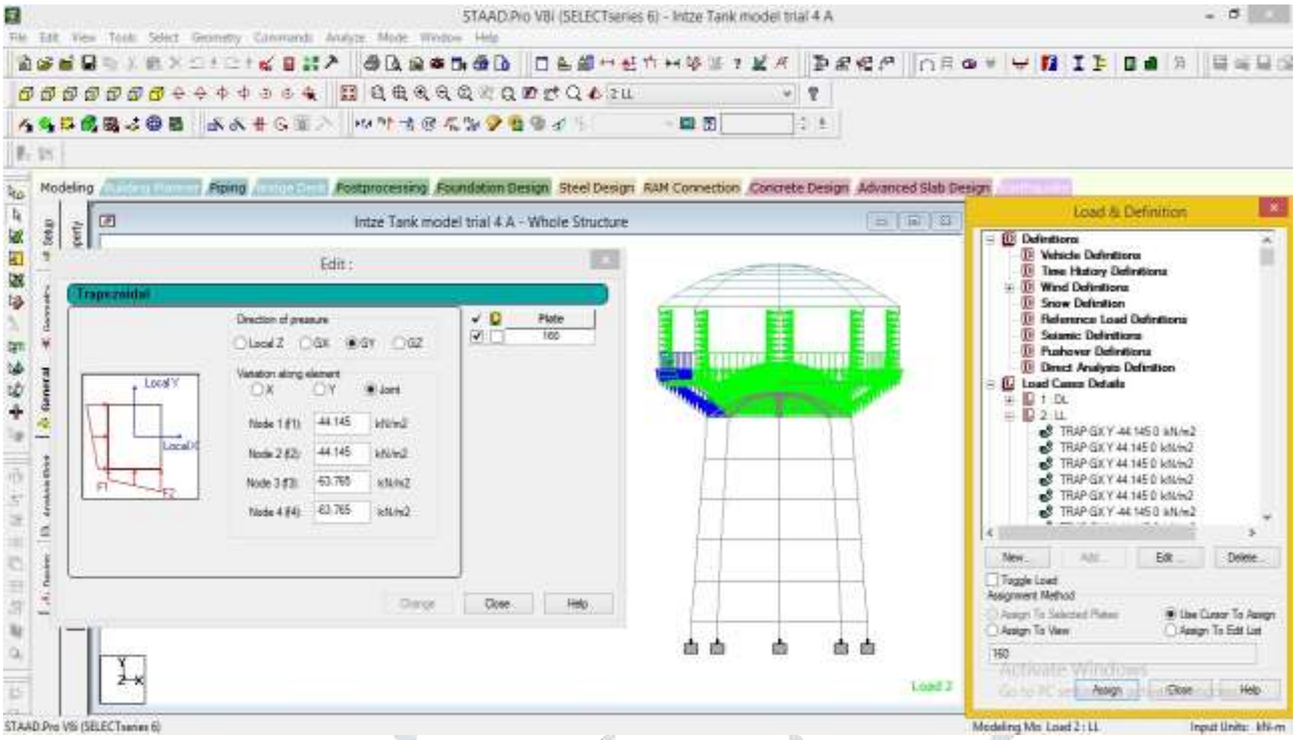


Figure 5.4: Application of Hydrostatic pressure (Live load)

3) Application of Wind load i.e. hydrostatic pressure (When tank is full):

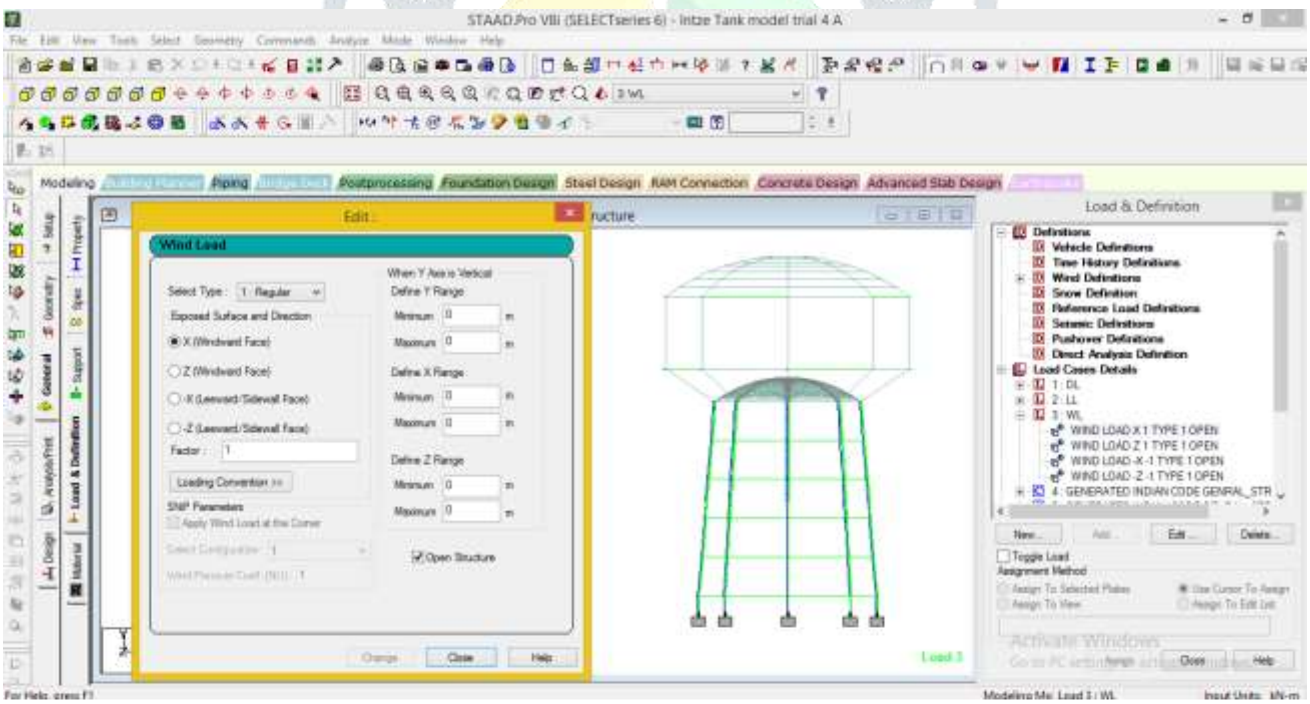


Figure 5.5: Application of Wind load

5.4 DESIGN

- Go to → DESIGN > Concrete > Current code: IS 456 > Define parameters > Select required parameters and insert values

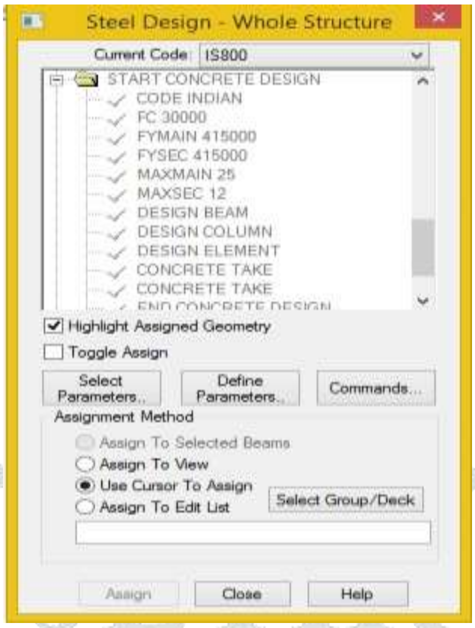


Figure 5.6: Showing applied design parameters

- After applying all the parameters, Go to → Load & definition > Load case details > Add > Auto load combination > Select load combination code: Indian Code > Select load combination category: General Select all load combinations > Add > Close.

5.5 ANALYSIS

Go to → Analysis/Print > Perform Analysis > All > Add > Analyze > Run analysis > Look for 0 Errors & 0 Warnings > go to post processing mode > done > Apply all > Ok

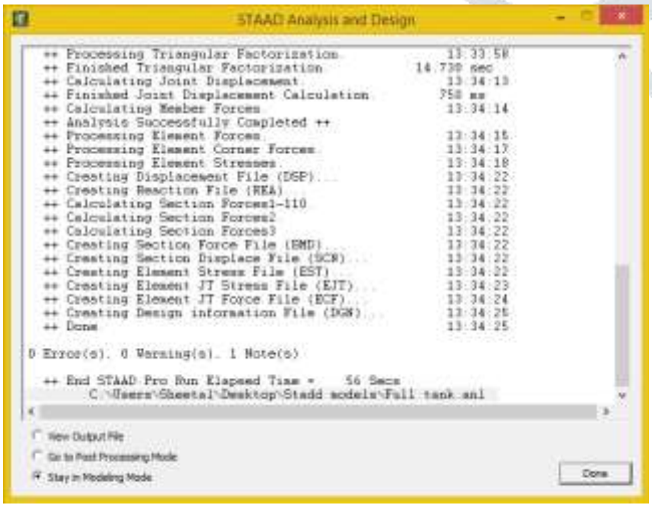


Figure 5.7: Stadd analysis when tank is full

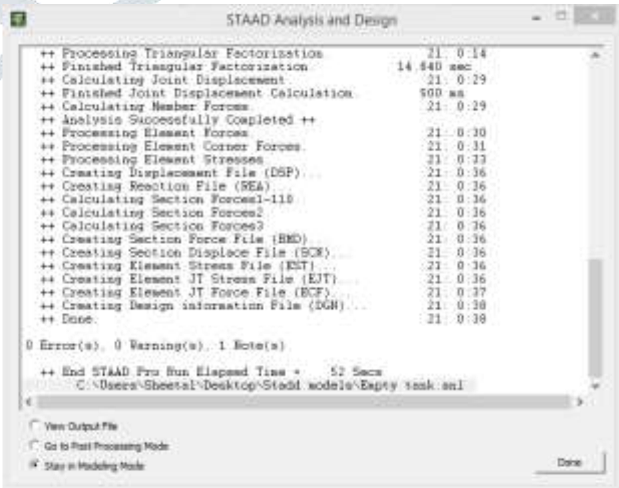


Figure 5.8: Stadd analysis when tank is empty

6 STADD PRO RESULT & OUTPUT

1) Result showing stresses on plate in different condition:

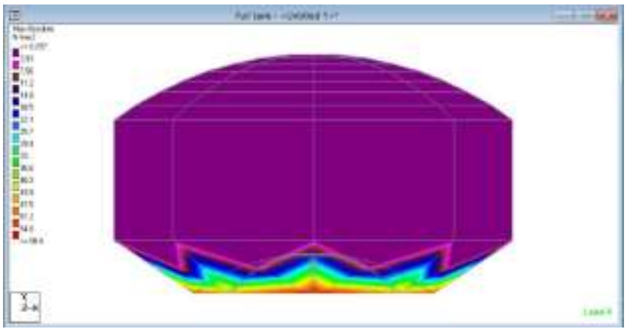


Figure 5.9: Maximum absolute & principal stress when tank is full

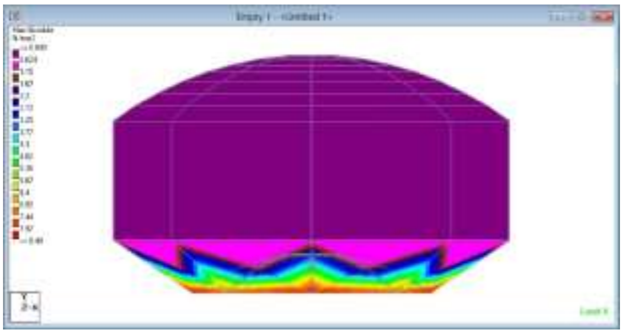


Figure 5.9: Maximum absolute & principal stress when tank is empty

2) Result showing end stresses on beam in different condition:

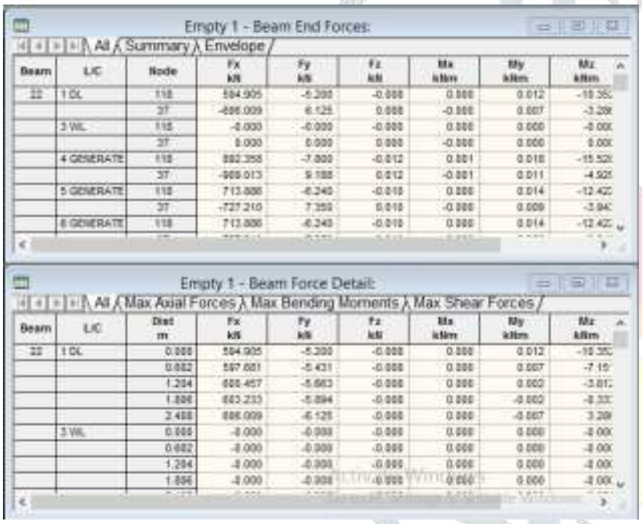


Figure 5.11: Beam end forces when tank is full

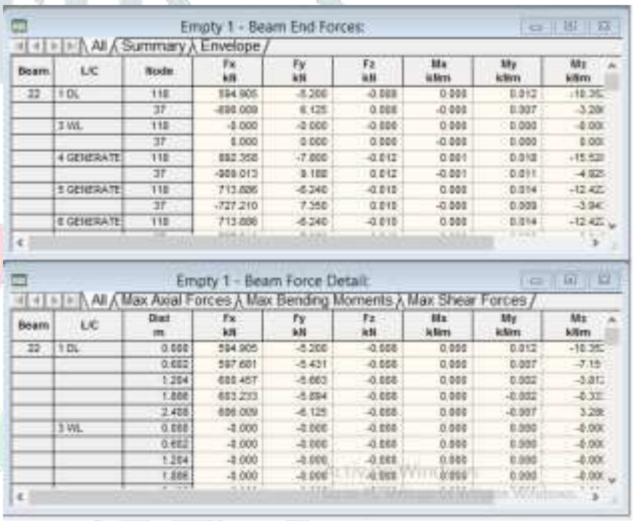


Figure 5.12: Beam end forces when tank is empty

4) Steel reinforcement details generated by STADD Pro. :

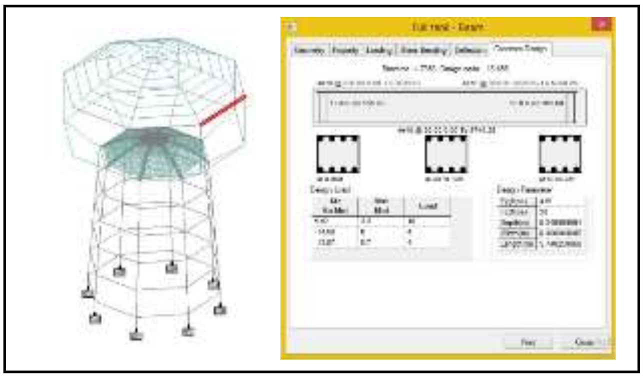


Figure 5.11: Reinforcement details of top ring beam

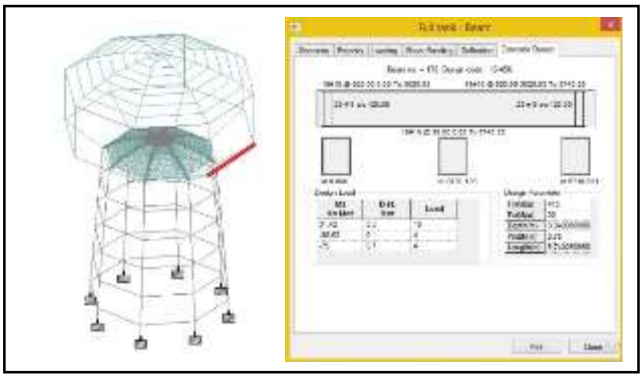


Figure 5.11: Reinforcement details of bottom ring beam

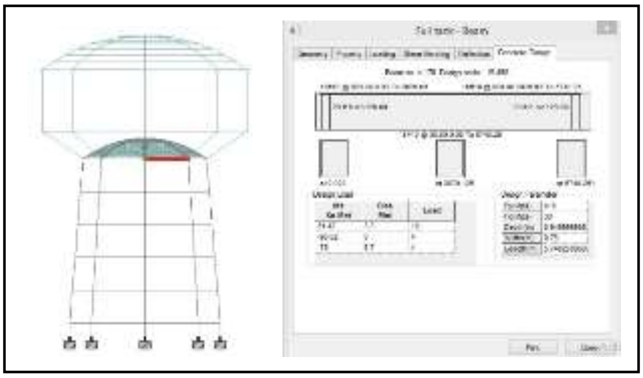


Figure 5.11: Reinforcement details of top circular girder

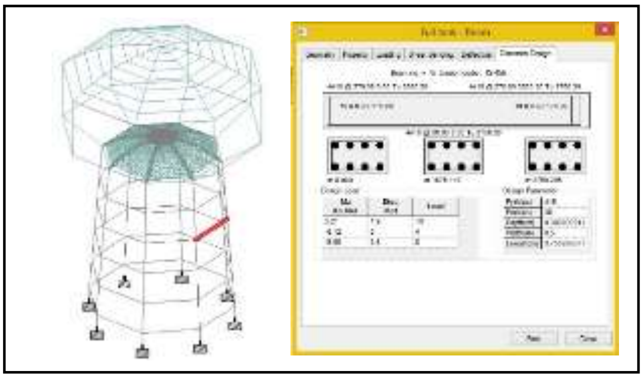


Figure 5.11: Reinforcement details of support brace

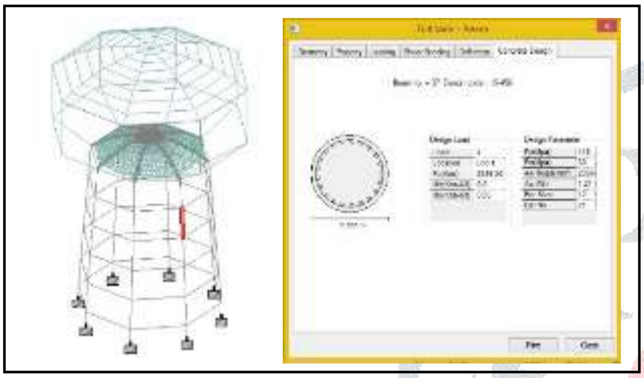


Figure 5.11: Reinforcement details of column when tank is full

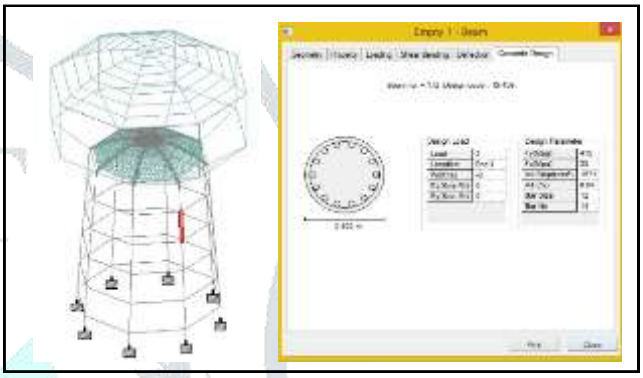


Figure 5.11: Reinforcement details of column when tank is empty

- It is observed that only at support columns, different reinforcement results has been given by STADD analysis in both conditions i.e. when tank is full & when tank is empty.

- Maximum absolute pressure observed at plates when tank is in full condition ranges minimum 0.130 N/mm² & maximum 58.40 N/mm². And when tank is in empty condition 0.099 N/mm² to 8.49 N/mm².

VI CONCLUSION

- An Intze water is designed with 90 KL capacity with 12m staging has designed with M30 grade of concrete.
- We design the tank by both manually and using STAAD Pro, the program results shown that design is safe
- After completion of Intze water tank design in STAAD Pro and from manual calculations we conclude that design is safe.
- Though design is safe but we observed that reinforcement is less when compared with manual calculations.
- There is an increase in moment when the height of the structure increases.
- STAAD Pro gives more accurate, economical & fast result as that of manual calculations.
- When using fix joint at the base its remarkable reduction in base settlement.
- This type tank is simplest form as compare to the circular tank.

- We have given the inclination to the staging of water tank because as respected inclination the tank performs better than that type of straight one.
- The staging has been designed with maximum safety and effects due to seismic force and wind force are also taken into account.

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