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Response of Intze Tank against Static and Earthquake Loads Using Staad Pro

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Abstract:- Water tanks are the largest utility structures in both public and industrial purposes. This project aims to analyze the response of an intze tank structure subjected to both static and earthquake loads using STAAD Pro software. Intze tanks are commonly used for water storage and distribution in various industries and municipalities. Understanding their structural behavior under different loading conditions is crucial for ensuring their stability and safety. The water tanks are the long life structures. Based on the studies India is the 60% is prone to earthquakes. But many of the structures are designed using the wind force as a lateral load but sometimes the seismic force also leads to collapse of a structure. By the consideration of combination of loads of live load, dead load and wind load or seismic load by using the code of practice IS 875 (part-v). So in this project we manually design a intze tank of 1000000 liters capacity using the codes of IS 456-2000, IS 3370 and will be statically checked using the code of practice of IS 875 part-v and IS 1893 part-II (2016). After the comparison of the manual design, these design parameters will be applied in STAAD Pro software and comparison will be done with manual design and software design.

Keywords:- *Intze Water Tank, STAAD Pro, Seismic Load And I.S Codes Etc.*

I. INTRODUCTION

A. *General*

Generally The Tanks Are Designed to Store the Liquids like Water, Oil, Petroleum etc. Tanks are mainly classified by the material used for the construction is R.C.C tanks and steel tanks. The steel tanks construction is very costly compared to reinforced cement concrete tanks. The cement concrete tanks construction is economical and its maintenance cost is low compared to steel tanks. In the construction of R.C.C tanks the richer mix usage made water tight. The richer mix means which have the mix proportion of not less than M_{20}.

In some cases for water tightening the water proof materials are also used for construction of tanks. For reduction shrinkage loss in water tanks construction use the cement content ranging from 330 Kg/m^3 to 530 Kg/m^3.and for attaining good strength in structure usage of steel is recommended higher than Fe 415.in this water retaining structures 0.1 mm crack is permissible. For the design of liquid retaining structures the code of practice recommended was IS 3370(part I-V). The storage tanks are mainly based on two types which was discussed below.

B. *Classification of R.C.C Water Tank*

➢ *Classification based on Position of the Heads:*

- Ground resting tanks
- Under ground
- Elevated

➢ *Classification based on the Shape of Heads:*

- Circular
- Rectangular
- Intze

We consider the elevated intze tank for the design and comparison with the STAAD pro software.

II. LITERATURE REVIEW

S. Deepika, Gugulothu. Swarna, "Design And Analysis of Intze Type Water Tank For Different Wind Speed And Seismic Zones As Per Indian Codes ", International Journal of Advanced Technology in Engineering and science, This project deals with the design and the analysis of intze type tank for the different wind speed and seismic zones as per Indian codes. The design of water tanks are subjected to live load, dead load and wind or seismic load as per IS codes of practices. The seismic load or earthquake load is also called as unstable load.

Issar Kapadia, PuravPatel, Nilesh Dholiya, Nikunj Patel "Analysis and Design of INTZE Type Overhead Water Tank under the Hydrostatic Pressure as Per IS: 3370 & IS: 456 -2000 by Using STAAD Pro Software", Water tanks are the important public utility, industrial and storage structure. The design and construction methods are followed by the size of the tank and staging pattern of the tank.

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III. DESIGN STEPS FOR THE INTZE TANK

- Design of Top Dome & Top Ring Beam
- Design of Cylindrical wall
- Design of ring beam at the junction of the cylindrical wall and conical dome
- Design of Conical Dome & Bottom Spherical Dome.
- Design of bottom ring beam
- Design of supporting structure, i.e., Staging
  - ✓ Analysis of wind force.
  - ✓ Analysis of lateral force.
- Design of Foundation.

A. Analysis of Wind Forces

The wind analysis of this structures are based on the code of IS: 875 Part III.The intze tanks are offer relatively less reduction factor of 0.7 to arrive at effective pressure. The basic formula used for the wind analysis is as per code is followed by

$$\mathbf{V_Z=V_bK_1K_2K_3}$$

Where,
V_z = Design wind speed at any height of the structure
V_b = Basic wind speed in m/sec
K_1 = Risk coefficient
K_2 = Structure factor and terrain height
K_3 = Topography factor

For chosen direction of wind, the maximum shear force occurs in a brace beam connecting a column, while the maximum bending moment occurs in a brace connecting a column, while maximum bending moment occurs in a brace. For the consideration of wind or seismic force for the design of tank is depending upon the tank location.

B. Analysis of Lateral Force:

Housner (1963) proposed a method of two mass model system for the lateral forces of elevated tank and it is used commonly used method in most of the codes. Pressure produced by the fluid in tank is divided into two separate masses they are;

- Impulsive
- Convective

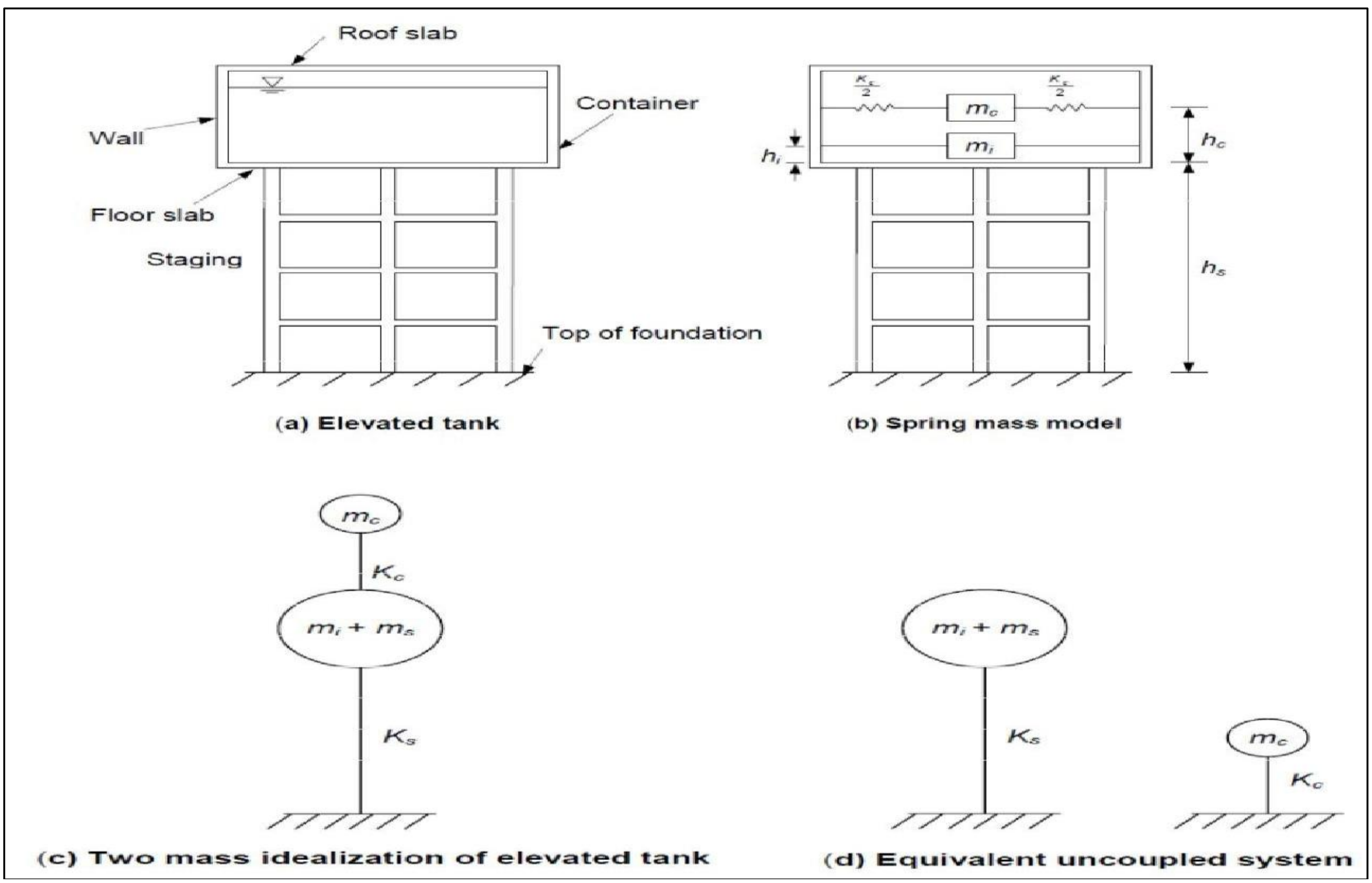

Fig 1: Idealization of Two Mass Model of the Elevated Tank

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➢ *Time Period:*

- *Impulsive Mode:*

Impulsive mode time period (T)
$T_i=2\pi\sqrt{(mi+mc)/Ks}$

Where:
mi = the impulsive mass
ms = mass of the staging
Ks = stiffness of the lateral staging

- *Convective Mode:*

The mode of convective time period, in seconds is given by

$T_c= 2\pi\sqrt{mc/Kc}$

Where:
The convective mode of the time period for the circular and rectangular tanks is taken from IS 1893 (part-II)

$T_C=C\sqrt{\frac{D}{g}}$

Where:
C_c= Coefficient of time period (T_c) for the convective mode. (Obtained from fig 2(b) of IS 1983 part-2)
D = diameter of tank (inner).

➢ *Damping:*

Damping is taken for all types of tanks in convective mode shall be taken as 0.5% of the critical. Damping in convective mode is 5% and in impulsive mode is 2%.

- Seismic zone factor, Z as per IS 1893 (Part 1): 2002 India has been divided into IV seismic zones. And the different zones are having different zone factors.
- Importance factor, (I) this factor is decided by the function of the structure. Mostly for water retaining structures I value is taken as 1.5 as per IS 1893(part-I)
- Response reduction factor, R This factor is depends on the perceived seismic damage performance of the structure Response reduction factor depends on the perceived as per code this value is taken as 2.5 (for SMRF).
- Structural response factor, (Sa/g) Design horizontal seismic coefficient (A_h)
- $A_h = (Z/2) \times (I/R) \times (S_a/g)$

Where
Z=zone factor
R= response reduction factor
I = importance factor
S_a/g= average acceleration coefficient

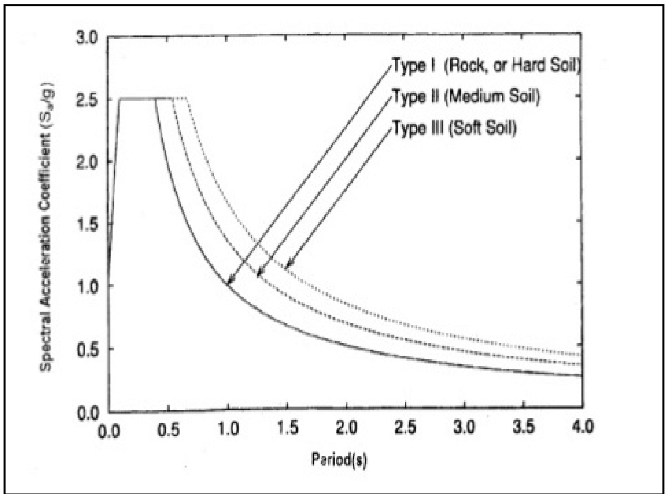

Fig 2: 5% Damping Response Spectra

➢ *Base Shear*

$V_i= (A_h)\ (m_i+m_s)g$ …………impulsive mode
$V_c= (A_h)\ (m_c)g$…………convective mode
Total base shear $V= \sqrt{Vi^2+Vc^2}$

Where:
m_s= (1/3) mass of the staging + mass of the container .
V= total base shear.
V_i= base shear in impulsive mode.
V_C= base shear in convective mode.

IV. METHODOLOGY

➢ *Calculation of Population*

- Total Population = 7000
- Per Capita Demand of 135 Liters/day (AS Per 1172-1971)
- Capacity of tank = (7000 × 135) = 945000 Liters
- Design Capacity of tank = 1000000 Liters

➢ *Tank Dimension*

- Grade of Steel =Fe 415
- Grade of Concrete = M30
- Tank Diameter (D) = 15 m
- Lower Ring Beam diameter (D0) = 15 × 0.6 = 9m
- Rise of the top dome (h1) = 3.0m
- Rise of the bottom dome (h2) = 2.0m
- Conical dome height (h0) = 2.5m
- Height of the cylindrical portion:
- Tank Capacity:

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$$= \frac{\pi}{4} \times (D^2 \times h) + \frac{\pi}{12} \times h_0\left(D^2 + {D_0}^2 + D \times D_0\right) - \frac{\pi}{3} \times {h_2}^2 (3R_2 - h_2)$$

$$R_2 = \frac{(\frac{D}{2})^2 + {h_2}^2}{2h_2}$$

R_2 = 6.0625 m

1000 m^3= ($\frac{\pi}{4}$× 15² × h)+$\frac{\pi}{12}$× 2 × (15²+9²+ 15× 9) -$\frac{\pi}{3}$×1.5²× (3×7.25 – 1.5)⇒ h = 4.28 m

Say , h = 4.5 m

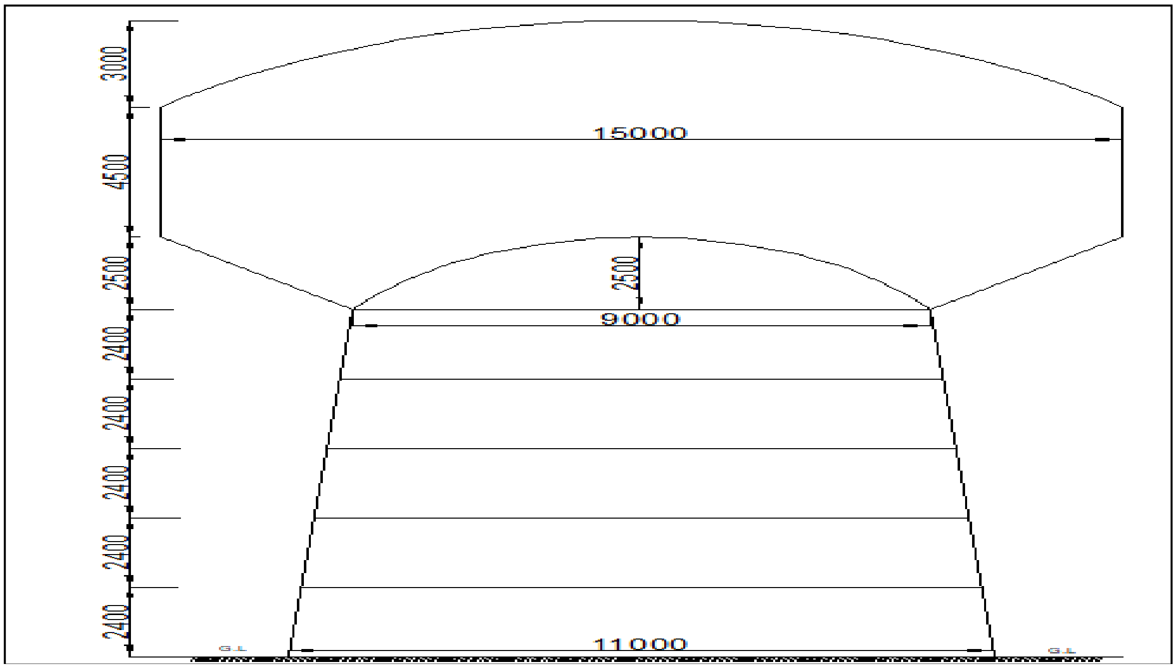

Fig 3: Water Tank Dimension

➢ *Top Dome Design*

Provide 150 mm thickness for the roof dome.

The angle subtended by dome is 2θ of its centre

∴ Sinθ =$\frac{d}{2R_1}$

R_1 = $\frac{(\frac{15}{2})^2 + 3^2}{2 \times 3}$

R_1 = 10.875 m

$$\theta = 43.6°$$

➢ *Loads Calculation*

Total Dead load = 0.15 × 25000

= 3750 N/ m^2

Dome Live load = 0.75 – 0.52 y^2(from Table 2, IS : 875 part – 2)

Y = $\frac{h1}{D}$ = $\frac{3.0}{15}$ = 0.2

Live load = 729.2 N/m^2

Total load (w) = 4.4792 KN/m^2

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- *Hoop Stress Calculation at the Level of Springing*

$$f = \frac{WR_1}{t}\left(\cos\theta - \frac{1}{1+\cos\theta}\right)$$

$$f = \frac{4485\times 10.875}{.15}(\cos 43.60 - \frac{1}{1+\cos 43.60})$$

f = 0.047 N/mm^2 < 5 (As per IS 875 code for M30 Grade)

- *Hoop Stress at the Level of Crown*

i.e , at $\theta = 0^0$

$$f = \frac{WR_1}{t}(1 - \frac{1}{1})$$

f = 0.188 N/mm^2 < 5 (As per IS 875 code for M_{30} Grade)

Meridional thrust of the springing, per meter run:

$$T_1 = \frac{WR_1}{1+\cos\theta}$$

T_1 = 28.25558KN/m

∴ Meridional stress = $\frac{28288.5}{150\times 1000}$

=0.188 N/mm^2

Provide nominal reinforcement because the stresses are very small

∴ nominal reinforcement (0.3%)

Ast = $\frac{.30}{100}$ × 1000 × 150 = 450 mm^2

∴ $Provide$ 8mm Ø @ 110mm c/c

- *Top Ring Beam*

Horizontal component of T1

W = T1 Cos θ

= 28.25558× cos 43.6

W= 20.4658 KN

Ring beam Hoop tension = 20.4658 × $\frac{15}{2}$

T = 153.453.5 KN

∴ Steel required for hoop tension = $\frac{153643.5}{150}$ = 1023.28mm^2

∴ Provide 6 bars 16 mm diameter (1206.37 mm^2 > 1023 mm^2)

- *Size of the Ring Beam*

The area of the ring beam = A mm^2

Equivalent concrete area = A + (m-1) x Ast (IS : 456 -2000, Cl B-1.3 page no.80)

A + (m − 1)Ast = A + (9.33 – 1) × 1130 = A +9412.9 mm^2

Limiting tensile stress of the equivalent concrete area to 1. 5 N/mm^2

$\sigma_{ct} = \frac{T}{A + (m-1)\,Ast}$ ∴ σ_{ct} = 1.5 (IS : 456 _ 2000 , Cl B-1.3 page no.80)

Area = 93778.41 mm^2

Provide 250 × 400 mm section

- *Shear Reinforcement in the Ring Beam*

Provide 2 legged stirrups 8 φ (IS: 456 – 2000, Clause no – 26 .5.1.6)

0.7 D + (0.75 × 400) = 300

2) 300 mm

3) Sv = $\frac{0.87 \times fy \times Asv}{0.4 \times b}$

Asv = 2 × $\frac{\pi}{4}$× 8^2 = 100.54 mm^2

Sv = 262.89 mm

Provide Sv =250< 262.89 mm

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

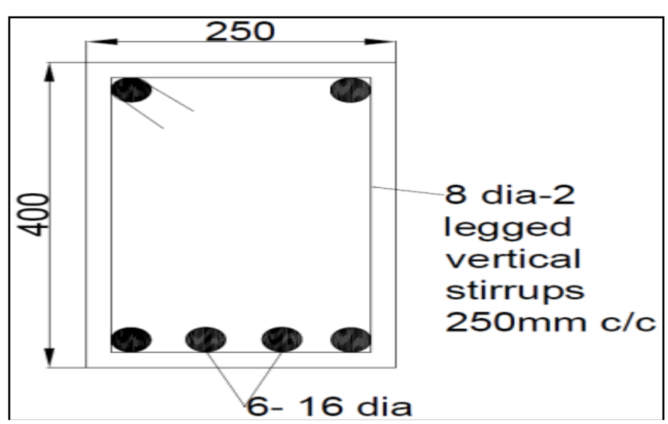

Fig 4: Top Ring Beam Detailing

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➢ *Cylindrical Wall*

Intensity of Pressure at the bottom of cylindrical wall =(4.5 × 10000) = 45000 N/m^2 (γ_w x h)

Consider 1m bottom strip of the wall.

Hoop tension = $\frac{Pd}{2}$ = 337.500 KN

∴ Ast = $\frac{337500}{150}$ = 2250 mm^2

Assume 12 mm diameter bars

Spacing = $\frac{\frac{\pi}{4}\times 12 \text{ x } 12 \times 1000}{2250}$ = 50.26 mm

∴Provide 12 mm dia bars @ 100 mm c/c near each face of cylindrical wall

Thickness of wall

Ct = $\frac{T}{(1000\,t)+(m-1)Ast}$N/ mm^2< 1.5 (As per IS 875 code for M_{30} Grade)

T = $\frac{\gamma w \times h \times D}{2}$ =$\frac{10\times4.5\times15}{2}$ = 337.5 KN

∴ Ast = $\frac{\pi}{4}\times 12^2 \times 110$ = 1028 mm^2

∴ 216.43 < t

∴ Provide thickness t = 300 mm

Thickness of the wall kept as 300 mm.

Steel in Distribution =$\frac{0.3}{100}\times(300\times1000)$ = 900 mm^2 (0.3% of gross area)

Spacing of 8 mm φ bars = = $\frac{\frac{\pi}{4}\times 8 \text{ x } 8\times1000}{900}$ = 55.85 = 55 mm

∴Provide 8 mm dia bars @ 110 mm c/c near each face.

Check for compressive stress @ the bottom of cylindrical wall.

Vertical component T1 = V1

T1 sin θ = 28255.58 × sin 43.6^0

= 19.4813 KN/m

Wall Weight = 0.3 × 4.5 × 25000 = 33.750 KN/m

Ring beam weight = 0.25 × 0.4 ×25000 = 25.000 KN/m

Total vertical load per meter (V2)= 19508.3 + 33750 + 2500

= 55731.3 N/m

Compressive stress = $\frac{55731.3}{300\times1000}$ = 0.185 N/mm^2

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

∴ Vertical steel = $\frac{0.3}{100}(300\times1000)$ = 900 mm^2

Spacing of 8 mm φ bars = $\frac{\frac{\pi}{4}\times 8^2\times1000}{900}$

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

➢ *Ring Beam Near Bottom Dome*

Let T2 is to be the thrust/meter run exerted at the junction B by the conical slab.

Resolving B vertically,

T2 Sin α = V2 = 55.7313 KN/m.

Tan α = $\frac{2.5}{3}$ = 0.84

α = 39.80^0

T2 = 55731.3 N/m.

Horizontally Resolving at B,

$H_2 = T_2 \cos\alpha = V_2 \cot\alpha$

= $\frac{55731.3}{0.833}$ = 66.90396 KN/m

The hoop tension @ ring beam B is produced by the horizontal load H2

Hoop tension due to H2 = $H_2\times\frac{d}{2}$ = 66903.96 $\times\frac{15}{2}$ = 501779.7N

Assume ring beam depth as 1000mm.

Water pressure in the ring beam = (1000 × 4.5) ×$\frac{900}{1000}$ = 40.500 KN/m

By the water pressure the hoop tension = 40.500 × $\frac{15}{2}$ = 303.750 KN

Total hoop tension = 501.779 + 303.750 = 805.529 KN

Steel for hoop tension = $\frac{805529.7}{150}$ = 5370.198 mm^2

So provide 25 mm φ bars and the number of bars required =

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$\frac{5370.198}{\frac{\pi}{4}\times 25^2} = 14$

Provide 14 bars of 25 mm dia.

Assume the area of ring beam is A.

Equivalent of concrete area = (A) + (m-1) × (A_{st}) = A + 57245.42

Limiting tensile stress on equivalent concrete area is to 1.5 N/mm^2 = $\frac{805529.7}{A+57245.42} = 1.5$

A= 479774.38 mm^2

∴ Provide,650 mm × 900 mm section

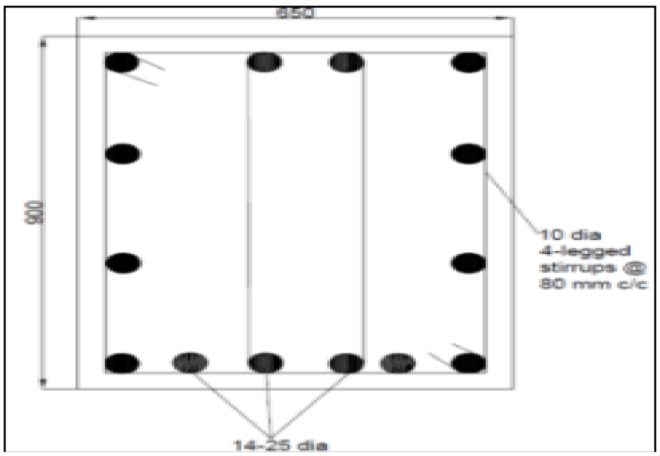

Fig 5: Detailing of Ring beam at bottom dome

- *Conical Slab Design*

The Conical slab is to be designed for the two conditions mentioned below:

- Hoop tension
- The span bending on a sloping slab from the ring beam @ B at the ring girder @ C.

- *Design for the Hoop Tension*

The hoop tension is given by on the conical slab = $\frac{Ww+Ws}{2\pi} + \frac{Ww}{2\pi}\tan\alpha$

Where,
Ww = the weight of water which is resting on the conical slab.
Ws = conical slab weight.
α = inclination of the conical slab compared to the horizontal.

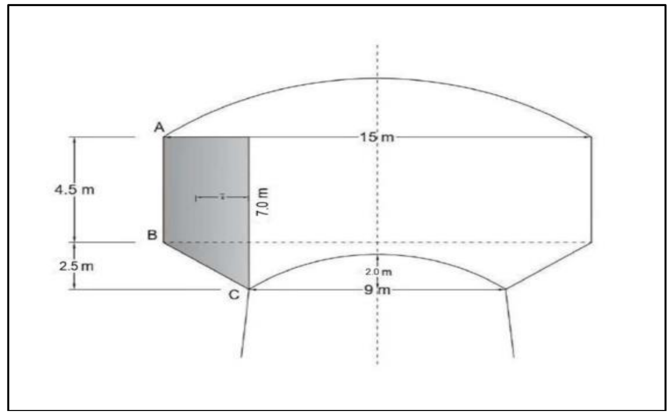

Fig 6: Dome Section Design of Tank

Water Section Area Standing on the conical slab

$$= \frac{3}{2}(4.5 + 7.0) = 17.25\ m^2$$

$$\overline{x} = \frac{(4.5\times 3)+(\frac{3\times 2}{6})}{17.25} = 0.9 \text{ m}$$

at the conical slab the weight of water resting on it is

$$W_w = \gamma_w \times A \times 2\pi \times ((9/2) + \overline{x})$$

$$W_w = 1000 \times 17.25 \times 2\pi \times (\frac{9}{2} + 0.9)$$

$$Ww = 5852.787 \text{ KN}$$

$$\text{Length of sloping slab} = \sqrt{3^2 + 2.5^2}$$

$$= 3.90 \text{ m}$$

sloping slab thickness = 300 mm.

$$\text{Weight of the conical slab (Ws)} = (3.9 \times 0.3 \times 25000 \times 2\pi) \times \frac{(\frac{15}{2}+\frac{9}{2})}{2} = 1102.6992 \text{ KN}$$

$$\therefore \text{Hoop tension} = \frac{5852787.1+1102699.021}{2\pi} + \frac{5852787.1}{2\pi} \times \frac{2.5}{3}$$

$$= 1756.698 \text{ KN}$$

$$\therefore \text{The hoop steel in the entire section} = \frac{175669.698}{150} = 11713.32\ mm^2$$

Provide 35 no. of bars of 20 mm diameter

These total bars are distributed at both faces of conical slab.

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➢ *Design for the Bending Moment*

Load per one meter width of the conical slab is

$= \frac{(\text{Ww+ Ws})}{(2\,\pi \times \text{mean radian})}$

$= \frac{5852787+1102699.021}{2\pi\times 6}$

= 183.704 KN

Maximum bending moment

$= \frac{wl^2}{8} = \frac{183704\times 3\ x\ 3}{8}$ = 206.667 KN-m.

Axial compression $T_2 = \frac{V_2}{\sin\alpha} = \frac{55731}{\sin 39.80}$

$$T_2 = 87.064\ KN$$

Provide 20 mm dia bars @ clear covers of the spacing 25 mm.

The effective depth = (300–25–10) = 265 mm.

Distance between the centre of section and the centre of steel

X = (d - $\frac{t}{2}$)= 265 – 150

X = 115 mm

The resultant bending moment = (M + T2 X)

$= 206.66 \times 10^3 + (87.064\text{x}10^3 \times 115)$

Resultant bending moment =10219.027 KN-mm

Ast $= \frac{\text{M}}{\sigma\text{st} \times \text{j} \times \text{d}} = \frac{10219027}{190\times 0.86\times 265}$

= 2359.99 mm^2

Spacing of 20 mm dia bar = $\frac{\frac{\pi}{4}\times 20^2\times 1000}{2359.99}$

= 133.118 mm

provide 20 mm diameter bars @ 130 mm c/c

➢ *Bottom Dome*

The Span of the bottom dome = 9 m

The Rise of the bottom dome = 2 m

Let be R is the radius of dome = $\frac{(\frac{\text{D}}{2})^2 + \text{h}^2}{2\times\text{h}}$

R = $\frac{(\frac{9}{2})^2}{(2\times 2)}$ = 6.0625 m

Let angle subtended by the dome is 2θ

$\sin\theta = \frac{(\frac{D}{2})}{R} = \frac{(\frac{9}{2})}{6.0625} = 0.75$

$$\theta = \ 48.6°$$

Thickness of dome is = 300 mm

➢ *Loads Calculation*

The Dead load = 25000 × 0.3 = 7.500 KN/m^2

The weight of water resting on bottom dome is

$$= \gamma_w \left[\frac{\pi}{4} \times d^2 \times h - \frac{\pi h_c}{3}\right](3R - \ h_c)$$

$= 1000[\frac{\pi}{4} \times 9^2 \times 7 - (\frac{\pi\times 2}{3})(3\times\ 6.0625 - 2)]$

= 4114.207 KN

Area of the dome surface = 2 π × R × h = 2π × 6.0625 × 2 = 76.18 m^2.

Load intensity due to the weight of water = $\frac{4114207.5}{76.1836}$

= 54.003 KN/m^2

The total load intensity = 54003.8 + 7500 = 61.503 KN/m^2

Meridional thrust = $\frac{WR}{1+\cos\theta}$

$= \frac{61503\times 6.0625}{(1+\cos 48.60°)}$ = 224.438 KN/m

The Meridional compressive stress =$(\frac{224438.25}{300\times 1000})$

= 0.75 N/mm^2

The hoop stress = $\frac{WR}{t}(\cos\theta - \frac{1}{1+\cos\theta})$

$= \frac{61503.8\times 6.6025}{0.3}(\cos 48.60° - \frac{1}{1+\cos 48.60°})$

= 0.073 N/mm^2

The hoop stress @ crown, i.e. at θ is 0˚

Maximum hoop stress is = $(\frac{WR}{t})(\cos\theta - \frac{1}{1+\cos\theta})$

$= \frac{61503.8\times 6.6025}{0.3}((1) - \frac{1}{2})$

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= 0.621N/mm^2

So provide nominal reinforcement of 0.3% of steel (stresses are low)

Provide 8 mm bars @ 100 mm of spacing.

➢ *Circular Girder*

The total load is calculated on the circular girder is: -
Weight of water, w_1= the weight of water on the (conical slab + dome)
= 5852787 + 4114207.5
= 9966994.5 N
= 9966.9945 KN (w1)

Weight of the top dome and the side wall, w_2= $V_2 \times 2\pi \times \frac{D}{2}$
= 55758$\times$ $2\pi \times \frac{15}{2}$
= 2627.5338 KN (w2)

Weight of the ring beam at B, w_3=(0.65 × 0.9 × 25000 × 2π × 4.5)
= 4133512.13 N (w3)

Weight of conical wall, w_4 = 1102.699 KN (w4)

Weight of lower dome, w_5= 25000 × 0.3 × 76.183 = 571.372 KN (w5)

Total load, W = w_1+ w_2+ w_3+ w_4+ w_5 = 14682.111 KN

Initially we assumed 8 number of columns for this tank so: -

Maximum negative bending moment =(0.0083Wr) = (0.0083) ×(1468211.36 × 4.5)
= 548.376 KN- m

Maximum positive bending moment =(0.00416Wr) = (0.00416) × (1468211.36 × 4.5)
= 274.849 KN-m

Max. Torsion = 0.0006Wr = 0.0006 × 14682111.36 × 4.5
= 39.641 KN-m

Shear force at support = $\frac{W}{2\times Number\ of\ columns}$
= $\frac{14682111.36}{2\times 8}$ = 917.631 KN

➢ *Support Section Design*

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

$$0.913bd^2 = M_{max}$$

∴ d = 960 mm

The overall depth of the beam = 985 mm

The Actual effective depth is = 960 mm [(985 – 25) (25 mm = clear cover)]

The Equivalent shear force is = Se = S + 1.6 ×$\frac{T}{b}$

= 917631+1.6× $\frac{39641.7\times 10^3}{650}$

= 1015.210 KN

Equivalent nominal shear stress

$$\tau_{vc} = \frac{S_e}{bd} = \frac{1015210.569}{650 \times 960}$$

$$\tau_{vc} = 1.626\ N/mm^2$$

Maximum shear stress τmax > τv

τmax = 2.2 N/mm^2 (for the mix of M_{30} grade of the concrete as per IS : 456-2000)

∴ Hence it is safe.

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

- *The Longitudinal Reinforcement*

Me =(M + M1)

M1 = $\frac{T(1+\frac{D}{b})}{1.7}$

= $\frac{39641.7\times 10^3(1+\frac{985}{650})}{1.7}$

= 58655366.06 N-mm

$$A_{st} = \frac{M}{\sigma_{st} \times j \times d}$$

= $\frac{548376.85\times 1000}{(230\times 0.9\times 960)}$ = 2760 mm^2

Provide 14 number of bars of 16 mm Ø (2814.86 mm^2)

- *Transverse Reinforcement*

Asv = $\frac{T \times Sv}{b1 \times d1 \times \sigma sv} + \frac{V \times Sv}{2s\times d1 \times \sigma sv}$

Distance b/w centers of the corner bars which is parallel to the width is

b1 = 650 – (2 × 40) = 570 mm

The distance b/w centers of the corner bars parallel to the depth is

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d1 = 985 - 2 × 40 = 905 mm

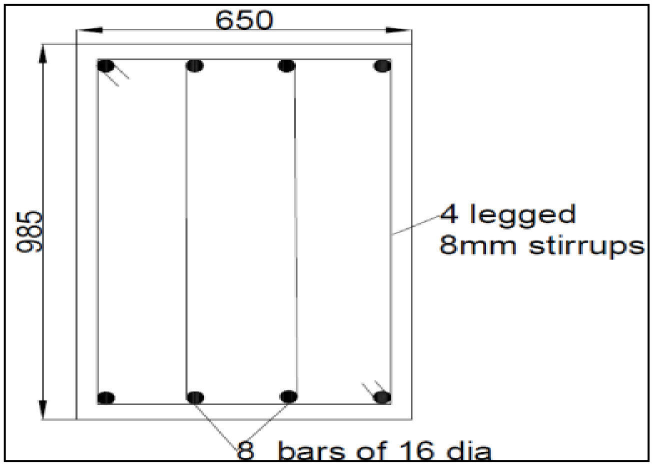

Fig 7: Reinforcement detailing of the girder

- *Area Section of Stirrups*

Asv = $[\frac{39641.7\times10^3}{570\times905\times230} + \frac{1015210.5}{2.5\times905\times230}]s_v$

Provide 4 legged 10 mm dia stirrups.

Asv = $4 \times \frac{\pi}{4} \times 10^2$ = 316 mm^2

Sv = 138 mm

Provide, Sv = 100 mm

Transverse reinforcement shall not be less than $\frac{(\tau_{vc}-\tau_c)bs_v}{\sigma_{sv}}$

315 = $\frac{(1.46-0.3)\times650}{230} \times s_v$

$$s_v = 83\ mm$$

∴Provide 80 mm of spacing (<83mm)

- *Steel for the Sagging Moment*

$$A_{st} = \frac{M}{\sigma_{st} \times j \times d}$$

= $\frac{274849.12\times10^3}{230\times0.9\times960}$

= 1383.09 = 1400 mm^2

So Provide 8 numbers of 16 mm diameter

$$A_{st} = 1608mm^2$$

- *Hoop Stress*

T_c = Thrust exerted by conical slab on the girder.

(T_c Sinα × 2πr) =(W_w+ Ws+ weight of upper dome and the cylindrical wall).

T_c sinα ×2πr = 5852787 + 2627533.84 + 413512.133

= 8893.83284 KN

(Tc sin33.69°)× 2π x 4.5

= 8893.83284 KN

The Horizontal component of Tc is = 567073.023 × cos 33.69°,

$$H_1 = 471.8336\ KN$$

Horizontal component due to dome T'= 224438.25 × cos48.60°

H2 = 148.432677 KN

∴ Net Horizontal force = H1 – H2

= 323.410 KN

∴ Hoop force = 323410 × 4.5 = 1455.345 KN

Hoop compressive stress is = $\frac{1455345}{650\times985}$ = 2.273 N/mm^2

➢ *Columns*

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

The Vertical load on the one column at top = $\frac{14682111.36}{8}$
= 1835.263 KN

Let α be the inclination of column with the vertical.
$\tan\alpha = \frac{1}{12}$, $\alpha = 4.76°$
sin α = (0.83); and $\cos\alpha = 0.99$

The Actual length of the column is = 1 sec α = 12.04 m

Providing the diameter of the colun is 500 mm.

Weight of column (dead load) $= \frac{\pi}{4} \times 0.5^2 \times 12.04 \times 25000 =$ 59.102 KN

Total vertical load = 1835263.9 + 59102 N
= 1894.365 KN

∴ Corresponding axis load = $\frac{1894365.9}{0.995}$
= 1903.885 KN

Weight of water in the tank is = 9966.994 KN or 9966994.50N

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Weight transmitted to the one column is = $\frac{9966994.5}{8}$ = 1245.875 KN

Vertical load in one column when the tank is totally empty in condition = 1894365.9 – 1245874.3
= 648.492 KN

∴ Corresponding axial load = $\frac{648491.58}{0.996}$
= 651.095 KN

By Ignoring the wind load effect on column if the steel requirement is A_{sc}

Then the A_{sc}
C (A – Asc) + Asc = 1903885.347

$$8\left(\frac{\pi}{4}\times\right) + 190 \times A_{sc} = 1903885.347$$

$A_{sc} = 1440\text{m}m^2$

Minimum Requirement of steel = 0.8%
= $\frac{0.8}{100} \times \left(\frac{\pi}{4} \times 500^2\right)$

= 1571 mm^2

Provide 7 number of bars of 20mm diameter = 2199 mm^2

(More of the steel has been subjected because the column is subjected to the bending moment caused by Wind load)

➢ *The Staging And Gravity Load*
The Height of the column = 12.04m

No. of column is = 8, Diameter of the column = 500 mm

Number of the braces is = 4, Size of the braces = 300 × 500 mm

➢ *The Gravity Loads*

- *From the Container*

When tank is in full condition = 14108.730 KN
When tank is in empty = 4663.689 KN

- *The weight of column is*

= $8 \times \frac{\pi}{4} \times 0.5^2 \times 12.04 \times 25$ = *472.809 KN*

- *The Weight of Braces*

The clear length of braces b/w two column = $2 \times \frac{(10.5)}{2} \times \sin 22.5° - 0.5$
= 3.519 m

Weight of braces is = (3.519 × 0.3× 25 × 0.5 ×4 × 8) = 422.160 KN

Total weight of the staging = 472.8097 + 422.160
= 894.970 KN

➢ *Lateral Forces Analysis*
The base shear = V=$\sqrt{Vi^2 + Vc^2}$

$V_i = (A_n)i\ (m_{i+}m_s)$

$A_h = \frac{Z}{2}$ x $\frac{I}{R}$ x $\frac{Sa}{g}$

- *Impulsive Mode:*

$\frac{hi}{h}$ = 0.35 (from fig (2) (a) of IS 1893 part-2)

$\frac{mi}{m}$ = 0.325

Z = 0.16 = III as per 1893 part-1)

I = 1.5 (Drinking water tanks IS 1893 part II)

R = 2.5 (Table no. 2 of IS 1893 part II)

$\frac{sa}{g}$ = The Average response acceleration coefficient

H = 4.5m, D = 15m

$T_i = 2\pi\sqrt{(\frac{mi+ms}{ks})}$

$m_s = {}^1/_3$ of staging and the mass of empty container

k_s = stiffness of staging in lateral direction

$K = \frac{n(12\,EI)}{\sum j3}$

$I = \frac{\pi}{64} \times 550^4$ = 4.4910 x10^9 mm^4

E = $5000\sqrt{fck}$

=$5000\sqrt{30}$

= 2.7386x10^4 Mpa

$K = \frac{8\times12\times2.7386\times10^4\times4.491\times10^9}{4\times3.01\times10^9}$

K= 1.0826×10^5N/mm

$T_i = 2\pi\sqrt{(\frac{9966.99\,x\,10^3 ms}{ks})}$

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$m_s = 4663.689 + \frac{1}{3}$ x 894.970

$m_s = 4692.012$

$T_i = 2\pi\sqrt{(\frac{9966.99 \ x \ 10^3 + 4692.012 \ x \ 10^3}{1.0826x105})}$

$T_i = 2.33$ sec

$\frac{sa}{g}$ Value is taken from fig for hard soil (IS 1893 part-1)

$\frac{sa}{g} = \frac{1}{T} = \frac{1}{2.33}$ (0.4 < T > 0.4)

$\frac{sa}{g} = 0.429$

$A_h = \frac{0.16}{2}$ x $\frac{1.5}{2.5}$ x 0.429

$A_h = 0.0206$

$V_{i\,=}$ 0.0206 (3239.25+4982.012) x 9.81

$V_i = 168.946$ KN

- *Convective Mode:*

$\frac{hi}{h} = 0.560$

$\frac{mi}{m} = 0.610$ (from fig 2(b) of IS 1893 part 2)

$Tc = Cc\sqrt{(\frac{D}{\gamma})}$

$\frac{h}{D} = \frac{4.5}{15} = 0.3$ (from fig 5 of IS 1893 part-2)

Cc = 4, D = 15m

= 1000 kg/m^3

$Tc = 4\sqrt{(\frac{15}{1000})}$

Tc = 0.489

$\frac{sa}{g} = \frac{1}{T} =$ (0.4 < T > 0.4)

$= \frac{1}{2.33} = 2.0412$

$A_h = \frac{0.16}{2}$ x $\frac{1.5}{2.5}$ x 2.0412

Vc = (A_{hc}) (m_c) g

m_c = convective mass of liquid

Vc = 0.0979 x 6079.86 x 9.81

Vc = 595.21 KN

$V = \sqrt{168.946^2 + 595.21^2}$

V = 618.730 KN

$V_i = 168.946x10^3$ N

$Vc = 595.21x\ 10^3$ N

$V = 618.730x10^3$ N

Take shear force for column for impulsive mode

Shear force for column = $\frac{168946.23}{8}$
= 211.182 KN

Maximum bending moment for the column = 211.182 x $\frac{3.01}{2}$
= 317.829 KN-m

- *Analysis of Wind Forces in Tank*

Terrain category – 2

The type of the structure – class A

The Basic wind speed of North to the East Indian, V_b = 50 m/sec or 180km/hr

The Design wind speed is VZ = V_b K_1 K_2 K_3

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

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

Topography factor is K3 = 1; for the slope of < 30, (it is taken from cl.5...3.1of IS: 875 -1987 part – 3)

Design wind speed is(Vz) = 50×1 × 0.9 × 1.124 = 50.58 m/sec

The Design wind pressure is Pz = (0.6 x V)= 1535 N/m^2

Reduction factor is taken as 0.7

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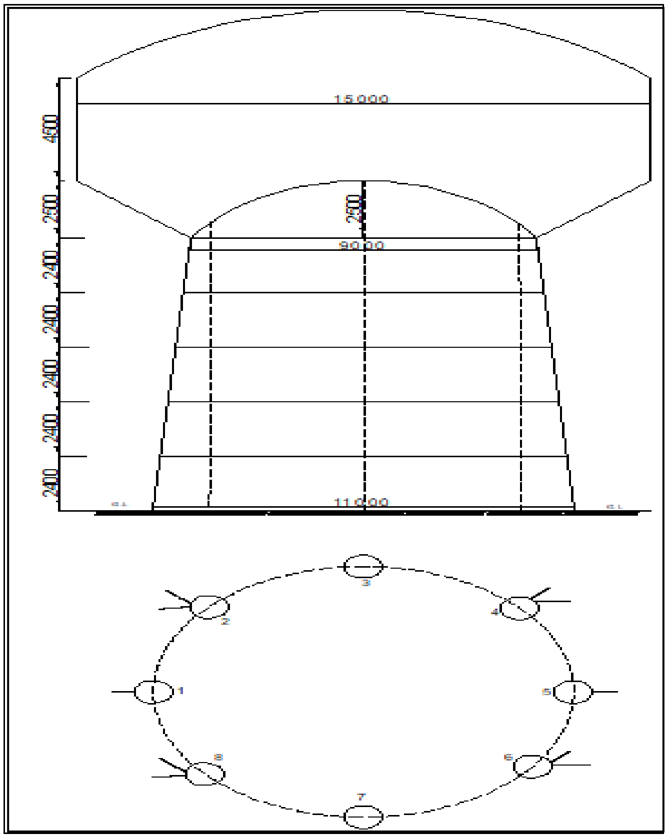

Fig 8: Wind Force Analysis in the Column

Now the wind force on the top of the dome and cylindrical walls is

$$= (4.5 + \frac{3}{2}) \times 15.6 \times 1535 \times 0.7$$

= 100.573 KN

Wind force Acting at the 16.9 meters above the base.

The Wind force acting on the circular wall = $\left(\frac{15.6+10.5}{2}\right) \times 2 \times 1535 \times 0.7 = 28.045\ KN$

Acting @ 13.04 meters above the base

The wind force acting on the circular girder = $10.5 \times 0.905 \times 1500 \times 0.7 = 9.978\ KN$

Acting @ 12.04meters above the base of tank

The Wind force acting on column and the braces is = $0.7 \times (12.04 \times 0.5 \times 5) + (7 \times 10.5) \times 1535$
= 71.831 KN

Acting at 6.02 meters above the base

The total moment of the wind pressure about the base is

= $(16.9 \times 96383) + (13.04 \times 28045) + (12.04 \times 9978) + (6.02 \times 71831)$
= 26179.578 KN-m

due to wind load the Vertical load on any column = $\frac{M\ X}{\Sigma X^2}$

$$\Sigma X^2 = 2r^2 + 4(r \sin\frac{\pi}{4})^2 = 2 \times 5.5^2 + 4(5.5 \sin 45°)$$

Whereas
r= radius of the column circle = 5.5 m

$$\Sigma X^2 = 121\ m^2$$

The maximum wind force is in most leeward and the most of windward side

= $\frac{2617951.789 \times 5.5}{121}$ = 118.997 KN

Maximum wind load in the column marked as no.5

= $\frac{2617951.789 \times 5.5}{121} \times \frac{5.5}{\sqrt{2}} = 462.792 KN$

Consider the column 1 as windward

Vertical load due to the load and wind load = 1894.3659 + 118.997 = 2013.362 KN

Corresponding axial load = $\frac{201336.29}{0.996}$ = 2021.448 KN

Since the columns are inclined to the horizontal component of the axial force caused by wind

This action reduces the horizontal shear in the column.

So Horizontal component of the axial force caused by wind action is

= 2 × 118997×(0.0996 + 4) × 462792 × (0.0996)×$\frac{1}{\sqrt{2}}$ = 130.397KN

∴ Actual horizontal force at the base = (100573.2 + 28045 + 9978 + 71831) – 13397.45

= 197.029 KN

∴ Horizontal shear per each column is =$\frac{197029.75}{8}$ = $24.62871\ KN$

∴ Bending moment maximum for the column is = $24628.71 \times \frac{3.01}{2}$ = 37.066 KN-m

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The effect of wind load is less than the seismic loads so;

The design staging is fully governs by seismic forces.

➢ *The Column Section Analysis*
Radius of the column circle = 5.5 m

The Axial force in the column due to gravity load when tank is full = 15003.700 KN

The overturning moment when the tank is full = 16.07 × 145.8714 = 2344.154 KN-m

The Maximum axial force on remotest column staging,

When the tank is full = $\frac{15003700}{8} \pm \frac{2344154}{\Sigma(x \times x)} \times R$

Where, $\Sigma X^2 = 2R^2 + 4(r \sin\frac{\pi}{4})^2 = 121$

= $\frac{15003700}{8} \pm \frac{2344154}{121} \times 5.5$ = 1982.015 KN

= $\frac{19820015}{0.995} = 1989.975\ KN$

Provide 9 no.s of 20 mm diameter, and an effective cover 50 mm.

Area of the steel, (Ast) = 9 × $\frac{\pi}{4}$ x 20^2 = 2828 mm^2

Equivalent concrete area = $\frac{\pi}{4} \times 500^2 + 12.33 \times 2828$ = 231219 mm^2

Polar moment of inertia of the equivalent concrete section is

$$= \frac{\pi d^4}{64} + A_{st}(m-1)r^2\left(\frac{n}{2}\right)$$

= $\frac{\pi \times 500^4}{64} + 2828 \times (13.33 - 1) \times (250 - 150)^2 \times \left(\frac{9}{2}\right) =$ $9.34 \times 10^9 mm^4$

Therefore, the Equivalent polar moment of inertia about a diameter of circle = 9.34× $10^9 mm^4$

The Direct stress in concrete section = $\frac{1989975}{231519}$ = 8.61 N/mm^2

The Bending stress in the concrete section is=$\frac{228270.4}{9.34 \times 10^9} \times 250 \times 1000 = 6.11$ N/mm^2

The designed Factored load, Pu = 1.5 × 1982015

= 2973.0225 x 10^3N

Mu = 1.5 × 228170.400 = 342.405 KN-m

Dimensionless parameter for seismic forces,

$$\frac{P_u}{f_{ck}D^2} = \frac{2973022.5}{30 \times 500 \times 500} = 0.396$$

$$\frac{M_u}{f_{ck}D^3} = \frac{342405.5}{30 \times 500^3} = 0.088$$

From SP:16, chart-55,

$$\frac{P}{f_{ck}} = \frac{0.12}{0.4} = 0.048$$

P = 30 × 0.048 = 1.44%

Requirement of the steel =$\frac{1.44}{100} \times \frac{\pi}{4} \times 500^2 = 2828\ mm2$

Provide 9 no. of bars 20 mm diameter at an effective cover of 50 mm.

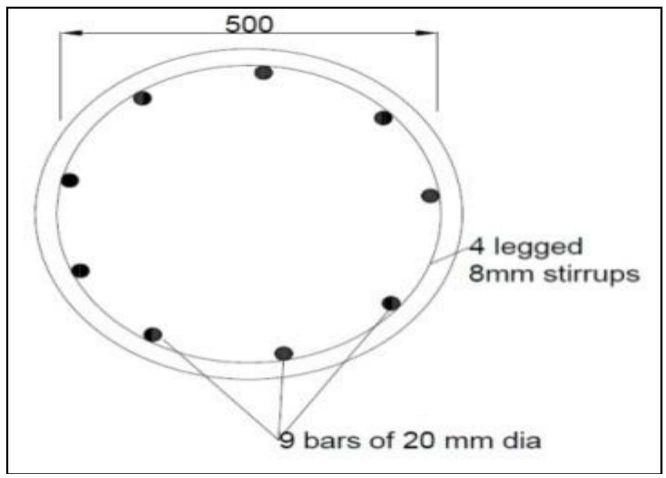

Fig 9: Reinforcement Detailing of the Column Section

➢ *Braces Design*
Consider the braces, say the base is BC

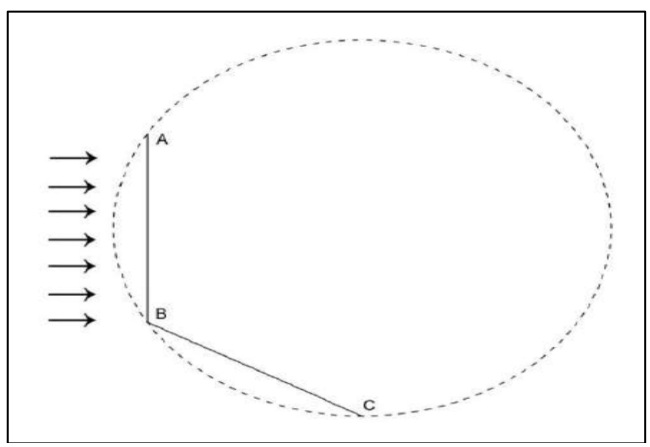

Fig 10: The Wind Pressure Acting on the Brace Beam AB

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

Moment in the brace BC = moment of the column × (sec 45̊)
= 317829.4 ×√2 = 449.478 KN-m

Providing the section of (300 ×500) mm and design it as a doubly reinforcement beam with the equal steel @ top and bottom,

So Asc = Ast =$\frac{449.478\times1000}{220\times420}$ = 4846.48 mm^2

Provide 6 no.of bars 20mm dia. @ top and equal amount of steel @ bottom.

Brace beam Shear force =$\frac{\text{B.M for Brace}}{0.5\times Spam\ of\ Brace}$

$= \frac{449478}{0.5\times3.518} = 255.530\ KN$

Therefore, Nominal shear stress, $t_v = \frac{s}{bd} = \frac{255530.41}{300\times500} =$ 170 N/$mm^2 <$ 1.8 N/mm^2

Hence this section is adequate with the shear reinforcement.

So, provide 4-legged 8 mm dia stirrups @ 160 mm c/c.

- *Foundation Design for Tank*

Total load in the columns when tank is in full condition = 1894365× 8 = 15154.927 KN

The Approximate weight of the foundation is taken as (10% of column load) = 1515.4927 KN

The total loads = 16670.420 KN or 16670420.235N

Safe bearing capacity of soil = 112.815 KN/m^2

The foundation area = $\frac{16670419.92}{112815} = 147.76\ m^2$

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

Therefore, the area of footing = $\frac{\pi}{4}(16^2 - 8^2)$ = 150.802 m^2

Therefore, the net upward pressure intensity is = $\frac{15154927.2}{150.802}$ = 100496.86 N/mm^2

= 100.496 KN/m^2< 112.815 KN/m^2

- *Circular Girder Design*

Maximum -Ve B.M. at the centre =(0.00416) × r ×w
= 0.00416× 15154927.5 × 5.5 = 346.744 KN-m

Maximum +Ve B.M. @ support = 0.0083 × r ×w
= 0.0083× 15154927.2 × 5.5 = 691.823 KN-m

The maximum torsion is = 0.0083 × W × r
= 0.0083× 5.5 × 15154927.2 = 50.012 KN-m

Maximum S.F. at support = $\frac{15154927.2}{2\times8}$ = 947.183 KN

- *Design of the Support Section*

The Moment of resistance = the maximum bending moment at the support
0.913 bd^2= 691822.4
bd^2= 757749.3× 1000
d = 1000 mm; b = 750 mm;
Effective depth is = (1000 – 60)= 940 mm.
The effective Shear Force is Se = S+1.6$\frac{T}{b}$ = 915007+1.6× $\frac{48312\times1000}{750}$ = 1018.073 KN
Equivalent nominal shear stress, qve = $\frac{S_e}{bd} = \frac{1018072.6}{750\times940} =$ 1.44N/mm^2
For M30 mix of concrete, the tc max is taken as = 2.2 N/m^2(from the code of IS 456:2000 table-24)

- *The Longitudinal Reinforcement of Support Section*

Equivalent B.M. = Me1 =(M + Mt)

Mt = T ×$\frac{1+\frac{D}{b}}{1.7} = \frac{48312000}{1.7} \times \left[1 + \frac{1000}{750}\right]$

= 66310.588 KN-m

Me1 = 668321500 + 66310588 = 734632.088 KN-m

Ast =$\frac{734632088}{230\times0.9\times900} = 3775mm^2$

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

- *The Transverse Reinforcement of Support Section*

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

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

Area of stirrups = Aw =$\frac{TP}{b_1\times d_1\times t_w} + \frac{SP}{2.5\times d_1\times t_w}$

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

316 = $[\frac{48312000}{250\times920\times670} + \frac{915007}{230\times920\times2.5}] \times P$

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316 = [1.73+0.34]P

P = 152.6 mm so provide 120 mm c/c(<152.6mm)

Steel for the hogging moment $=\frac{334966.5\times1000}{230\times0.9\times940} = 1722mm^2$

Provide 6 no. of bars 20 mm Ø (1885 mm^2).

➢ *Design of the Bottom Slab*

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

Max bending moment for 1meter width of strip = $174.225 \times \frac{0.5\times0.5}{2}$ = 21.778 KN-m

Equating the moment of resistance to the beam section is

$$0.913 \times 1000 \times d^2 = 21778125$$

depth (d)= 154 mm
provide d = 200 mm.

So the effective depth is = 200 – 40 = 160 mm (40 mm clear cover)

Ast $= \frac{21778125}{230\times0.9\times160} = 658mm^2$

Spacing of 12 mm diameter bars $=\frac{113\times1000}{658} = 172$ mm

provide 150 mm c/c (<172mm)

Distribution steel = $\frac{0.12}{100} \times 200 \times 1000 = 240mm^2$

Provide 8 mm diameter bars @ 200 m c/c.

➢ *Check for the Sliding*

When the tank is empty the total load acting on the foundation
= 15154927.2 – 9966994.5
= 5187.933 KN

Horizontal force acting on the base = 1166.9713 KN

Assuming the co-efficient of friction is 0.5.

So the factor of safety against sliding is = $\frac{0.5\times5187932}{1166971.3} = 2.22>$ 2.0 Hence, OK.

➢ *Check for Overturning*

Factor of safety against overturning = $\frac{5187932.7\times8}{16.07\times1166971.3} = 2.21>$ 2.0

Hence the check is ok so the tank is safe against overturning.

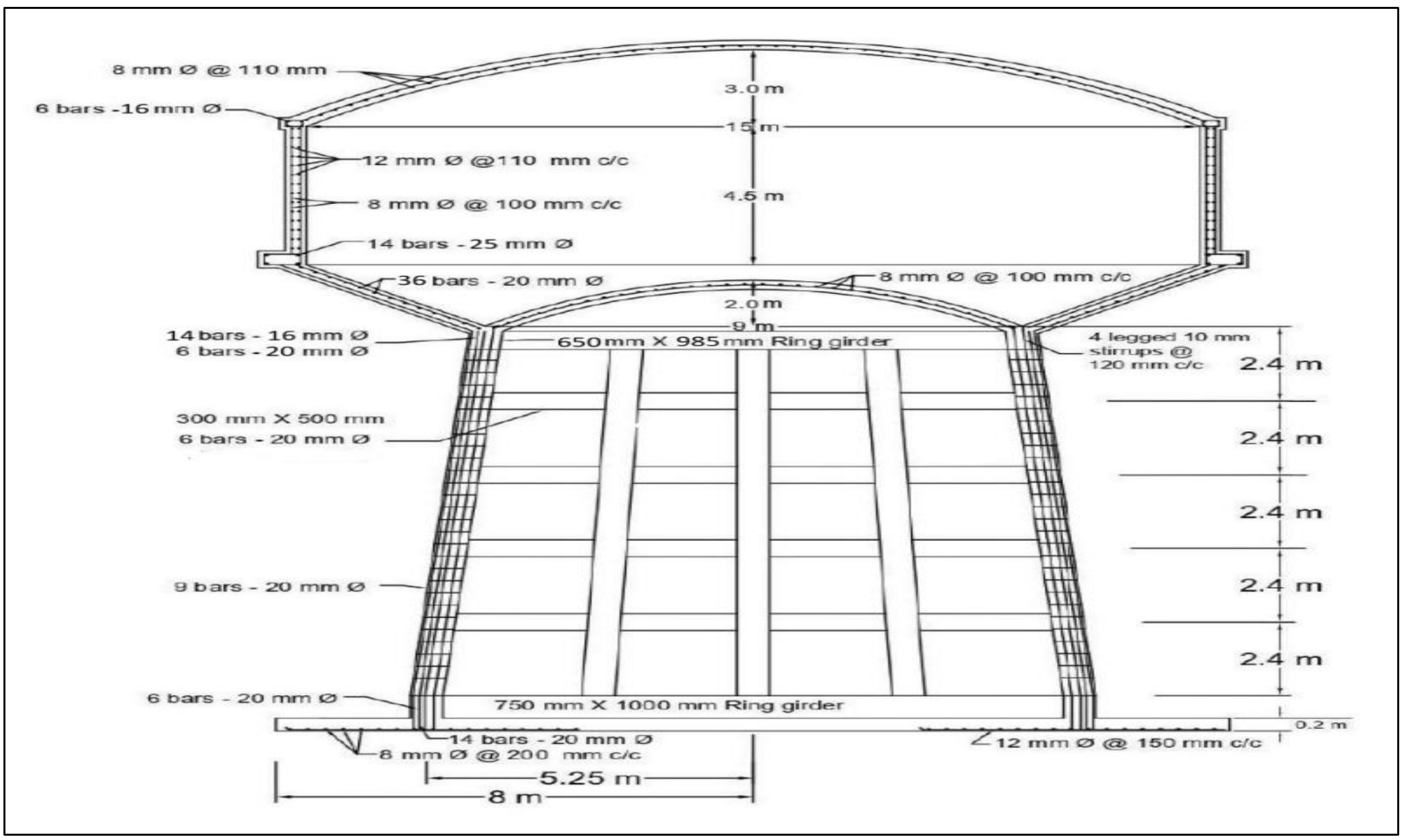

Fig 11: Intze Tank Reinforcement Detailing

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V. STAAD PRO RESULTS

A. The Parameters Applied in Staad Pro Design

➢ *Geometric Parameters for Frame Staging Tank:*

- Top Dome thickness is 150 mm
- Top Ring Beam size 250x400 mm
- Cylindrical Wall thickness 300 mm
- Bottom ring beam size 650x900 mm
- Conical dome thickness is 300 mm
- Bottom dome thickness is 300 mm
- Size of circular ring beam 650x985mm
- Column diameter is 500 mm
- Bracing size is 300x500 mm

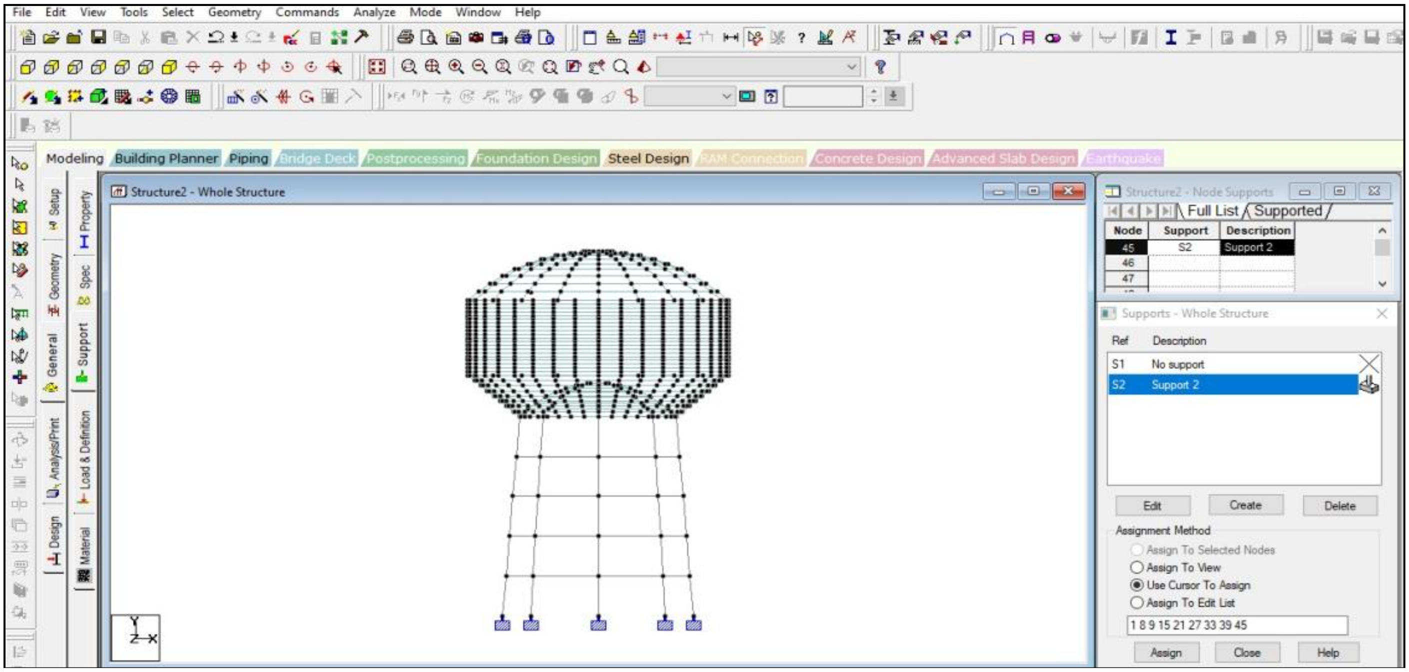

Fig 12: Node Diagram of Intze Tank

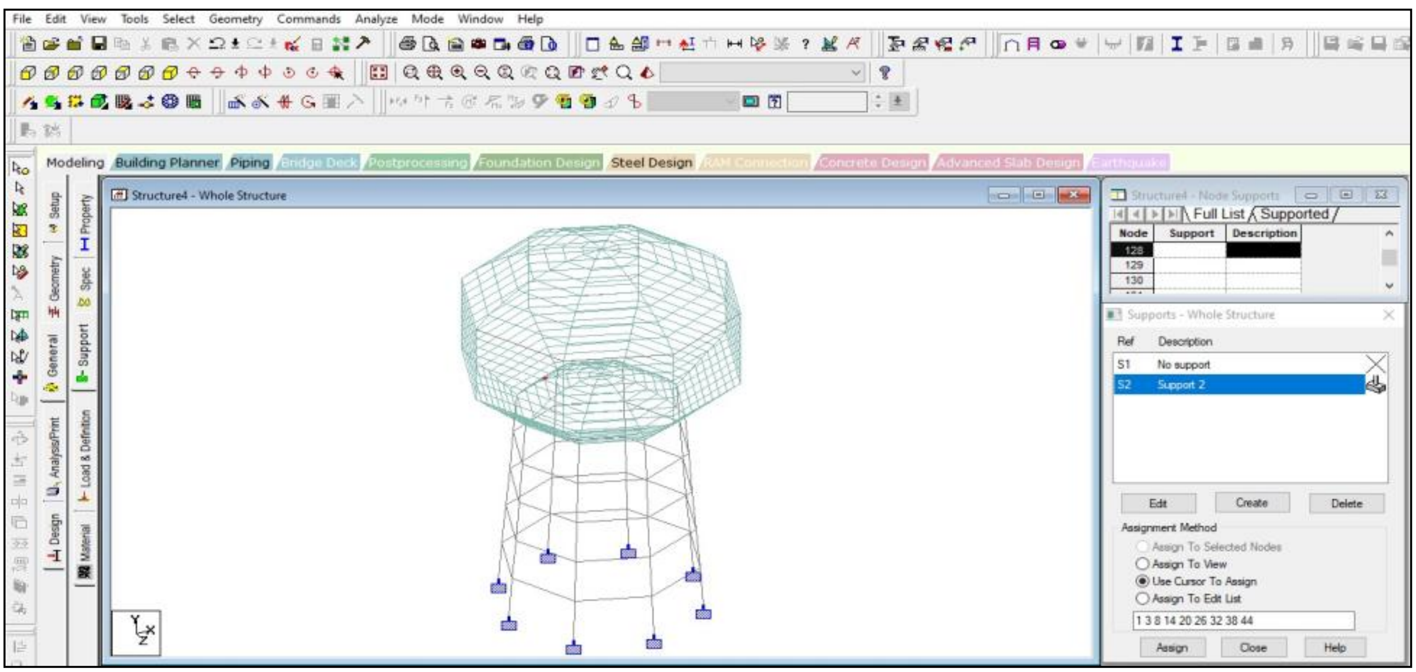

Fig 13: Wire Frame Structure of Intze Tank

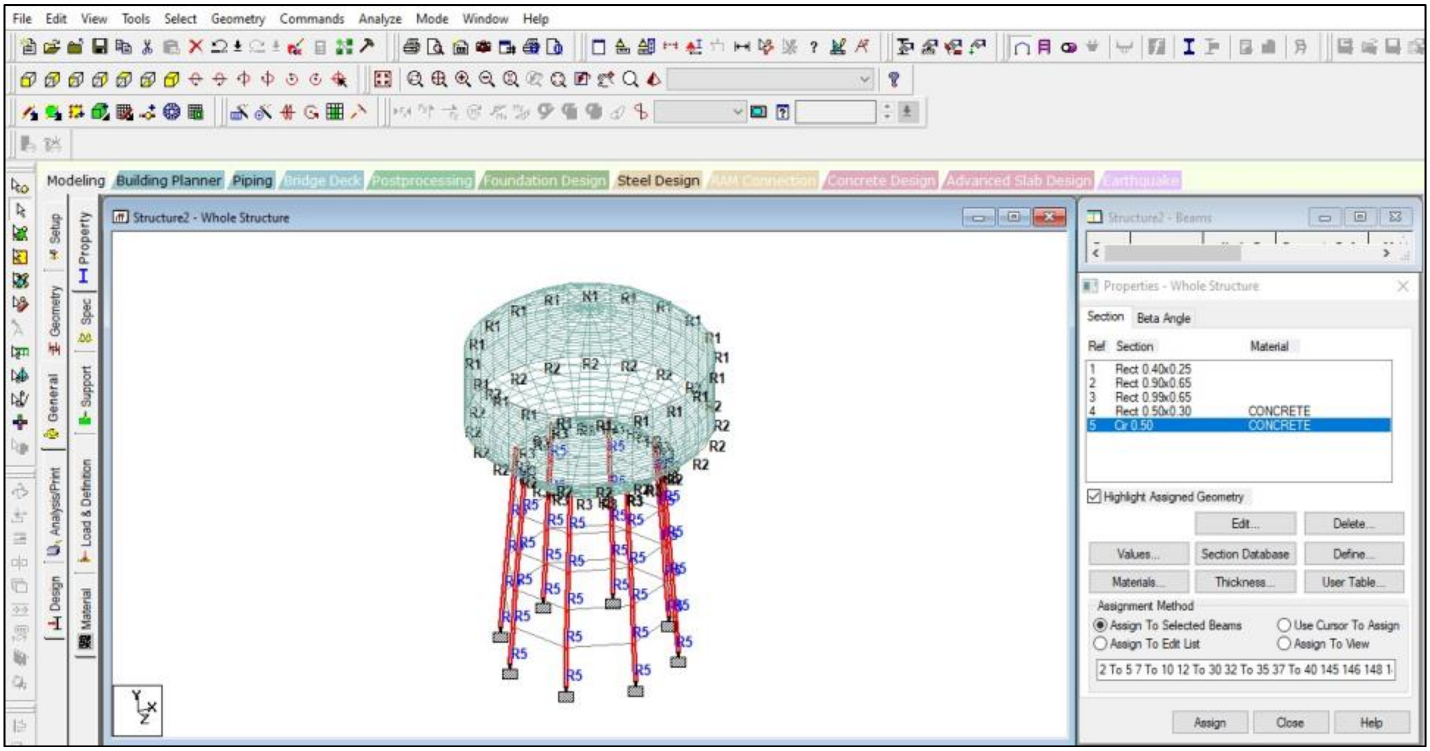

Fig 14: Assigning of Circular Column

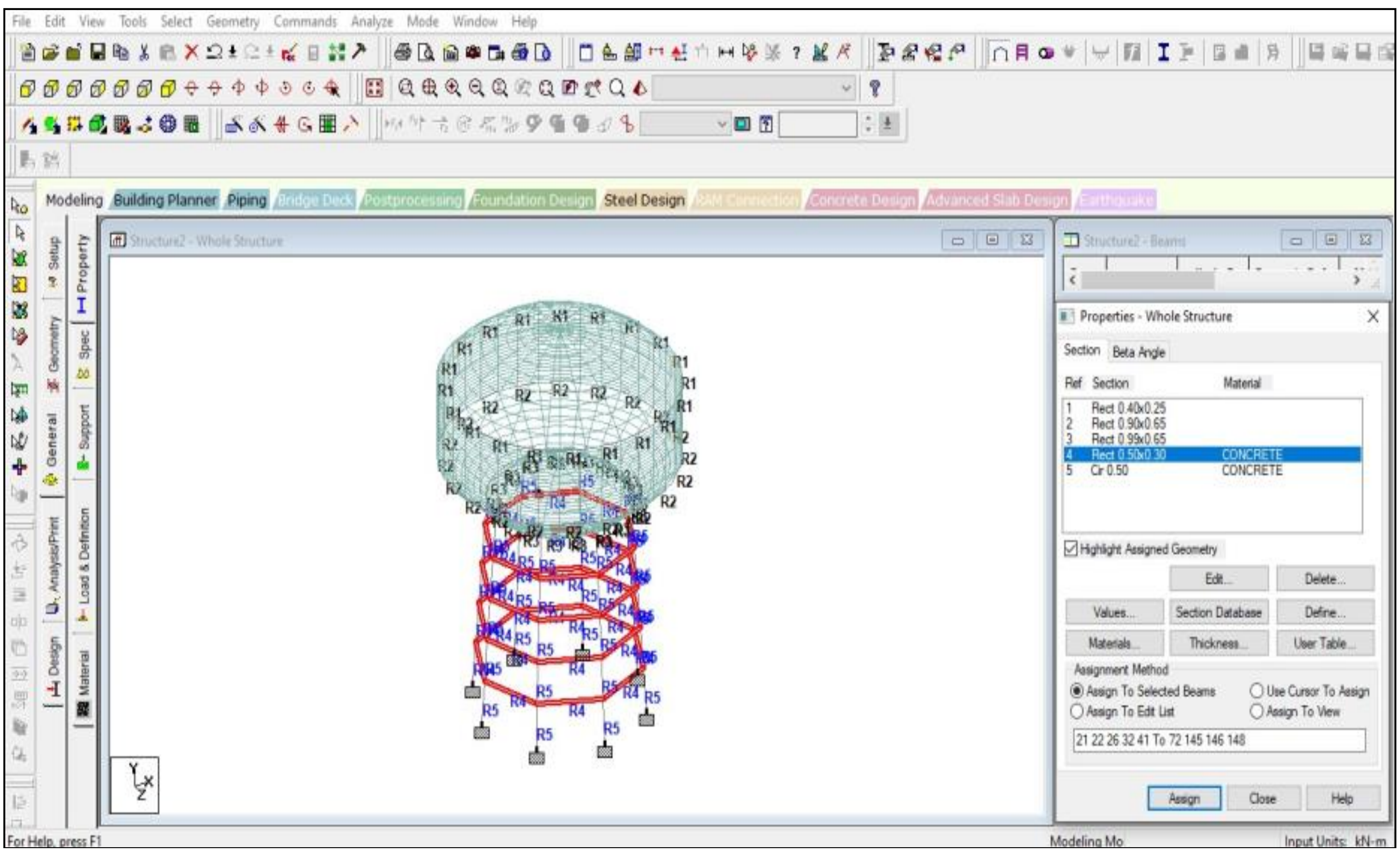

Fig 15: Assigning of braces

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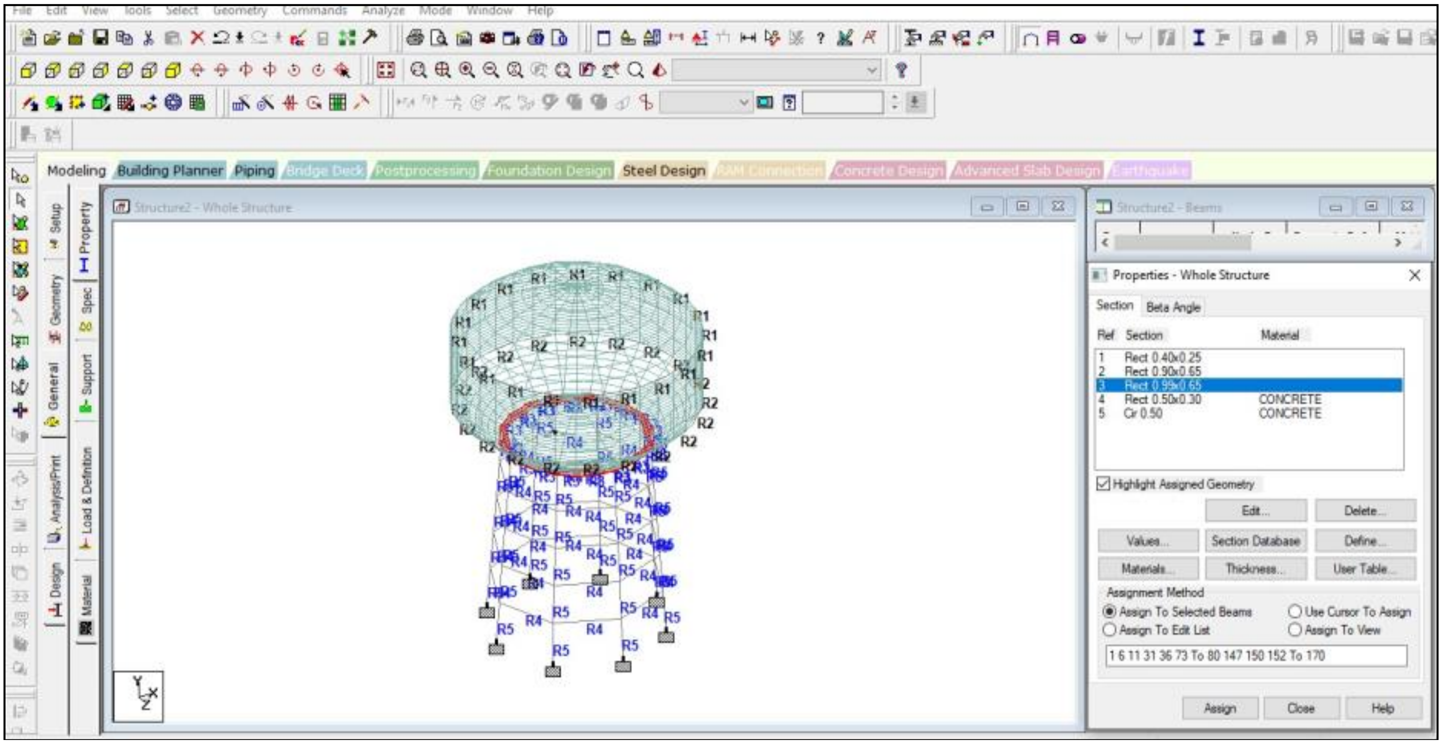

Fig 16: Assigning of Ring Girder

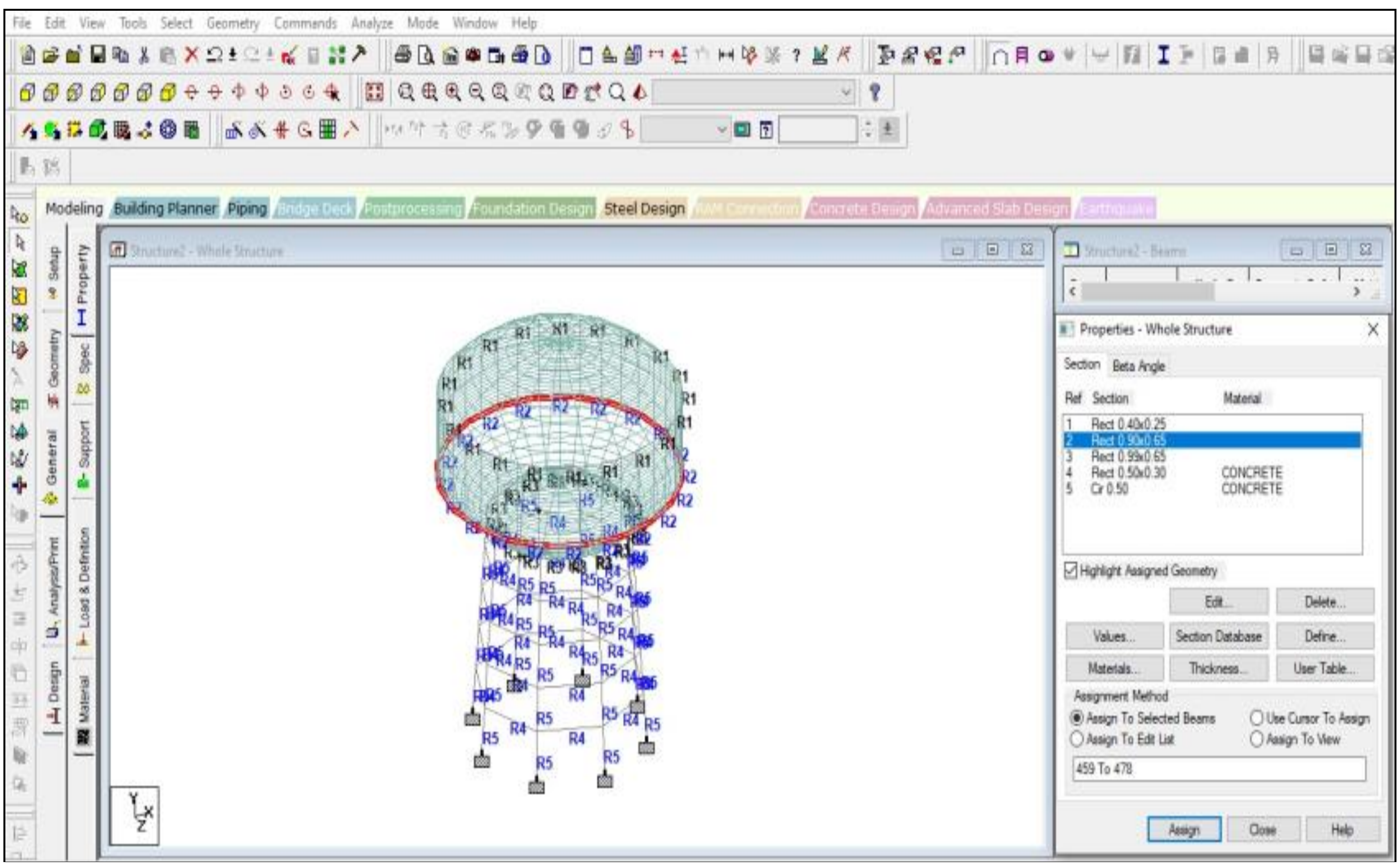

Fig 17: Assigning of Bottom Ring Beam

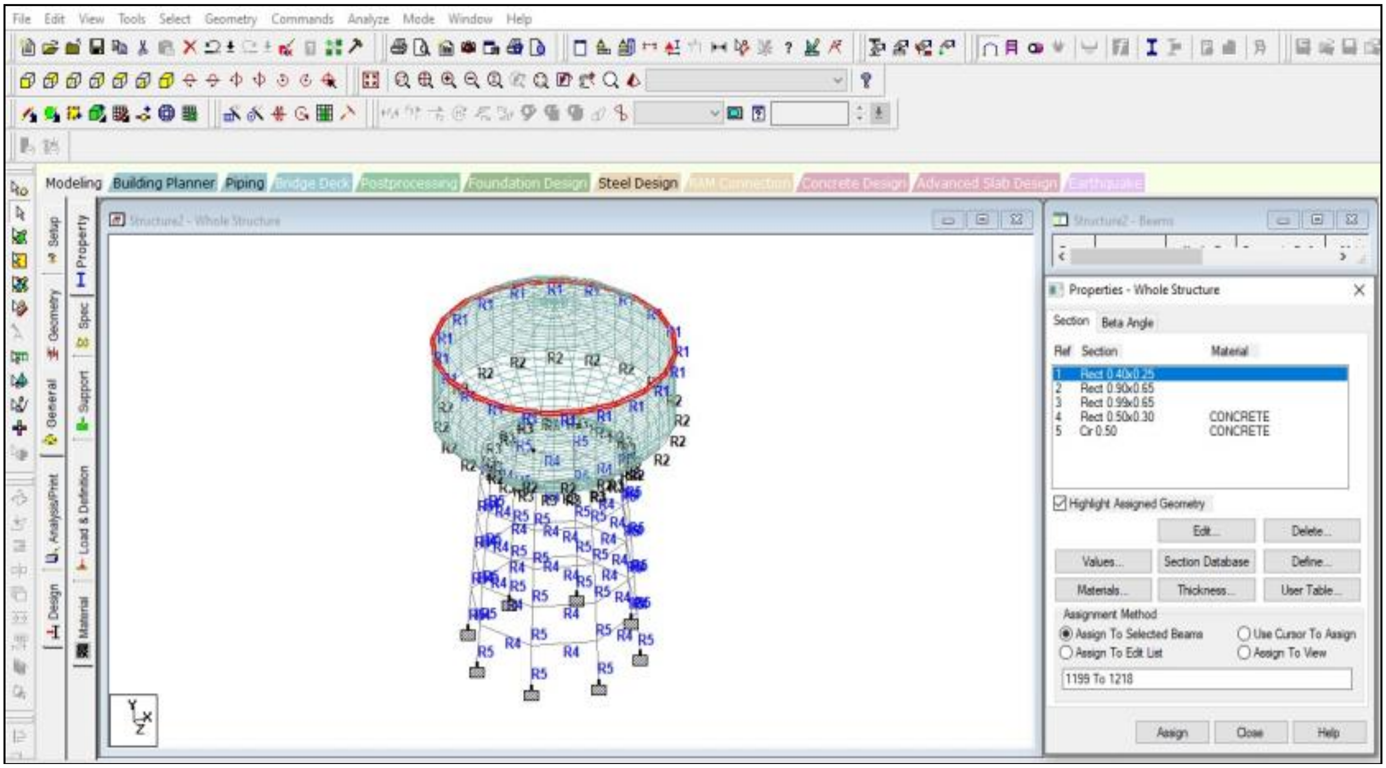

Fig 18: Assigning of Top Ring Beam

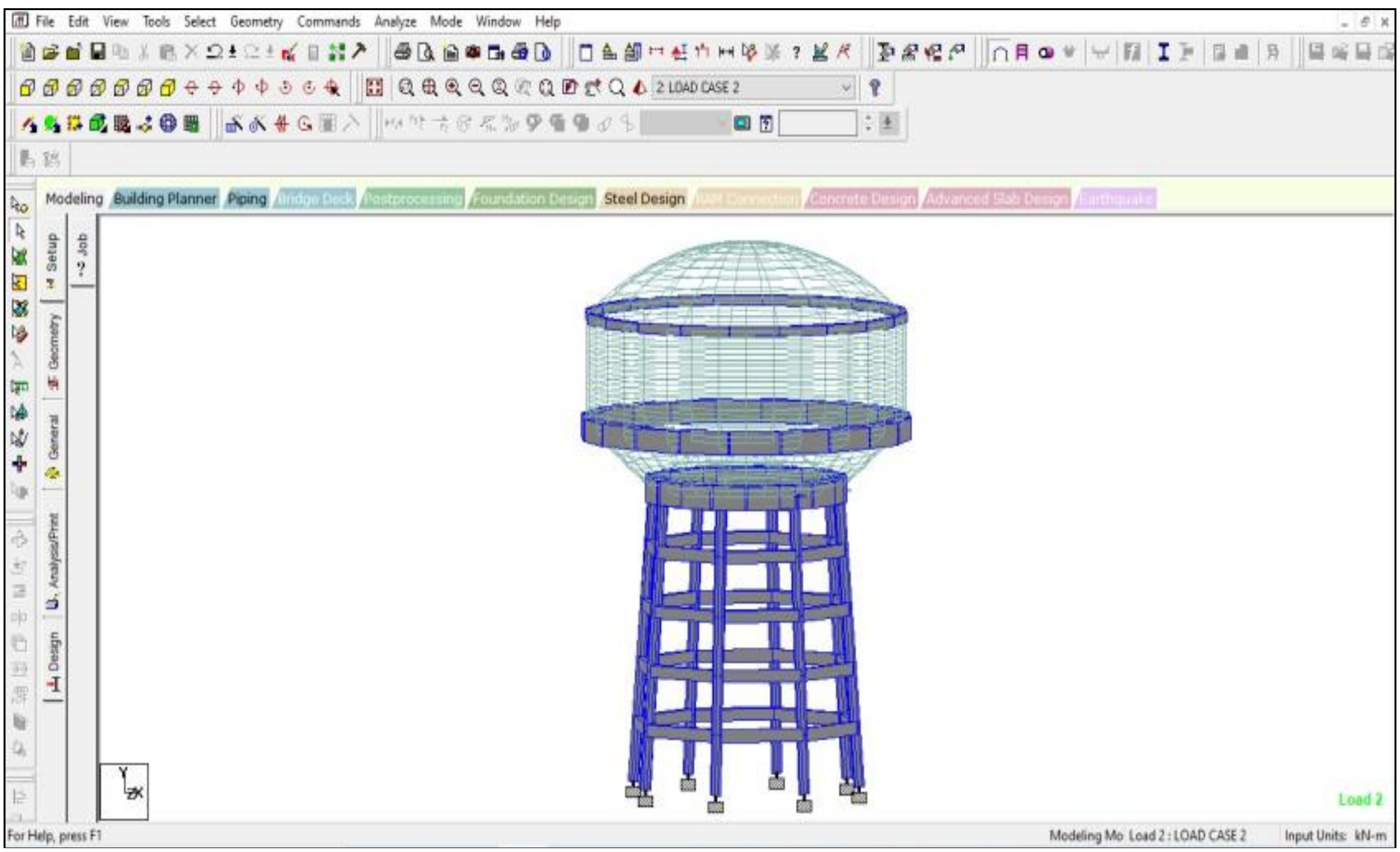

Fig 19: Whole Structure of Intze Tank with Assigned Elements

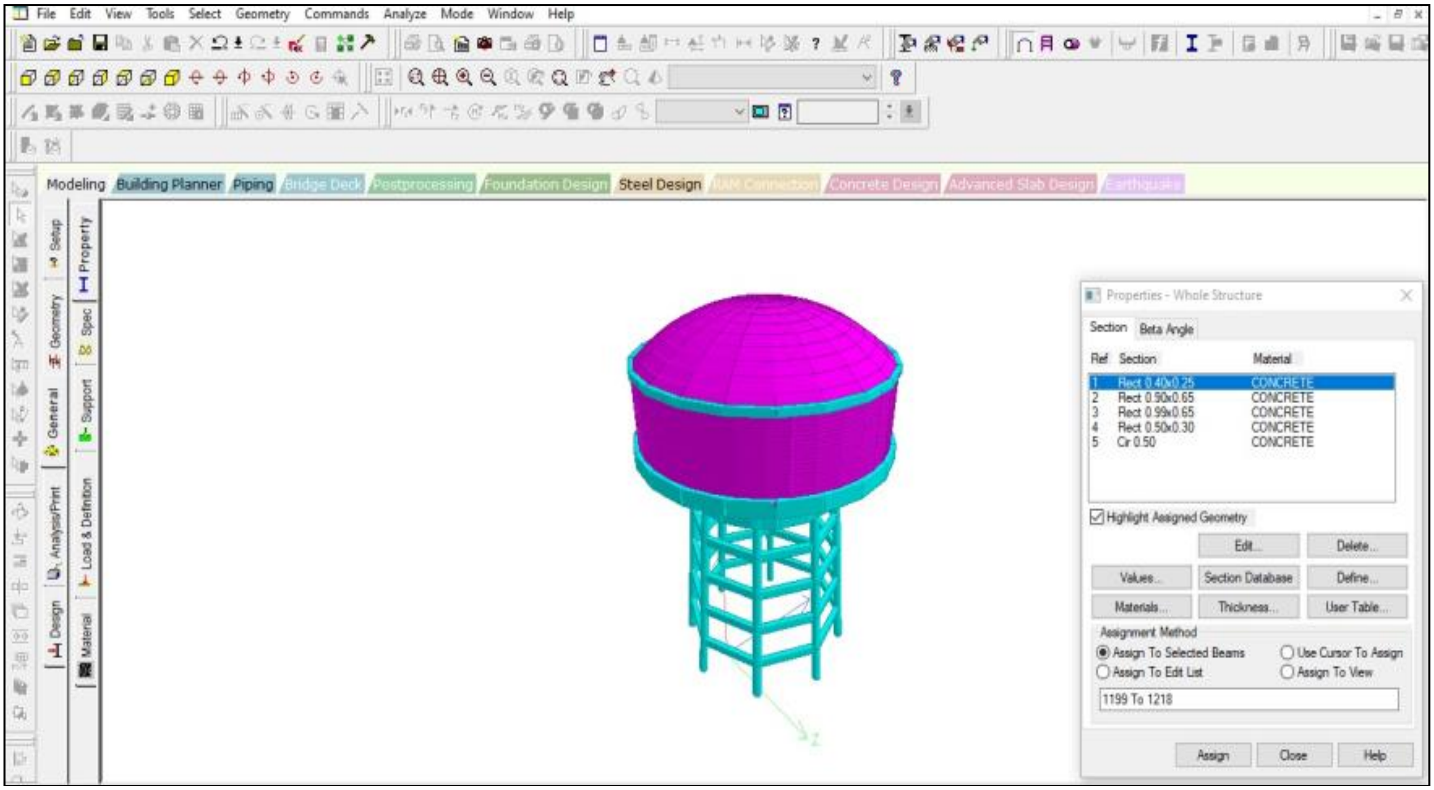

Fig 20: 3D Rendering View

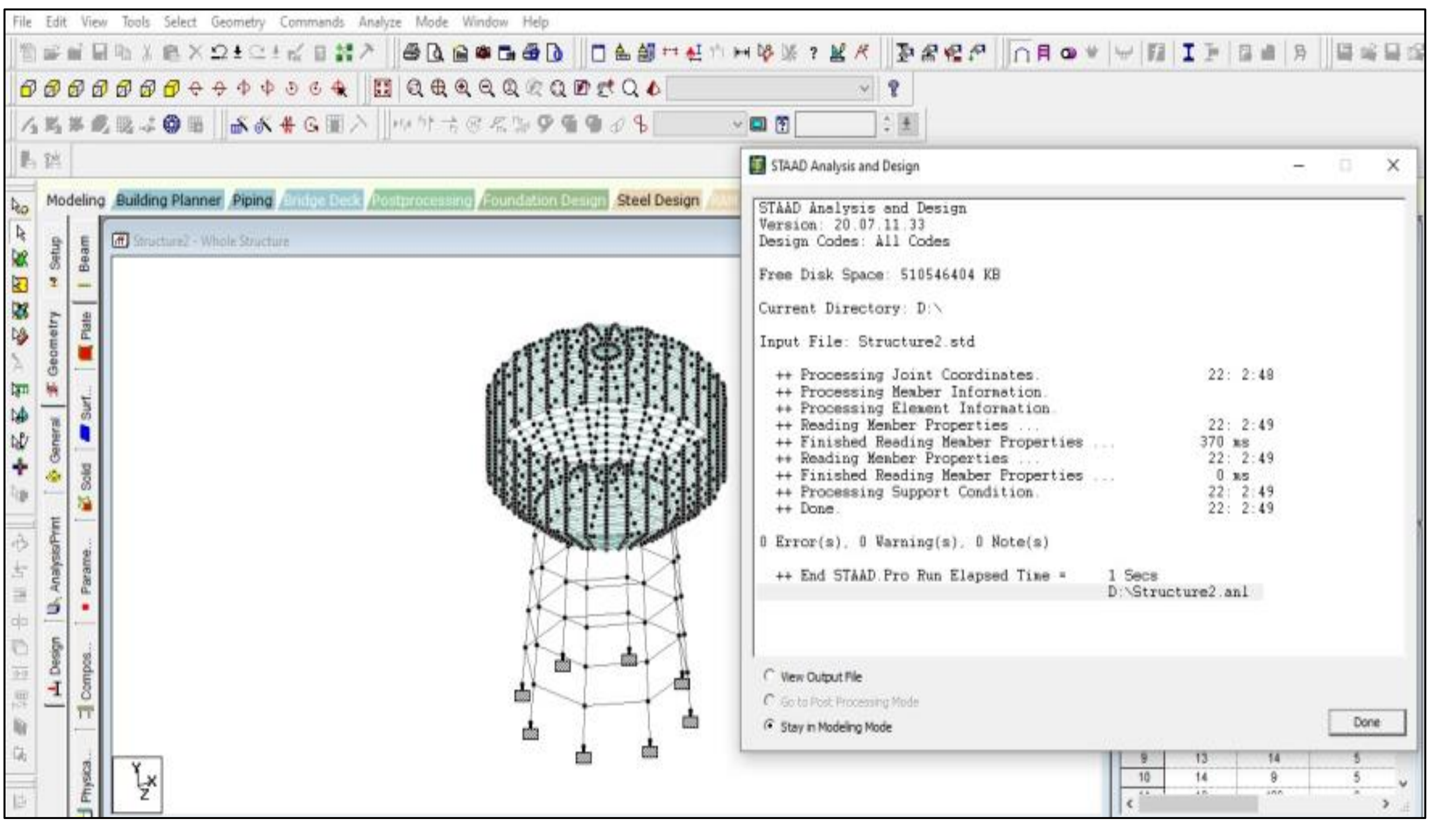

Fig 21: Analysis Results of Tank

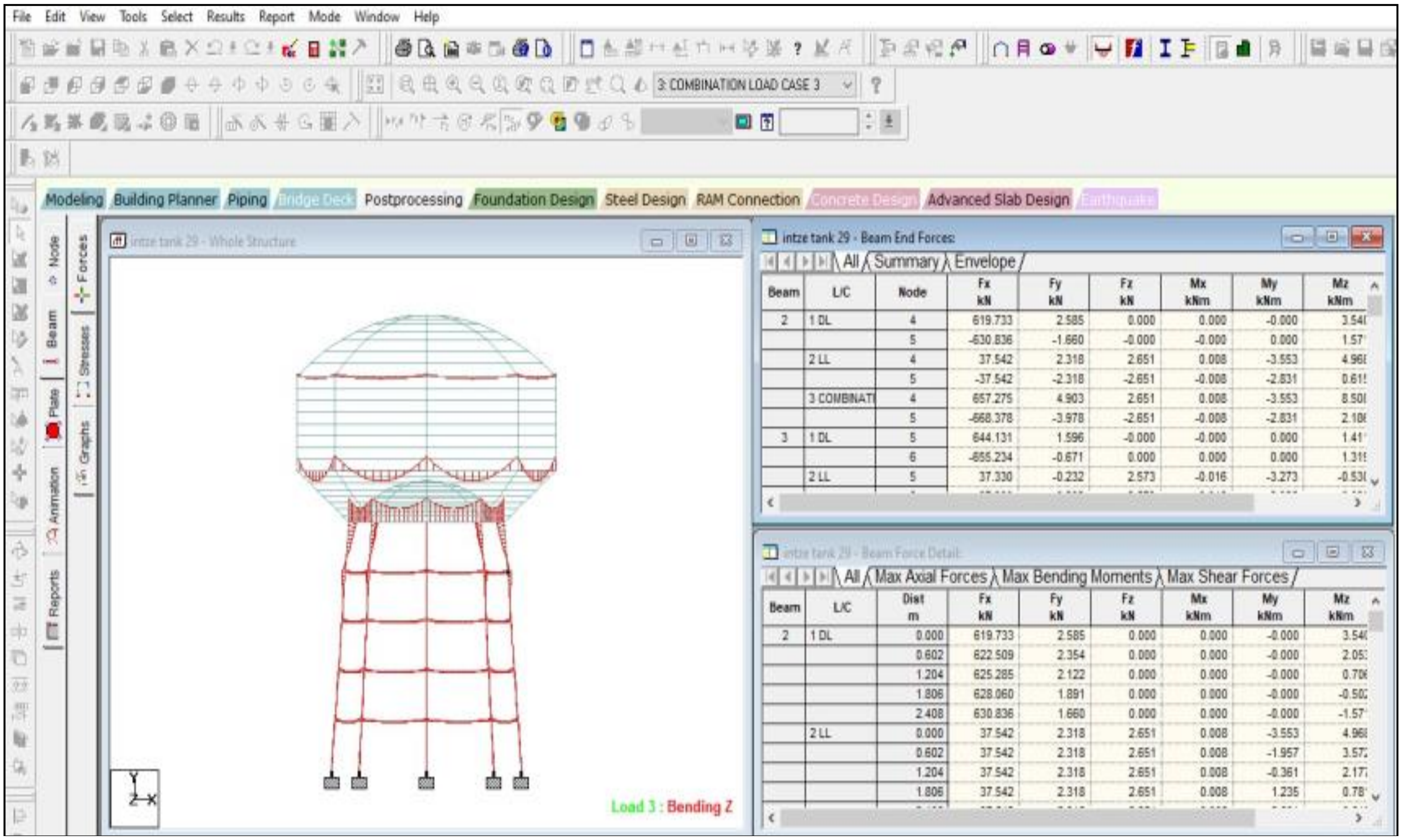

Fig 22: Deflections of Tank

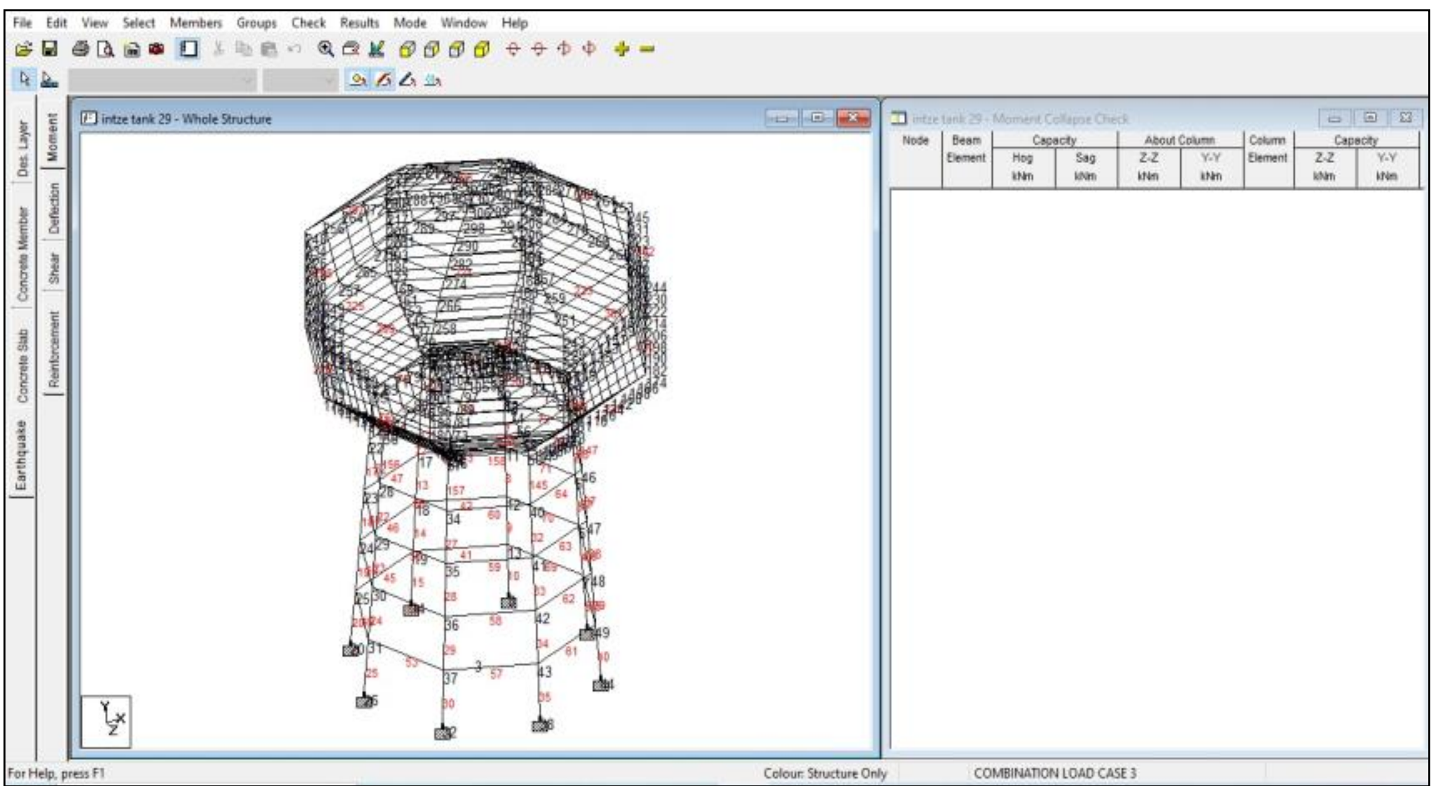

Fig 23: Moments and Deflections in Earthquake Condition

Table 1: Reactions Obtained in STAAD Pro

Reactions

		Horizontal	Vertical	Horizontal	Moment		
Node	L/C	FX (kN)	FY (kN)	FZ (kN)	MX (kNm)	MY (kNm)	MZ (kNm)
1	1:DL	-58.378	703.331	0.000	0.000	-0.000	-0.801
	2:LL	-3.071	37.220	-1.923	-2.920	-0.324	-0.062
	3:COMBINATIC	-61.449	740.551	-1.923	-2.920	-0.324	-0.863
8	1:DL	-41.280	703.331	41.280	-0.566	-0.000	-0.566
	2:LL	-4.105	62.517	2.185	-2.638	-0.226	0.279
	3:COMBINATIC	-45.385	765.848	43.465	-3.203	-0.227	-0.287
14	1:DL	0.000	703.331	58.378	-0.801	0.000	-0.000
	2:LL	0.000	72.299	4.967	-2.322	0.000	-0.000
	3:COMBINATIC	0.000	775.629	63.346	-3.123	0.000	-0.000
20	1:DL	41.280	703.330	41.280	-0.566	0.000	0.566
	2:LL	4.105	62.517	2.185	-2.638	0.226	-0.279
	3:COMBINATIC	45.385	765.847	43.465	-3.203	0.227	0.286
26	1:DL	58.378	703.330	0.000	0.000	-0.000	0.801
	2:LL	3.071	37.220	-1.923	-2.920	0.324	0.062
	3:COMBINATIC	61.449	740.550	-1.923	-2.920	0.324	0.863
32	1:DL	41.280	703.330	-41.280	0.566	-0.000	0.566
	2:LL	0.238	11.923	-2.158	-2.550	0.226	0.367
	3:COMBINATIC	41.518	715.253	-43.438	-1.985	0.226	0.932
38	1:DL	0.000	703.331	-58.378	0.801	-0.000	-0.000
	2:LL	0.000	2.141	-1.175	-2.198	-0.000	-0.000
	3:COMBINATIC	0.000	705.472	-59.553	-1.396	-0.000	-0.000
44	1:DL	-41.280	703.331	-41.280	0.566	0.000	-0.566
	2:LL	-0.238	11.923	-2.158	-2.550	-0.226	-0.367
	3:COMBINATIC	-41.518	715.254	-43.438	-1.984	-0.226	-0.933

Table 2: Reaction Summary Obtained in STAAD pro

Reaction Summary

			Horizontal	Vertical	Horizontal	Moment		
	Node	L/C	FX (kN)	FY (kN)	FZ (kN)	MX (kNm)	MY (kNm)	MZ (kNm)
Max FX	26	3:COMBINATIC	**61.449**	740.550	-1.923	-2.920	0.324	0.863
Min FX	1	3:COMBINATIC	**-61.449**	740.551	-1.923	-2.920	-0.324	-0.863
Max FY	14	3:COMBINATIC	0.000	**775.629**	63.346	-3.123	0.000	-0.000
Min FY	38	2:LL	0.000	**2.141**	-1.175	-2.198	-0.000	-0.000
Max FZ	14	3:COMBINATIC	0.000	775.629	**63.346**	-3.123	0.000	-0.000
Min FZ	38	3:COMBINATIC	0.000	705.472	**-59.553**	-1.396	-0.000	-0.000
Max MX	38	1:DL	0.000	703.331	-58.378	**0.801**	-0.000	-0.000
Min MX	8	3:COMBINATIC	-45.385	765.848	43.465	**-3.203**	-0.227	-0.287
Max MY	26	2:LL	3.071	37.220	-1.923	-2.920	**0.324**	0.062
Min MY	1	3:COMBINATIC	-61.449	740.551	-1.923	-2.920	**-0.324**	-0.863
Max MZ	32	3:COMBINATIC	41.518	715.253	-43.438	-1.985	0.226	**0.932**
Min MZ	44	3:COMBINATIC	-41.518	715.254	-43.438	-1.984	-0.226	**-0.933**

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B. Wind Force Acting on Tank

Table 3: Wind Force Magnitude at Different Levels

LEVEL OF TANK	WIND FORCE MAGNITUDE
@16.9m above the base	100.573 KN
@13.04m above the base	28.045 KN
@12.04m above the base	9.978 KN
@6.02m above the base	7.183

C. Comparision Between Manual Results and Staad Pro Results:

Table 4: Comparison between Manual and STAAD Design

MANUAL DESIGN RESULTS	STAAD pro RESULTS
TOP RING BEAM: F_{ck} = 30 N/mm^2 F_y = 500 N/mm^2 A_{st} = 1026.37 mm^2 Dia of bar = 16 mm No. of bars = 6 Size of ring beam = 0.25 x 0.40m	**TOP RING BEAM:** F_{ck} = 30 N/mm^2 F_y = 500 N/mm^2 A_{st} = 678.58 mm^2 Diameter of bar = 12 mm No. of bars = 6 Size of ring beam = 0.25 x 0.40m
BOTTOM RING BEAM: F_{ck} = 30 N/mm^2 F_y = 500 N/mm^2 A_{st} = 5890.48 mm^2 Dia of bar = 25 mm No. of bars = 14 Size of ring beam = 0.65 x 0.90m	**BOTTOM RING BEAM:** F_{ck} = 30 N/mm^2 F_y = 500 N/mm^2 A_{st} = 4908.738 mm^2 Dia of bar = 25 mm No. of bars = 10 Size of ring beam = 0.65 x 0.90m
RING GIRDER: F_{ck} = 30 N/mm^2 F_y = 500 N/mm^2 A_{st} = 2814.86 mm^2 Dia of bar = 16 mm No. of bars = 14 Size of ring beam = 0.65 x 0.985m	**RING GIRDER:** F_{ck} = 30 N/mm^2 F_y = 500 N/mm^2 A_{st} = 2199.11 mm^2 Dia of bar = 10 mm No. of bars = 28 Size of ring beam = 0.65 x 0.985m
COLUMN: F_{ck} = 30 N/mm^2 F_y = 500 N/mm^2 A_{st} = 2827.43 mm^2 Dia of bar = 20 mm No. of bars = 9 Size of ring column = 0.5m dia	**COLUMN:** F_{ck} = 30 N/mm^2 F_y = 500 N/mm^2 A_{st} = 2544.69 mm^2 Dia of bar = 18 mm No. of bars = 10 Size of ring beam = 0.5m dia
BRACE BEAM: F_{ck} = 30 N/mm^2 F_y = 500 N/mm^2 A_{st} = 1884.95 mm^2 Dia of bar = 20 mm No. of bars = 6 Size of ring beam = 0.3 x 0.50m	**BRACE BEAM:** F_{ck} = 30 N/mm^2 F_y = 500 N/mm^2 A_{st} = 628.318 mm^2 Dia of bar = 10 mm No. of bars = 8 Size of ring beam = 0.3 x 0.50m
TOP DOME: F_{ck} = 30 N/mm^2 F_y = 500 N/mm^2 A_{st} = 8 mm dia @ 110mm c/c A_{st} = 502.65 mm^2 Dia of bar = 8 mm No. of bars = 10 Thickness of dome = 0.150m	**TOP DOME:** F_{ck} = 30 N/mm^2 F_y = 500 N/mm^2 A_{st} = 8 mm dia @ 100mm c/c A_{st} = 452.92 mm^2 Dia of bar = 8 mm No. of bars = 9 Thickness of dome = 0.150m

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BOTTOM DOME: F_{ck} = 30 N/mm^2 F_y = 500 N/mm^2 A_{st} = 8 mm dia @ 100mm c/c A_{st} = 552.92 mm^2 Dia of bar = 8 mm No. of bars = 11 Thickness of dome = 0.30m	**BOTTOM DOME:** F_{ck} = 30 N/mm^2 F_y = 500 N/mm^2 A_{st} = 8 mm dia @ 100mm c/c A_{st} = 552.92 mm^2 Dia of bar = 8 mm No. of bars = 11 Thickness of dome = 0.30m

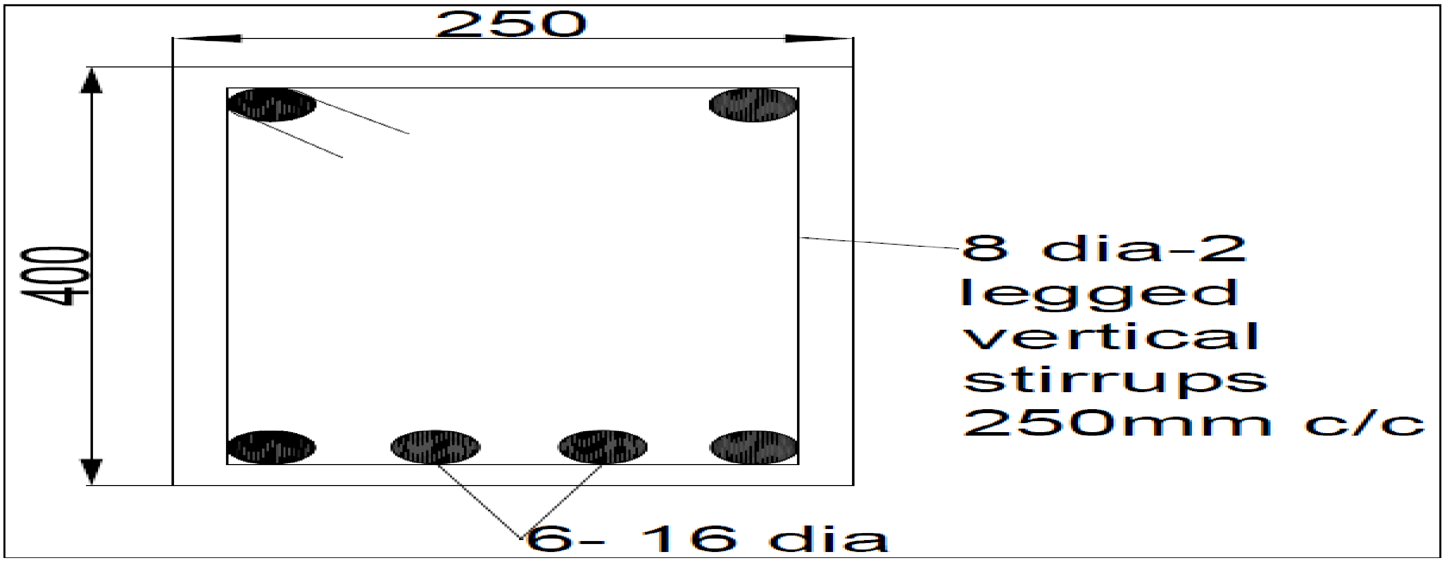

Fig 24: Top Ring Beam Detailing in Manual Design

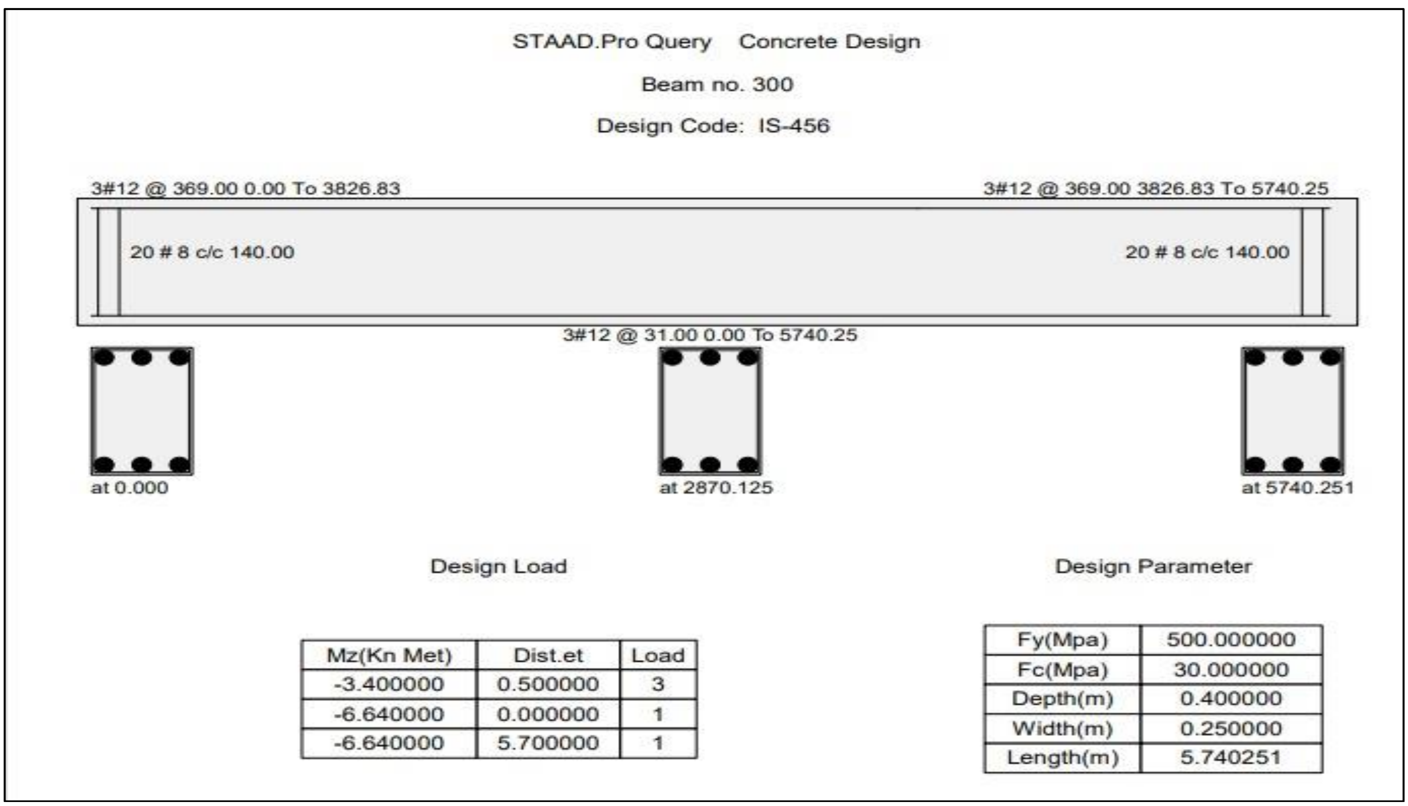

Fig 25: Top Ring Beam Detailing in STAAD Pro

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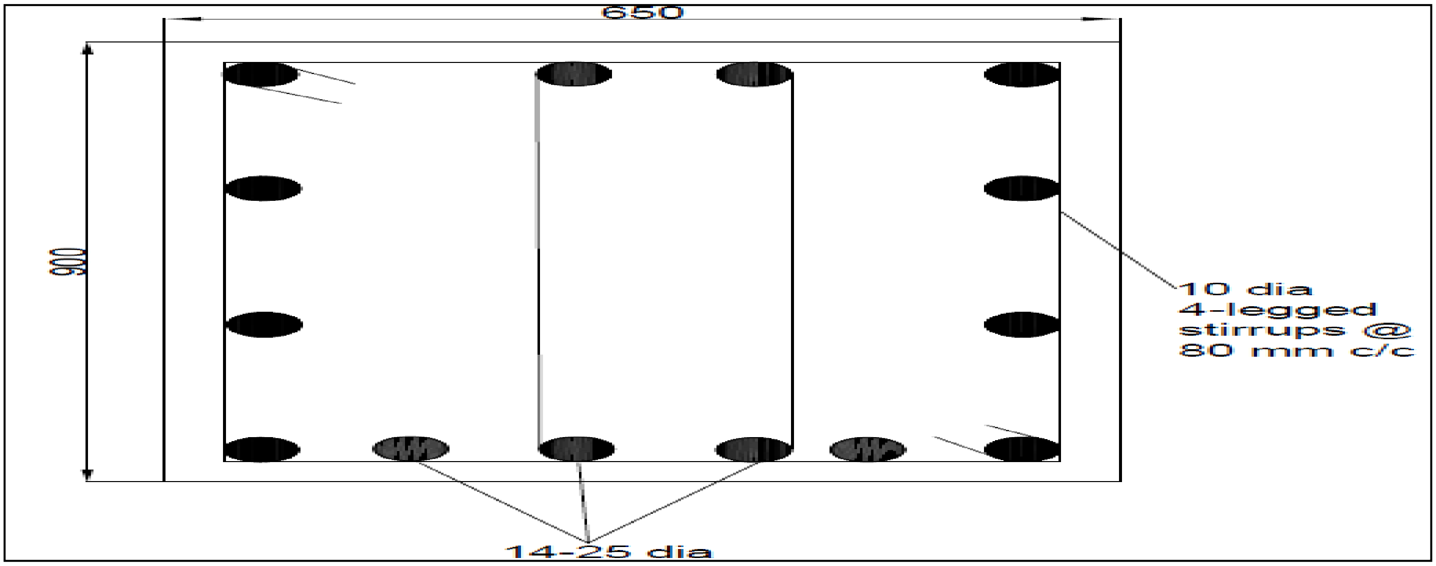
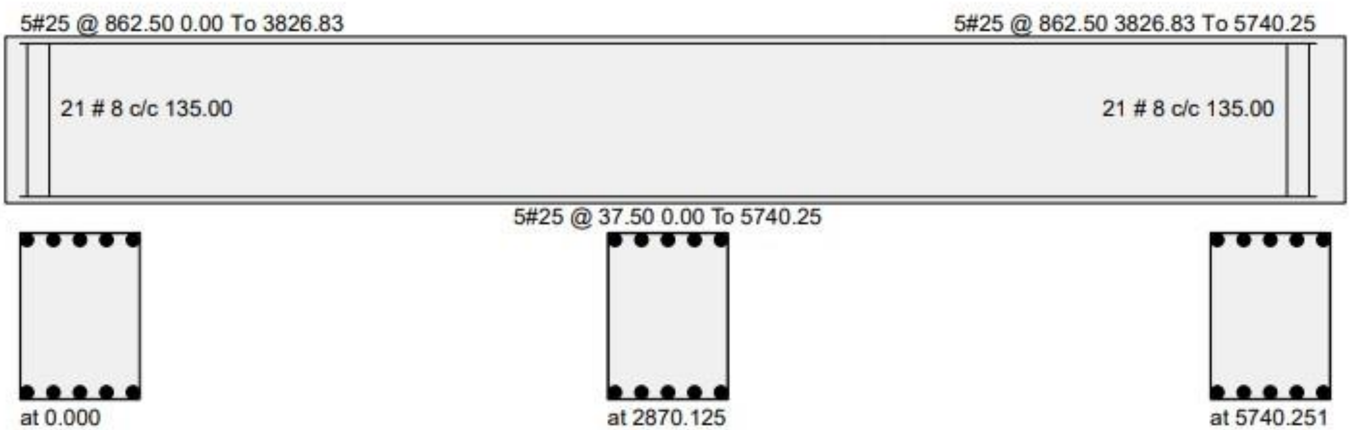


Fig 26: Bottom Ring Beam Detailing in Manual Design

STAAD.Pro Query Concrete Design

Beam no. 228

Design Code: IS-456



Design Load

Mz(Kn Met)	Dist.et	Load
22.750000	3.300000	3
-36.900002	0.000000	1
-36.900002	5.700000	1

Design Parameter

Fy(Mpa)	500.000000
Fc(Mpa)	30.000000
Depth(m)	0.900000
Width(m)	0.650000
Length(m)	5.740251

Fig 27: Bottom Ring Beam Detailing in STAAD Pro

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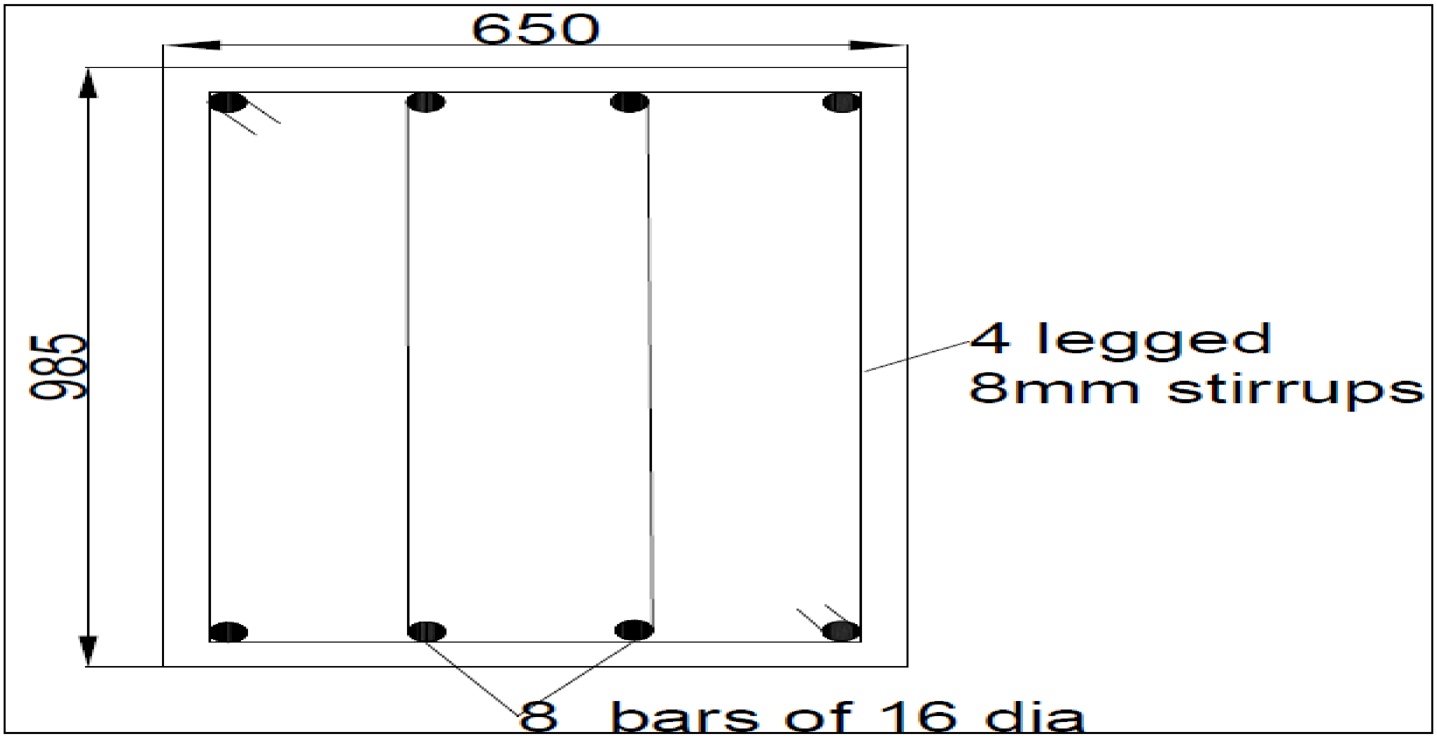
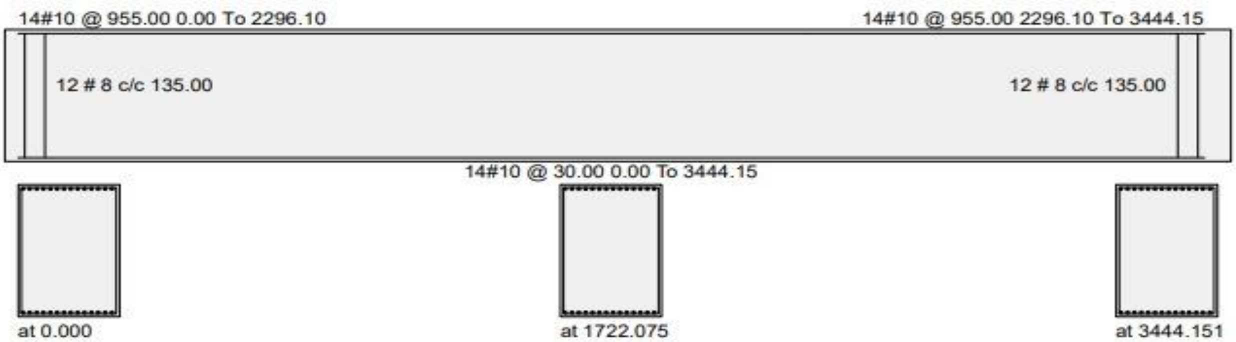


Fig 28: Ring Girder Detailing in Manual Design

STAAD.Pro Query Concrete Design

Beam no. 158

Design Code: IS-456



Design Load

Mz(Kn Met)	Dist.et	Load
-48.020000	0.300000	3
-54.930000	0.000000	3
-54.029999	3.400000	3

Design Parameter

Fy(Mpa)	500.000000
Fc(Mpa)	30.000000
Depth(m)	0.985000
Width(m)	0.650000
Length(m)	3.444151

Fig 29: Ring Girder Detailing in STAAD Design

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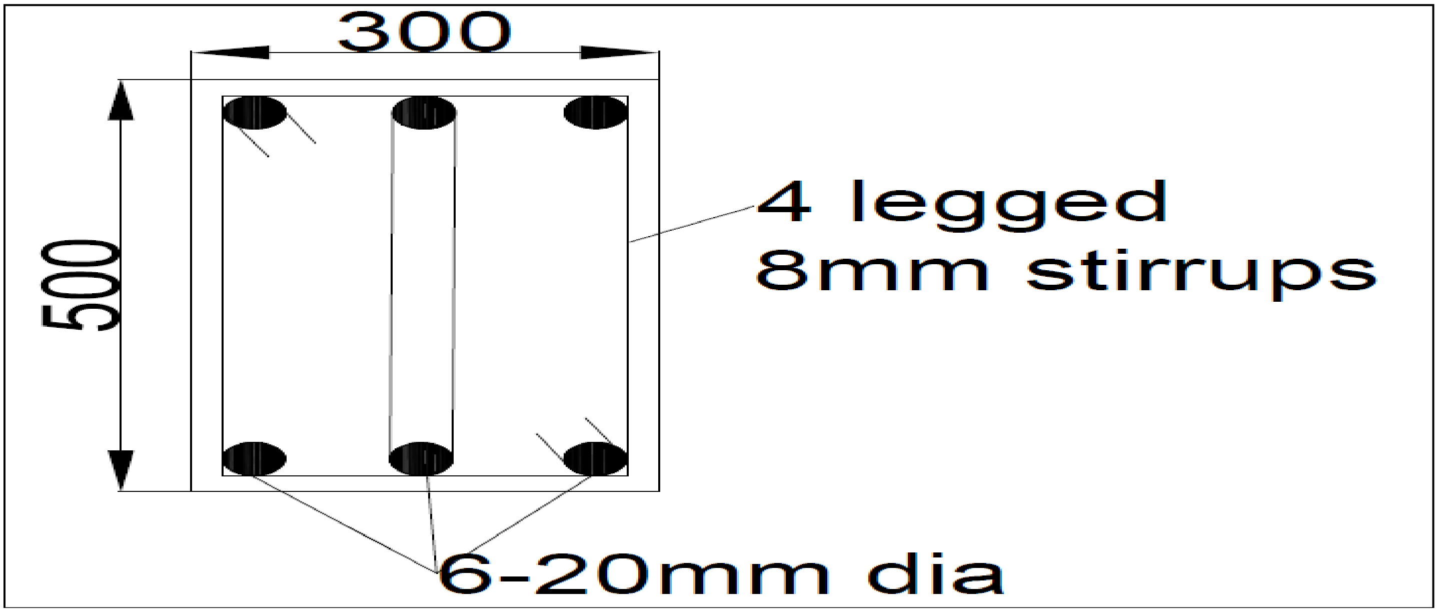

Fig 30: Brace Beam Detailing in Manual Design

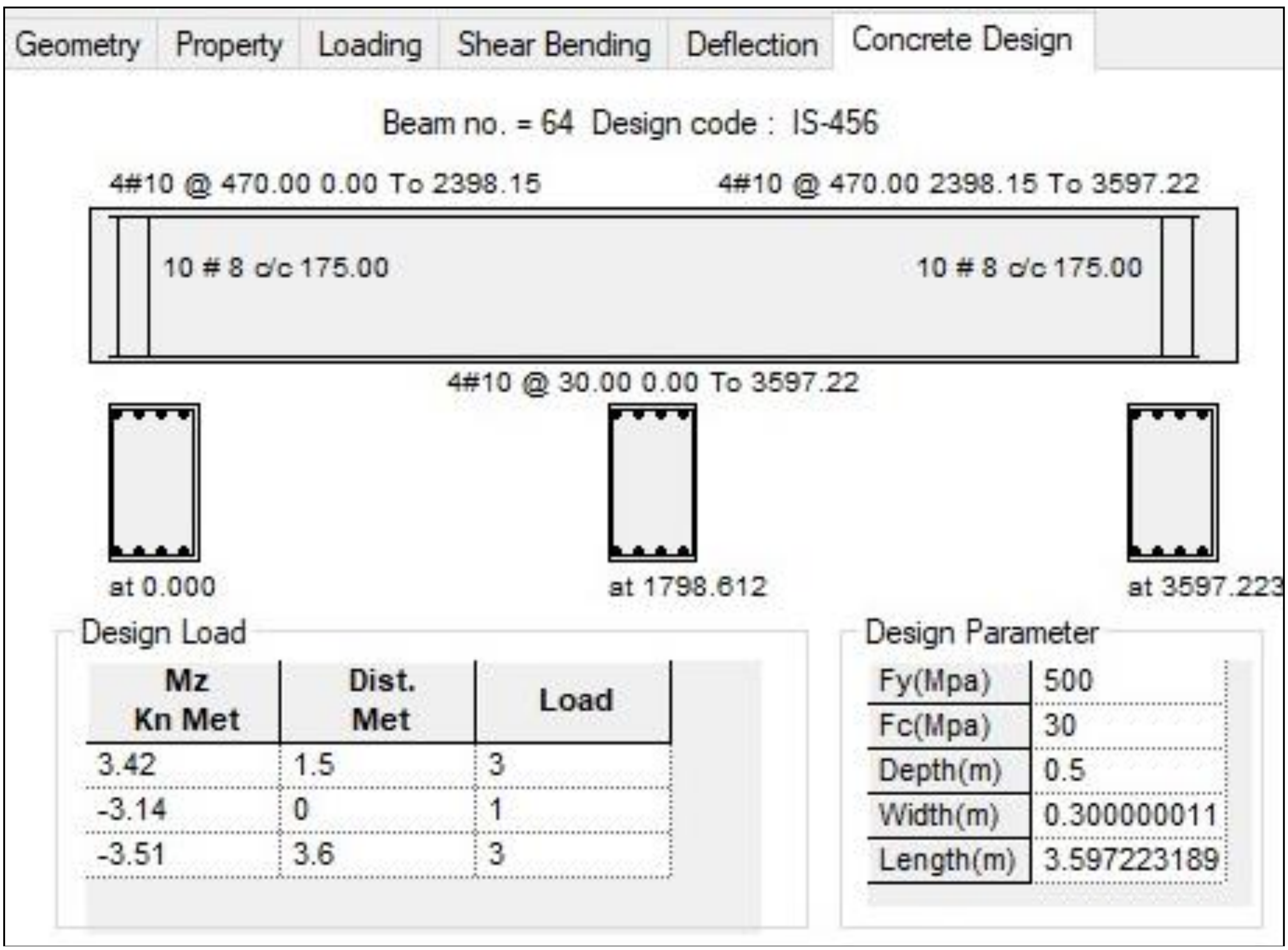

Fig 31: Brace Beam Detailing in STAAD Pro Design

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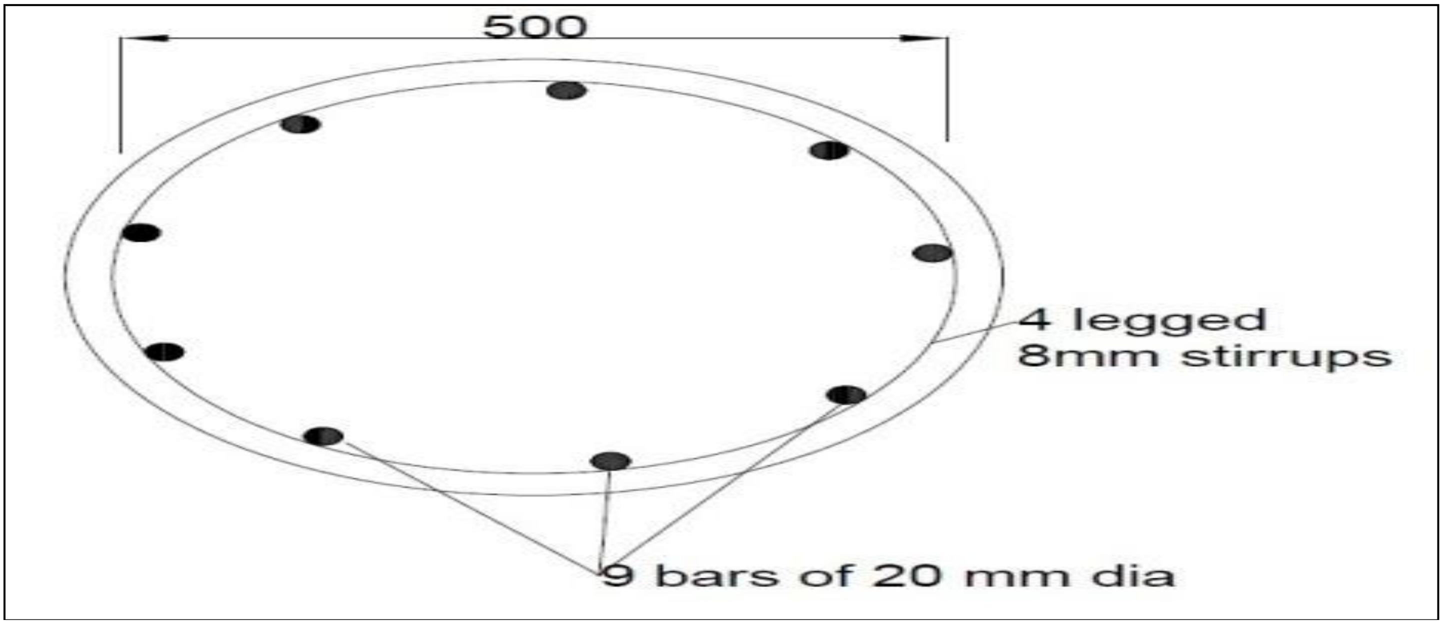
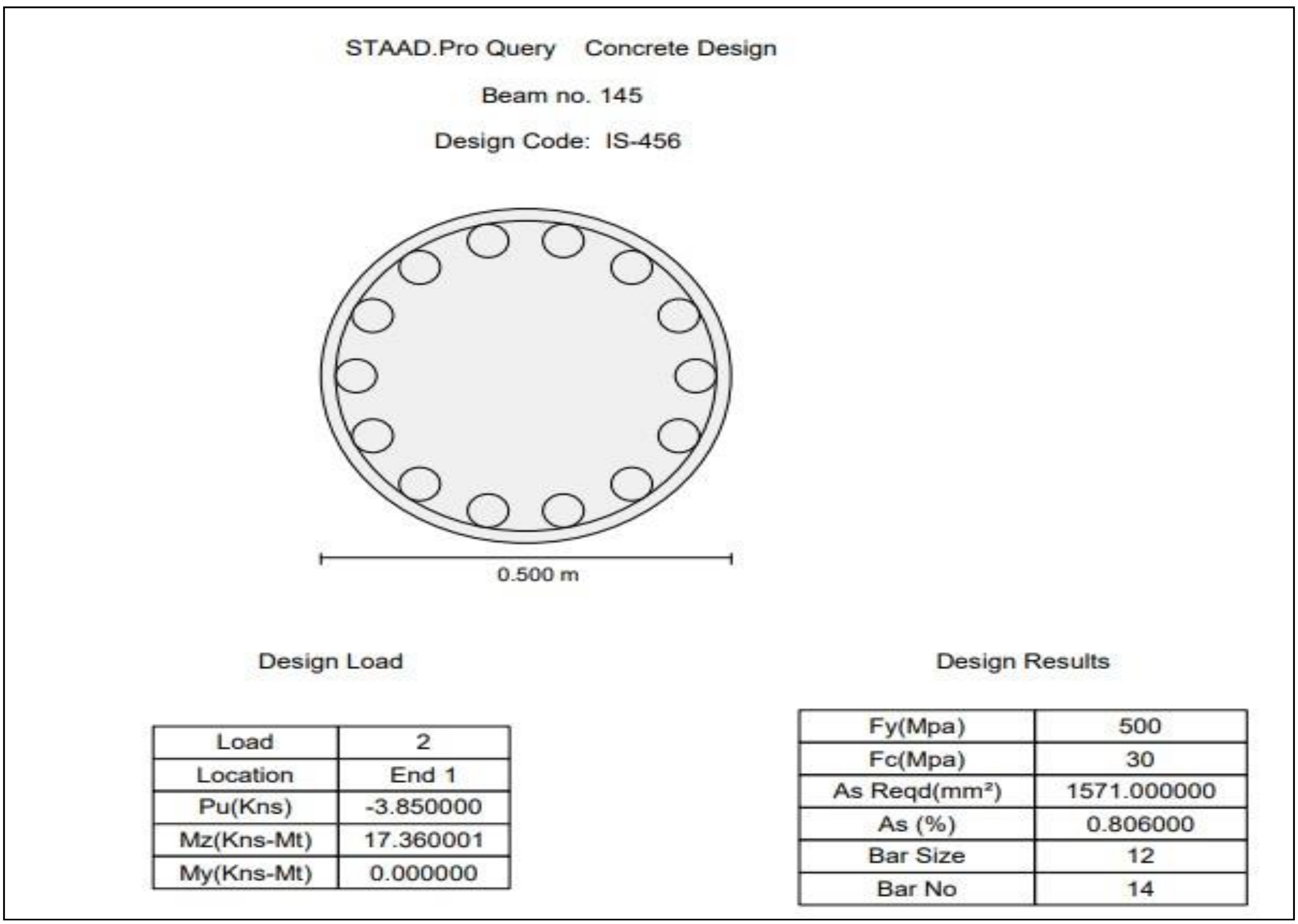


Fig 32: Column Detailing in Manual Design



Load	2
Location	End 1
Pu(Kns)	-3.850000
Mz(Kns-Mt)	17.360001
My(Kns-Mt)	0.000000

Fy(Mpa)	500
Fc(Mpa)	30
As Reqd(mm²)	1571.000000
As (%)	0.806000
Bar Size	12
Bar No	14

Fig 33: Column Detailing in STAAD Pro Design

VI. CONCLUSION

- Both STAAD pro and manual designs are safe against static and earthquake loading condition
- The wind force magnitude increases with the increase in height.
- All the designed elements are safe in both manual and STAAD pro design.
- The maximum value of shear force obtaining from manual design is 947.185 KN and that from the STAAD pro results is 775.629 KN.
- The percentage variation of manually calculated area of steel in ring beam and that done using STAAD pro software is 16.67%.
- The percentage difference between manual calculating an area of steel in column and plates doing it using STAAD pro software is 9.99%.
- So that we conclude the use of STAAD pro software gives more economical design as compared to manual design.

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