https://dx.doi.org/10.22161/ijaers.5.10.18 ISSN: 2349-6495(P) | 2456-1908(O)

Study and Analysis of Intz water tank with manual and software-based design with base isolation

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Abstract— *The seismic response of an overhead water tank, cylindrical, extra-large water storage tank by using triple friction pendulum system is analyzed. Most of the overhead tanks have a fundamental frequency which includes a series of resonance of greatest earthquake ground motions. It is an operative way to reduce the response of an isolation system used for storage of water tanks under a sturdy earthquake. However, it is problematic to implement in preparation with common isolation bearings*

The research is directed with study of existing studies in the field of seismic behavior of intz water tank. Base isolation is one of the technologies applied to decrease the consequence of earthquake effect. The principle is to separate the base of the overhead water tank from footing ground. The problematic is taken as Intz water tank design to survive water tank against seismic accomplishment. three categories of base are used to analyses and compare overhead first is manual design of intz water tank with fixed base + response by SRSS and second case is intz water tank with fixed base by sap2000 and Third case with is intz water tank with triple friction pendulum on sap2000.

The software SAP 2000 are used to assessment fixed and triple friction pendulum base intz water tank. It is primary period in India when overhead water tank is tested with triple friction pendulum isolation are analyzed for seismic zone V. It is initiate from results that deflection and base shear analyzed with triple friction pendulum are lesser than fixed base with outstanding margin and it is determined that study endorses use of triple friction pendulum base isolation for seismic zone V in India.

***Keywords*— *Intz water tank, Seismic, Fixed Support, triple friction pendulum Support, SAP2000, Deflection, base shear*.**

I. INTRODUCTION

Elevated Water tanks are salvation developments which are being constructed in growing numbers to store water for drinking purpose. The capacities of these containers are huge and have capacities of around 1800 m³ or it may be large depends on the population of that area. Elevated water tank consists of an RCC container or it may made up of steel tank, which contains the large amount of water, the seismic study of these structures is a difficult and thought-provoking task because the construction of the tank based on the with a smaller number of footing and column as compare to building and the soil erection contact must be considered. water tanks present an excessive risk if they were failed during an earthquake.

Base isolation is a demonstrated knowledge for the seismic strategy of structures. The system diminishes the probability of structural and non-structural damage to a water tank exposed to seismic forces. By using base isolation, we can reduce the lateral forces and displacement of the structure which can damage the structure through earthquake. Due to which we can save the government property and water which get distributed to people.

However, in spite of base isolation’s safety benefits, the technology is under operated. Although tall, flexible, and non-critical facilities such as office buildings are not the most ideal candidates for base isolation, they may still achieve an optimal seismic design by using the technology. Therefore, in order to increase the quality and prevalence of base isolated structures, there is a need to study the technology’s seismic performance enhancements and cost effectiveness for projects on which the system is infrequently used.IS 1893:2002 is the code to design structures under earthquake zones. There are two major methods of seismic analysis which are

1. **Response Spectrum Analysis:** This Analysis is based on ideal predefined statistics which are not actual time data collected since actual earthquake in the part.

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a) SRSS b) CQC

2. **Time History Analysis:** This Analysis is based on genuine real time data composed under actual earthquake. Elevated water tank response and behavior is composed in real time and can be used to design future elevated water tank under seismic loading.

Need of study

1.Baseisolation technique is newly isolated structure which is provided at the base of structure.it is only performed on building and hospital etc. None of the research is done on base isolation on elevated water tank.

2.So the approach is done on Base isolation for Elevated water tank or manual and software comparison.

II. PROBLEM DESIGN

The design of overhead Intze Water Tank is carried out using the manual and computer aided design software sap2000 Elevated storage reservoir. The design is carried out as per relevant analysis procedures combined with Indian Standard Codes of Practices. The water tank dome is designed by working Stress's method. The foundation forces at the level of safe bearing capacity are also evaluated The software also gives the shape description of the tank and keeping various constraints, one can change the governing constraint to get the optimum result and safe design with economy.

PLAN DATA: Structural design of intz water tank of capacity 900000 liters.

Location of site: BHUJ (GUJARAT)

Type of tank: Intze water tank

Staging System Chosen: Column Braced

Geometrical Data

Seismic Zone: V

Soil properties

Soil Description: Medium soil

Safe Bearing Capacity at Depth 1 m: 150 Kn/m²

Manual Design of intze water tank :-

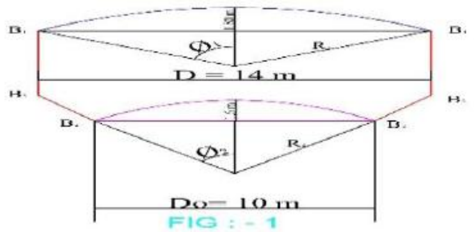

Fig. 1: Dimension of intz tank

Volume of water Tank = 900000.00 litre capacity

Height of Staging =16.00 m

Suppose the Diameter of Cylindrical's portion = D = 14.00 m

And Radius of Cylindrical's Portion R = 7.00 m

Suppose the Diameter of Ring Beam B2 = Do = 10.00 m

And Radius of Ring Beam B2 = Ro = 5.00 m

Suppose Height h_o of Conical Dome = 2.00 m

Suppose Rise h1=1.800 m ; Rise h2 = 1.600 m

The Radius R2 of lowest dome is given by

$h2 * (2 * R2 - h2) = R^2 0$

$1.6 * (2 * R2 – 1.6) = 5^2$

Radius of lowest dome R2=8.610 m

sinØ2 = 5/8.61 = Ø2= 35.500°

cosØ2 = 0.8141 tanØ2 = 0.71330

suppose h be the height of cylindrical's portion

From which h = 4.780 m

Permitting for free board keep h = 5.00 m

For the top dome, the Radius R1 is given by

$h1* (2 * R1 – h1) = R^2 0$

$1.600 * (2 * R1 – 1.600) = 7.00^2$

Radius of lowest dome R1=14.510 m

SinØ1 = 5.00/14.510 = 0.48240

Ø1 = 28.840°

CosØ1 = 0.87600

2 - DESIGN OF TOPMOST DOME

Suppose Thickness t1 = 100.00 mm

Taking Live load = 1500.00 N/m²

Total P per sq.m of dome = 0.1 x 25000.00 + 1500.00

Ptotal= 4000.00 N/m²

Meridional's Thrust's at edges

T1= P*R1/1+ CosØ1

T1= 4000.00 * 14.510 / 1.00+00.8760 = 30938.00 N/m

Meridonal's.Stress's per metre = T1/t x d

Meridonal's.Stress's per metre = 30938.00 / 100.00 x 1000.00

Meridonal's.Stress's =00.31 N/mm² -: safe

Extremes hoops Stress's arises at the centre and its magnitude's = P*R1/t1 x 2

=4000.00 x 14.510/2.00 x 0.10=290200.00 N/m²

Extremes hoops Stress's= 00.29 N/mm² -: safe

Since's the Stress's are in safe limit, offer's nominal reinforcements @ 0.3 %

As = 00.30 x 100.00 x 1000.00 / 100.00

As = 300.00 mm²

Using 8 mm Ø bar , AØ = 50.00 mm²

Space = 1000.00 x 50.00 / 300.00 = 160.00 mm

-: 8.00 mm Ø bar @ 160.00 mm c/c in both direction

3 DESIGN OF TOPMOST RING BEAM B1

Horizontal Element of T1 is given by

P1 = T1 * CosØ1

P1 = 30938.00 x 00.8760 =27102.00 N/m

Whole tension's tending's to ruptures the beams = P1 x D/2

Ac = 27102.00 x 14.00 / 2.00 =189712.00 N

Whole's tensions tendings to rupture's the beam = 189712.00 N

Permissible Stress's in HYSD bars = 150.00 mm²[IS 456 :2000]

Ash = Whole tension tending to rupture the beam/ Permissible Stress's hysd bars

Ash = 189712.00 / 150.00 = 1265.00 mm²

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Ash actual = 1265.00 mm^2
No. of 20.00 mm Ø bars = Ash actual / Area of bar
No. of 20.00 mm Ø bars = 1265.00/ 314.160
No. of 20.00 mm Ø bars = 4.00
Actual Ash offerd = 314.160 x 4.00 =1257.00 mm^2
The areas of cross sections of rings beams is given by
Ac =Whole tension tending to rupture the beam/A+(m-1) x Ashp
= 189712.00 / A + (13.0-1.0) x 1257.00 =A = 143014.00 mm^2
Offer ring beam of 360.0 mm depth and 400 mm width. Tie the 20 mm Ø rings 6.00 mm Ø Nominal stirrups @ 200.00 mm c/c
Topmost Ring Beam =400.0x360.0 =144000.0 mm^2 **-: Safe**

4 DESIGN OF CYCLINDRICAL's WALL's

In the membrane analysis , the tank is projected to be free at top and bottom. Extreme hoops tension's occur's at the base of wall ,its magnitude is given by :-

$$P = w * h \times D / 2$$
$$P = 9800.00 * 5.00 \times 14.00 / 2.0$$
$$P = 34300.00 \text{ N/m}$$

Area of Steel Ash = P / Permissible Stress's
Ash = 34300.00 / 150.00
Ash = 2286.00 mm^2 per metre height
Provided that ring's on both the faces,
Ash on each-face = 2286.00/2.00
Ash on each-face = 1143.00 mm^2
Space of 12 mmØ rings@per m =1000.00 x 113.00 / 1143.00 =98.90 mm
Offer 12 mm Ø rings @ 95 mm c/c at bottom. This space can be increased at the top
Actual Ash offerd = 1000.00 x 113.00 / 95.00
Actual Ash offerd = 1190.00 mm^2 on each-face
Permitting 1.200 N/mm^2 Stress's on composite section
1.2= At /1000.00 x t +(m-1) Ash offerd x 2.00
1.2= 343000.00 / 1000.00 x t + (13.0-1.0) x 1190.00 x 2.00
t = 257.330 mm
minimum thickness = 3H' +5.00 = 3x5.00 + 5.00 =200.00 mm
Average t = 300.00+200.00/ 2.00 = 250.00 mm
% of distribution steel =0.30 [250.0-100.0/450.0-100]x0.1 =0.24
Ash offerd = 0.24 x 250 x 1000 / 100 = 650.00 mm^2
Area of steel on each-face = 325.00 mm^2
Space of 8 mm Ø bar = 1000.0 x 00.785 x 8^2 / 325 = 155. mm
-: offer 8.00 mm Ø bars @ 150.00 mm c/c on both faces
To resist the hoop tension at 2 m below top

Ash =2.00 x 2286.00 /5.00
Ash = 914.400 mm^2

Space of 12.00 mm Ø rings = 1000.00 x 113.00 /914.40/2.00
Space of 12.00 mm Ø rings = 247.00 mm
-: offer Space of 12.00 mm Ø rings 240.00 mm c/c in the top 2.00 m height's
At 3.00 m below's the top

Ash =3.00 x 2286.00 /5.00
Ash = 1372.00 mm^2

Space of 12.00 mm Ø rings = 1000.00 x 113.00 /1372.00/2.00
Space of 12.00 mm Ø rings = 164.700 mm
-: offer Space of 12.00 mm Ø rings 160.00 mm c/c in the next 1.00 m height's
At 4 m below's the top

Ash =4.00 x 2286.00 /5.00
Ash = 1829.00 mm^2

Space of 12.00 mm Ø rings = 1000.00 x 113.00 /1829.00/2.00
Space of 12.00 mm Ø rings = 123.60 mm
-: offer Space of 12.00 mm Ø rings 120.00 mm c/c in the next 1.00 m height's
-: offer Space of 12.00 mm Ø rings 95.00 mm c/c as found earlier

5 DESIGN OF RING BEAM B3

The ring beam joins the tank wall through conical dome. The vertical load at the junction of the wall with conical dome is shifted to ring beam B3 by meridional's thrust's in the conical dome. The horizontal's element of the thrust's causes hoop's tensions at the joint The ring beam is offerd to take up this hoops tensions refer fig 2 the load W transmitted through tank wall at the top of conical dome consist of the following

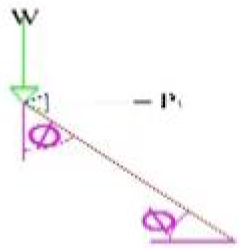

Fig. 2: Load Transmitted

1) Load's of top dome = T1 SinØ1 = 30938.00 x 0.4824.00 = 14924.00 N/m
2) Load's due to the ring beam B1 = 0.360 x (0.40 – 0.20) x 1.00 x 25000.00 = 1800.00 N/m
3) Load's due to tank wall = 5.00 (0.20 + 0.30 / 2.00) x 1.00 x 25000.00 = 31250.00 N/m
4) Self 'sload's of beam B3 = (1.00 – 0.30) x 0.60 x 25000.00 = 10500.00 N/m

Entire W=14924.0 + 1800. 0+31250.0 +10500.0 =58474.0 N/m`
Angle of conicals domes wall with Vertical Ø^o = 45.00^o

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sinØ° = cosØ° = 0.70710 tan Ø° = 1.00

Pw = W x tan Ø° = 58474.00 x 1.00

Pw= 58474.00 N/m

Pw1 = w x h x d3 = 9800.00 x 5.00 x 0.600

Pw1 = 29400.00 N/m

-: hoops tension's in the rings beams is given by

P3 = (Pw + Pw1)x D/2

P3 = (58474.00 +29400.00) x 14.00 /2.00

P3 = 615118.00 N

This to be resisted entirely by steel hoops, the area of which is

Ash = P3 / permissible Stress's

Ash = 615118.00 / 150.00 = 4100.00 mm²

Number of 28 mm Ø bars = 4100.00/615.75.00

-: offer 7 rings of 28 Ø bars

Actual Ash = 0.785 x 28 ² x 7

Ash = 4310.26 mm²

Stress's in equal section = P3 / d x h + (m-1) x Ash offerd

Stress's in equal section = 615118.00 / (1000.00 x 600.00) + (13.0 -1.0) x 4310.260

Stress's in equal section = 00.940 N/ mm² < 1.200 N/mm² safe

8.00 mm Ø distribution's bars offerd in the wall @ 150.00 mm c/c should be taken to rounded off the above ring acts as stirrups

6 DESIGN OF TAPERING DOME

1)Meridional's thrusts; - the weight's of water (fig 3)

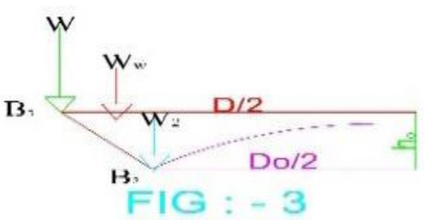

Fig. 3: Forces at conical section

$Ww = \frac{\pi}{4} D^2 * h + \frac{\pi}{12} * ho\ (D^2 + Do^2 + D * Do) - \frac{\pi}{3} h *$ 2²(3R2-h)

Ww = 4392368.00 N

Suppose the thickness of conical slab be 400.00 mm

-: Total Self Weight Ws is given by

$Ws = [\pi * \left(\frac{D+Do}{2}\right) x\ l\ x\ to]\ x\ \gamma c$

Ws = 25000.00 x 3.140 (14.00+10.00/2) x 2.820 x 0.40

Ws = 1066131.00 N

Weight W at B3 = Pw

W = 58474.00 N/m

-: Perpendicular load W2 per metre run is given by

$W2 = \frac{\pi * D * W + Ww * Ws}{\pi Do}$

$W2 = \frac{\pi\ x\ 14.00\ x\ 58474 + 4392368.00 + 1066131}{\pi x\ 10.00}$

W2 = 255613.00 N/m

Meridional's thrusts To in the conical dome

To = W2/cosØo

To = 255613.00 x 1.4140

To = 361437.00 N/m

Meridional's Stress's = To / b*d

Meridional's Stress's = 361437.00 / 1000.00 x 400.00

Meridional's Stress's = 0.900 N/mm²

00.90 N/mm² < 1.200 N/mm² -: safe

b) Hoop Tension: - Fig no 3 Diameter of tapering dome at any height h' above base is D' = 10.0 + (14.0-10.0/2)h' = 10.0 + 2h'

Intensity's of water pressure P= (5.00+2-h') x 9800.00 N/m²

Hoop's tensions P'o is given by

P'o = (P/cosØo + q x tanØo)D'/2

P'o = [(5.0+2-h') x 9800.0 x1.414.0 +1000 x 1{10.0 + 2h'/2.0}]

P'o = 535075.00 + 37720.00 x h' – 13859h'²

The value of P'o at h'=0 ,h'=1 and h'=2 are tabulated below

h'	Hoop Tensions
0	535075.00 N
1	558936.00 N
2	555079.00 N

Table .1: Hoop tension

For Maxima , d P'o /dh'= 0

37720.00 – 2.0 x 13859.00 x h' = 0

From Which **h' = 1.3610 m**

Max Po' = 535075.0 + 37720.0 x (1.3610 – 13859.0) x (1.3610)²

Max Po' = 560739.00 N

C) Design of Walls : -

Meridonal's.Stress's = 0.900 N/mm²

Max Hoop Stress's = 560739.00 N

Whole of Which is to be resisted by steel

As = max hoop Stress's / permissible Stress's

As = 560739.00/150.00 = 3738.00 mm²

Area of Each-face = 3738.00 / 2.00 =1869.00 mm²

Space of 16.0 mm Ø bars=1000.0 x0.7850x 16.00²/100. =107.05

Space of 16.0 mm Ø hoops @ 100.0 mm c/c on each-face

Actual Ash = 1000.0x0.7850 x 16.0² / 100.0 =2010 mm²

Ash offerd = 2010.00 mm²

Max. tensile Stress's in composite's section

= 560739.00 / (400.0 x 1000.0) + (13-1) x 2010.0 x 2.0

Max. tensile Stress's in composite section = 1.3850 N/mm²

This tensile Stress's is more than the permissible, value 1.20 N/mm².-: increase the thickness **420.0 mm. this will reduce the tensile Stress's to 1.1980 N/mm² -: safe**

In the meridional direction offer reinforcement @ {0.30- [420.0-100.0/420.0-100.0] x 0.10} =0.210%

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Asd =0.21 x 4200 =882.00 mm²

Asd on each-face = 882.00 /2.00 = 441.00 mm²

Space of 10 mm Ø bars = 1000.0x 0.7850 x 10²/441.0 = 178 mm

-: offer 10.00 mm bars @ 175.00 mm c/c on each-face clear- cover 25.00mm

7 DESIGN OF LOWEST DOME

Lowest dome develops compressive Stress's both meridionally's as well as hoop's, due to weight's of water buoyed by it and also due to its own weight

R2 = 8.610 m; sinØ2 = 0.58070; cosØ2 = 0.81410

Weight of Water Wo of the dome is given by

Wo=[$\frac{\pi}{4} Do^2 Ho - \frac{\pi}{3} h2^2$(3R2-h2)] x w

Wo=[$\frac{\pi}{4} 10^2 x7 - \frac{\pi}{3} x1.6^2$(3x8.610-1.60)] x 9800.00

Wo= 4751259.00 N

Let the thickness of bottom dome be 250.00 mm

Self-Weight = 2 x3.140 x R2 x h2 x t2 x γc [γc = 25000]

Self-Weight = 2 x3.140 x 8.610 x 1.60 x 0.250 x 25000.00

Self-Weight of dome = 540982.00 N

Total Weight WT = Ww +Wo

Total Weight WT = 540982.00+ 4751259.00

Total Weight WT = 5292241.00 N

Meridional's Thrust T2 =WT/3.140 x Do x sinØ2

Meridional's Thrust T2 = 5292241.0 / 3.140 x 10.0 x 0.58070

Meridional's Thrust T2 = 290093.00 N/m

Meridional's Stress's = T2 / b*d

Meridional's Stress's = 290093.00 / 250.00 x 1000.00

Meridional's Stress's = 1.160 N/mm²

1.160 ≤ 1.200 N/mm² -: safe

Intensity of loading per unit area = P2 = WT /2 x 3.140 x R2 x h2

P2 = 5292241.00 / 2.00 x 3.140 x 8.610 x 1.60

P2 = 61142.00 N/m²

Max. hoop Stress's at Centre of dome = P2*R2 /2*t2

Max. hoop Stress's at Centre of dome = 61142.0 x 8.61 / 2x 0.25

Max. hoop Stress's at Centre of dome = 1.050 N/mm² Safe

Area of minimum steel = 0.3 – [250-100/450-100] x 0.1 = 0.26 %

As =0.260 x 2500.00 = 650.00 mm² in each direction

Space of 10 mm Ø bars = 100.0 x 0.785 x 10² / 650 =121.00 mm

-: offer Space of 10.00 mm Ø bars @ 120.0 mm c/c in both directions also offer 16.0 mm Ø meridionals bar @ 100.0 mm c/c near water face. For 1.0 m length to take care of the continuity effect the thickness of the dome may be increased from 250.0 mm to 280.0 mm gradually in 1.0 m length

8 DESIGN OF LOWEST CIRCULAR BEAM B2;-

Inner thrust from tapering dome = To x sinØo

Inner thrust from tapering dome = 361437.0 x 0.7070

Inner thrust from tapering dome = 255613.0 N/m

outer thrusts from bottom dome = T2 x sinØ2

outer thrusts from bottom dome = 290093.0 x 0.81410

outer thrust from bottom dome =236165.00 N/m

Net Inner thrusts = Inner – Outer

Net Inner thrusts = 255613.00 – 236165.00

Net Inner thrust = 19448.00 N/m

Hoop compression in beam = 19448.00 x 10 / 2 =97240.00 N

Suppose the sizes of beams be 600.0 mm x 1200.0 mm

Hoop Stress's = 97240.0 / 600.0 x 1200.0 = 0.1350 N/mm²

0.1350 N/mm² ≤ 1.20 N/mm² Safe

Vertical load on beam per metre run = To x sinØo + T2 x sinØ2

Vertical load on beam per metre run = 255613+290093 x 0.5807

Vertical load on beam per metre run = 424070 N/m

Self-weight of beam = b x d x γc

Self-weight of beam = 0.6 x 1.20 x 1 x 25000.00

Self-weight of beam = 18000.00 N/m

The load on beam = W = vertical loading + self-weight = 424070.00 + 18000.00 = 442070.00 N/m

The loading on beam =W=442070.00 N/m

Let,s us support the beam on 8.00 similarly spaced column at a mean radii of lowest curved beam R = 5.00 m

2ϴ = 45° ; ϴ = 22.5°

C1=0.0660: C1=0.0300; C1=0.0050 [IS CODE TABLE 20.1]

Øm= 9.50°

MO = SUPPORT's MOMENT's B.M -VE = C1 X WR² X 2ϴ

MO =0.066 x 442070.0 x 5.0² x 0.7580

MO = 572882 Nm

Extreme + ve B.M at support = Mc = C2 X WR² X 2ϴ

Mc = 0.030 x 442070 x 5² x 0.785

Mc = 260401 Nm

Extreme To0rsional moment Mm = C3 X WR² X 2ϴ

Extreme Torsional moment Mm =0.005 x 442070 x 5² x 0.785

Extreme Torsional moment Mm = 43400 Nm

For M-20 concrete [IS 456:2000]

σcbc = 7.00 N/mm² HYSD bars σst = 150.00 N/mm²

We have K=0.3780 : j = 0.8740 ; R=1.1560

-: effective depth = $\sqrt{\frac{572882 \ x\ 1000}{600\ x\ 1.156}}$

effective depth = 909.00 mm

-: keep total depth = 1200.00 mm from shear point of view

suppose d = 1140 mm

Max shear force at support , Fo = W*R*ϴ

Fo = 442070.00 x 5.00 x 3.14/8

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Fo = 868002.00 N

SF at any point is given by F = WR(Θ – Ø)

At Ø = Øm ; F = 442070 x 5 x (22.50° - 9.5°) x 3.14 / 180

F = 501512.00 N

BM at the point of extreme torsional's moment's

Ø = Øm = 9.5° is given by

MØ = WR² (Θ x sinØ + Θ x cotΘ x cosØ – 1)

MØ = 442070 x 5² (Π/8 x sin9.5° + Π/8 x cot22.5° x cos9.5° - 1)

MØ = - 1421.00 Nm (sagging)

At the support Ø = 0

Mo = WR² (Θ - Ø) = 0

At mid span Ø = Θ = 22.5° = $\frac{\pi}{8}$ radians

MØ = WR² (Θ x cosΘ – Θ $\frac{cos\emptyset}{sin\emptyset} sin\emptyset$) = 0

At the support Mo = WT

Mo = 572882 Nm

At the Mid span

Mc = 260401 Nm sagging +ve

At the point of max. torsion (Ø=Øm =9.5°)

MØ = 1421 Nm

M_m^t = 43400 Nm

Foremost and Longtudinals Reinforcements

a) Section at point of extreme torsion

T = M_{max}^t = 43400 Nm ; MØ=M= 1421 Nm

Me1 = M + MT

MT = 76588 Nm

Me1 = 1421 + 76588 =78009 Nm

Ast1 =78009 x 100 / 150 x 0.874 x 1160

Ast1 = 513 mm²

No. of 25 mm Ø bars = 513 / 491 =1.05

Let us offer a minimum of 2 bars

Since MT > M

Me2 = MT – M

Me1 = 76588 – 1421

Ast2 = $\frac{75167 \ x \ 1000}{150 \ x \ 0.874 \ x \ 1160}$

Ast2 = 494.30 mm²

Number of 25 mm bar = 1 offer minimum of 2 bar at the pint of extreme torsion ,offer 2 -25 mm Ø bar each at top and bottom

b) Section at extreme hogging B.M (support)

Mo = 57882 Nm M_o^t=0

Ast = $\frac{572882 \ x \ 1000}{150 \ x \ 0.874 \ x \ 1140}$

Ast = 3767 mm²

No. of 25 mm Ø bar = $\frac{3767}{\pi x \ 25^2/_4}$ = 8

Then Offer 6 no.s of 25 mm Ø bars in one layer and 2 layer in the 2nd layer offer at top of the section near support

c) Section at max. sagging B.M (mid span)

Mc = 260401.00 Nm : M_c^t=0

For positive B.M ,Steel will be to th other fac , Where Stress's in steel (σst) can be taken as 190.00 N/mm². The Constant for M-20 concrete having σcbc = 7.00 N/mm², m = 13.00 will be

K = 0.324 : j = 0.892 : R = 1.011

Ast = $\frac{260401 \ x \ 1000}{190 \ x \ 0.892 \ x \ 1160}$

Ast = 1325.000 mm²

No. of 25 mm Ø bars = $\frac{1325}{\pi x \ 25^2/_4}$ = 2.7

-: the structure of reinforcements will be follow's at the support, offer 6.0 – 25.0 mm Ø at top layer and 2.0- 25.0 mm Ø bar in the 2nd layer continue these up to section of extreme torsion at a distance = R x Øm = 5. 00x 0.166 = 0.830 m

Ld = 52 x Ø = 52 x 25

Ld = 1300.00 mm from support

At this point , discontinue 4 bars while the remaining 4 bar similarly offer 4 bar 25.00 mm Ø

At the bottom throughout the length. these bars will take care of both the max positive B.M as well as extreme torsional moment

Transverse Reinforcement: -

a) At point of max. torsionals moments

At point of max. torsion V = 501512.00 N(step – 8 value obtained)

Ve = V + 1.6 x $\frac{T}{b}$ Where T= M_m^t = 43400.00 Nm

b = 600.00 mm

Ve = 501512.00 +1.60 x $\frac{43400.00}{0.60}$

Ve = 617245.00 N

τve = **Ve / b x d**

τve = 617245.00 / 0.60 x (1200.00 – 400.00)

τve = 0.887 N/mm²

This is less than τmax = 1.800 N/mm²

for M-20 concrete (IS 456 : 2000 table 20.8)

τc = 0.230 N/mm²

Since τve > τc, Shear reinforcement is necessary

The area of cross section Asv of the stirrups is given by

Asv =$\frac{T \ x \ Sv}{b \ x \ d \ x \ \sigma sv} + \frac{V \ x \ Sv}{2.5 \ x \ d \ x \ \sigma sv}$

Where b1 = 600 – (40 x 2) – 25 = 495.00 mm

d1 = 1200 – (40 x 2) – 25 = 1095.00 mm

$\frac{Asv}{Sv} = \frac{43400}{495 \ x \ 1095 \ x \ 150} + \frac{501512}{2.5 \ x \ 1095 \ x \ 110}$ = 1.755

Minimum Transverse's reinforcements is governed by

$\frac{Asv}{Sv} \geq \frac{\tau ve - \tau c}{\sigma sv}$ x b

$\frac{Asv}{Sv} = \frac{0.887 - 0.23}{150}$ x 600.00

$\frac{Asv}{Sv}$ = 2.628

-: Depth $\frac{Asv}{Sv}$ = 2.628

Using,s 12.0 mm Ø 4 legged stirrups , Asv =4 x $\frac{\pi}{4} \ x \ 12^2$=452.0 mm²

Or Sv = $\frac{452}{2.628}$ = 172.00 mm

But, the space should not exceed the least of X1 , $\frac{X1+Y1}{d}$ and 300 mm

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-: offer 12 mm Ø 4 legged stirrups @ 170 mm c/c

b) At the point of max. shear (support)

At support ; Fo = 868002 N

$\tau v = \frac{868002}{600 \times 1160} = 1.25$ N/mm²

At Support ,

$\frac{100 \times As}{bd} = \frac{100 \times 8 \times 491}{600 \times 1160} = 0.564$

τc = 00.310 N/mm² , Shear Reinforcement is necessary

Vc = 00.310 x 600.00 x 1160.00 = 215760.00

Vs = Fo – Fs = 868002.00 – 215760.00 = 652242.00 N

Space of 10.0 mm Ø 4 legged stirrups having Asv = 314.0 mm²

Given by

$Sv = \frac{\sigma sv \times Asv \times d}{Vs}$

$Sv = \frac{150 \times 314 \times 1160}{652242} = 83.80$ mm

-: offer 12 mm Ø 4 legged stirrups

$Asv = 4 \times \frac{\pi}{4} \times 12^2 = 452.39$ mm² at space

$Sv = \frac{150 \times 452.39 \times 1160}{652242} = 120.0$ mm

C) **At Mid span**

At the mid span, SF is Zero -: offer minimum shear reinforcement given by

$\frac{Asv}{bSv} \geq \frac{0.4}{fy}$

$\frac{Asv}{Sv} = \frac{0.4 \times b}{fy}$ for HYSD bar fy =415 N/mm²

$\frac{Asv}{Sv} = \frac{0.4 \times 600}{415} = 0.578$

Choosing 10 mm Ø 4 legged stirrups Asv =314 mm²

$Sv = \frac{314}{0.578} = 543$ mm

Max. permible space 0.75d = 0.75 (1200-40) = 870 or 300 mm

Whichever is less -: offer 10 mm Ø 4 legged stirrups @ 300 mm c/c

Side Reinforcement: -

Since the depth is more than 450 mm, offer side face reinforcement @ 0.1 %

$At = \frac{0.1}{100} \times 600 \times 1200 = 720$ mm²

Offer 3-16 mm Ø bar on each-face having total At = 6 x 201 =1206 mm²

9 DESIGN OF COLUMNS

The tank is supported on 8 columns symmetrically placed on a circle of 10 m mean diameter. Height of staging above ground level is 16 m let us divide this height into four panels each of 4 m height. Let column connected to raft foundation by means of a ring beam, the top of which is offerd at 1 m below the ground level, so that the actual height of bottom panel is 5 m.

A) Vertical loads on columns :-

1) Weight of water = Ww + Wo = 4392368 + 4751259 = 9143627 N

2) Weight of tank :-

i) Weight of top dome + cylindrical walls = 58474 x π x14 =2571821 N

ii) Weight of tapering dome = Ws = 1066131 N

iii) Weight of lowest dome = 540982 N

iv) Weight of lowest ring beam = 18000 x π x 10 = 565487 N

Entire weight of tank = i + ii + iii + iv

Entire weight of tank = 4744421 N

Total Superimposed load = weight of water + Total weight of tank

Total Superimposed load = 9143627 +4744421

Total Superimposed load = 13888048 N

Load Per column = 13888048 / 8 =**1736000 N**

Supposing the column be 700.00 mm diameter

Weight of column per metre height = $\frac{\pi}{4} \times 0.7^2 \times 1 \times 25000 = 9620$ N

Supposing the bracing be of 300 mm x 600 mm size

Length of Each Brace = L = $R \times \frac{\sin\frac{2\pi}{n}}{\cos\frac{\pi}{n}} = 5 \times \frac{\sin\frac{2\pi}{8}}{\cos\frac{\pi}{8}}$ = **3.83 m**

Clear length of each brace = 3.830 – 0.70 = 3.130 m

Weight of Each brace = 0.3 x 0.6 x 3.13 x 25000 = **14085 N**

-: total Weight of column just above each brace is tabulated below

Brace GH:

W = (134720 + 11760 + 33060) + 4 x 9620 = 1774480.00 N

Brace EF:

W = (134720 + 11760 + 33060) + 8 x 9620 + 14085= 1827045.00 N

Brace CD:

W = (134720 + 11760 + 33060) + 12 x 9620 + 2 x 14085= 1879610.00 N

Bottom of column:

W = (134720 + 11760 + 33060) + 17 x 9620 + 2 x 14085= 1941795.00 N

Wind loads

Intensity of wind pressure = 1500.00 N/m²

Suppose take a factor of 0.7 for section in circular in plan

Wind load on tank, domes and ring beam

= [(5 x 14.4) + (14.2 x 2/3 x1.9) + (2 x 12.8) + (10.6 x 1.21)] x 1500 x 0.7 = 134720 N

This may be assumed to act at about 5.7 m above the bottom of ring beam.

wind load on each panel of 4 m height of column = (4x0.7x8) x1500x0.7 +(0.6x10.6) x1500

wind load on each panel of 4 m height of column = 33060 N

wind load at the top end of top panel = 0.5 x 23520 = 11760 N

wind load are shown in fig below

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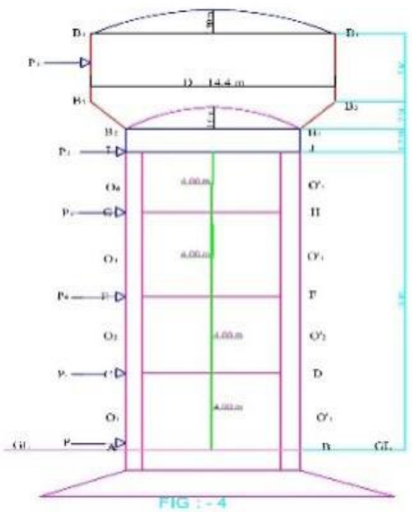

Fig. 4:wind pressure on intz tank

The point of contra flexure O1 O2 O3 and O4 are assumed to be at the mid height of each panel. the shear forces Qw and moment Mw due to wind at these planes are given below

Level	Shear Force Qw (N)	Moment Mw (Nm)
O4	146480.00	1060860.00
O3	179540.00	1712900.00
O2	212600.00	2497180.00
O1	245660.00	3418930.00

Table. 2: Shear force and moment

The Axial thrust Vmax = 4 x Mw / n x Do

The Axial thrust Vmax = 4 x Mw / 8 x 10 = 0.05 Mw

The Axial thrust **Vmax =0.05** Mw in the farthest leeward column the shear force

Smax = 2 x Qw / n = 2 x Qw / 8 =0.25 Qw

Smax = 0.25 Qw

In the column on the bending axis at each of the above levels and the bending moment

M= Smax x h/2 in the column are tabulated below

Table. 3: column forces and bending

Level	Vmax =0.05	Smax = 0.25 Qw	M= Smax x h/2
O4	53040	36620	73240
O3	85650	44895	89770
O2	124860	53150	106300
O1	170950	61420	153550

Table. 4: Axial force and moment

Level	Fartest leeward column		Column on bending axis	
	Axial (N)	Vmax(N)	Axial(N)	M (Nm)
O4O'4	1774480	53040	1774480	73240
O3O'3	1827045	85650	1827045	89770
O2O'2	1879610	124860	1879610	106300
O1O'1	1941795	170950	1941795	153550

The farthest leeward column will be endangered to the superimposed axial load plus Vmax given above The column on the bending axis on the permissible Stress's in the material may be enlarged by 33.33% for the farthest leeward column the axial thrust Vmax due to wind load is less than even 10 % of the superimposed axial load. -: the effect of wind is not critical for the farthest leeward column however, column is situated on the bending axis need to be considered to see the effect of extreme B.M of 153550.00 Nm . due to wind along with the superimposed axial load of 1941795.00 N at the lowest panel .

Use M-20 Concrete For Which

σcbc = 7.00 N/mm² σcc = 5.00 N/mm² [IS 456:2000]

For Steel σst = 230.00 N/mm²

All these three can be increased by 33.33%. When considering action. Diameter of column = 700.00 mm

Use 13 bars of 28 mm Ø at an effective cover of 40 mm

Asc = $\frac{\pi}{4}\ x\ 28^2\ x\ 13$

Asc = 8482 mm²

Equal area of column = $\frac{\pi}{4}\ x\ 700^2\ x\ (13-1)\ x\ 8482$

Equal area of column = 486629 mm²

Equal moment of inertia = $\frac{\pi}{64}\boldsymbol{d^4} + (\boldsymbol{n}-\boldsymbol{1})\frac{\boldsymbol{Asc}}{\boldsymbol{8}}\boldsymbol{xd'^2}$

Where d = 700.00 mm d' =700.00- 40.0 x2.0 =620.00 mm

Ic= $\frac{\pi}{64}700.00^4 + (13.0-1.0)\frac{8482.00}{8.00}x620.00^2$

Ic = 1.6676600000 x 10^{10} mm⁴

Direct Stress's in column = σcc' = 1941795.0 /486629.0 =3.990 N/mm²

Bendings Stress's in column = σcbc' = $\frac{153550\ x\ 1000}{1.66766\ x\ 10^{10}}$ =3.22 N/mm²

For the safetys of column's, we have the condition

$$\frac{\sigma cc'}{\sigma cc} + \frac{\sigma cbc'}{\sigma cbc} \leq 1$$

$$\frac{3.99}{1.33\ x\ 5} + \frac{3.22}{1.33\ x\ 7} \leq 1$$

0.95 < 1 safe

Use 10.00 mm Ø wire rings of 250.00 mm c/c to ties up the mains reinforcements. Since the columns are of 700.00 mm diameters rise the width of curved beam B2 from 600.00 mm to 700.00 mm

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10 DESIGN OF BRACES

The bending moment m1 and extreme value in a brace is governed by step 9

$\tan(\theta + \frac{\pi}{8}) = \frac{1}{2}\cot\theta$

We get $\theta = 24.80^o$

$m1 = \frac{Qw1\ x\ h1+Qw2\ \ h2}{nsin\frac{2\pi}{8}}cos^2\theta\sin(\theta + \frac{\pi}{n})$

For the lowest junction C h1=5.00 m and h2 = 4.00

$m1= \frac{245660\ x\ 5+212600\ x\ 4}{8\ x\ sin\frac{2\pi}{8}}\ cos^2 24.8^o\sin(24.8^o + \frac{\pi}{8})$

m1max = 222540.00 Nm

The max. shear force Sbmax in a brace is given by for = $\frac{\pi}{8}$

$Sbmax = \frac{245660\ x\ 5+212600\ x\ 4}{3.93\ x\ 8\ x\ sin\frac{2\pi}{8}}\ 2cos^2\frac{\pi}{8}\sin\frac{2\pi}{8}$

Sbmax = 112870.00 N

for = $\frac{\pi}{8}$ the value of m1 is given by

$m1max = \frac{245660\ x\ 5+212600\ x\ 4}{8\ x\ sin\frac{2\pi}{8}}\ cos^2\frac{\pi}{8}\sin(\frac{\pi}{8} + \frac{\pi}{8})$

m1max = 221786.00 Nm

Twisting's moments at $\theta = \frac{\pi}{8}$ is M' = 0.05 m1

M' = 0.05 x 221786 =11090.00 Nm

Thus, the bracing will be exposed to a combination of max. shear forces and a twisting moments when wind blow parallel to it = $\frac{\pi}{8}$

Use M-20 Concrete For Which

σcbc = 7.00 N/mm² σcc = 5.00 N/mm²

For Steel

σst = 230.00 N/mm²

k = 0.283 j =0.906 and R = 0.897

Depth of NA = 0.283 d = kd

Supposing Asc =Ast = pbd and dc=0.1d

Equating the moment of equal area about NA P=0.00560

Since the brace is endangered to both BM as well as twisting moment we have

Me1 = M' + MT

Where M' =B.M =22250.00Nm

$MT = T\ (\frac{1+\frac{D}{b}}{1.7})$ =MT = 11090.00 x $(\frac{1+\frac{700}{300}}{1.7})$

MT = 21745.00 Nm

Me1 = 222540.00 +21745.00 = 244285.00 Nm

In order to find the depth of the section, compare the moment of resisting of the section to the external moment

b x n c/2 [d.n/3] + (m-1) Asc.C' (d-dc) = Me1

C = 1.330 x 7.00 = 9.310 N/mm²

mc = 1.50

m =1.50 x 13.00 = 19.5

C' = 9.310 (0.230-0.10)/0.2830 = 6.020 N/mm²

-:300.00 x 0.283 *d x 9.310/2.0 x [1.0-0.2830/3] d + (19.50-1.0) (0.00560 x 300.0*d)*6.02(1.0-0.10) *d=244285.00 x 10^3

d = 680.00 mm

Approve D =700.00 mm so that d =700 -25 -10 =665.00 mm

Asc=Ast=pbd =0.0056 x 300.0 x 700.0 =1176.00 mm²

No. of 20 mm Ø bars each at top and bottom

100 xAs /bd = $\frac{100\ x\ 4\ x\ 491}{300\ x\ 700}$ = 0.94%

Mximum Shear = 112870.00 N

$Ve = V + \frac{1.6\ T}{b}$

$Ve = 112870.00 + \frac{1.6\ x\ 11090}{0.3}$

Ve = 172017.00 N

τve = 172017.00 /300.00 x 700.00

τve= 0.820 N/mm²

This is smaller than τcmax but more than τc =0.37 N/mm² -: transverses reinforcements is necessary

$Asv = \frac{T.Sv}{b1xd1x\sigma sv} + \frac{V.Sv}{2.5xd1x\sigma sv}$

b1 = 230.00 mm; d1 = 630.00 mm

Using 12.00 mm Ø 2 legged stirrups, Asv =226.00 mm²

$Asv = \frac{11090\ x\ 100}{230\ x\ 630\ x\ 230} + \frac{112870}{2.5x\ 360\ x\ 230}$

Asv / Sv =0.645

Minimum transverse reinforcement is given by

$\frac{Asv}{Sv} \geq \frac{\tau ve - \tau c}{\sigma sv}$ x b

$\frac{Asv}{Sv} = \frac{0.82 - 0.37}{230}$ x 300

$\frac{Asv}{Sv} = 00.5870$

Sv = 350.00 mm

However, the space should not exceed the least of X1 , $\frac{X1+Y1}{d}$ and 300 mm

Where

X1 = Short dim stirrip= 230.0+20.0+12.0 =262.0 mm

Y1= Long dime stirrups = 630.0 + 20.0 +12.0 = 662.0 mm

$\frac{262+662}{4}$ = 391.00 mm

-: offer 12.0 mm Ø 2 legged stirrups at 230.0 mm c/c throughout. Since depth of section exceeds 450 mm offer side reinforcement @ 0.1 %

A1 = 0.1/100 x 300 x 700 = 210 mm²

Offer 2 -10 mm Ø bar at each-face giving total

AL = 4 x 78.5 = 314 mm²

Offer 300 mm x 300 mm haunches at the junction of braces with column and reinforce it with 10 mm Ø bar sizes of various components and geometry

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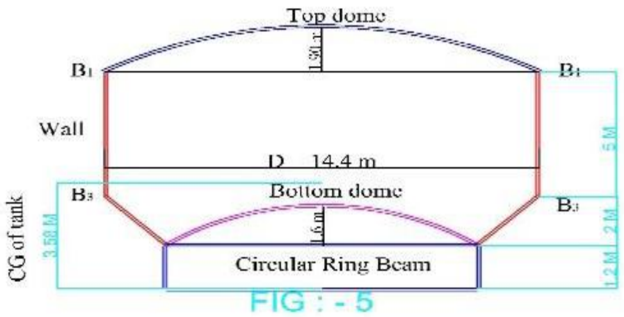


Fig. 5: Component Name

Sizes of various Components are

Top Dome 100 thick

Top Ring Beam B1 400 x 360

Cylindrical Wall 200 thick; Bottom Ring Beam B3 700 x 600

Circular Ring Beam B2 600x1200; Bottom Dome 250to 280 thick

Conical Dome 250 thick: Braces 300 x 700

Columns 700 diameter

Constraints of Spring Mass Model

Total weight of water = 9143.627 Kn.

Volume of water = 9143.627 / 9.81 = 932.072 m3

Mass of water, m = 932072.06 kg.

Inner diameter of tank, D = 14.00 m.

For outcome parameters of spring mass model, an comparable circular container of similar volume and diameter equal to diameter of tank at top level of liquid will be measured.

Let h be the height of equal circular cylinder,

π $(D/2)2$ h = 932.072

h = 932.072 / [π x (14 / 2)2] = 6.05 m

For h / D = 6.05 / 14 = 0.43 ,[IS CODE 1893 Part II P.No 10]

m_i / m = 0.48; m_i = 0.48 x 932072 = 447394.56 kg

m_c / m = 0.50, m_c = 0.50 x 932072 = 466036 kg

h_s =18.20 m h_i / h = 0.395 ; h_i = 0.395 x 6.05 =2.38m

h_i^*/ h = 0.90 ; h_i^* =0.9 x 6.05 = 5.445 m

h_c / h = 0.60; h_c = 0.60 x 6.05 = 3.63 m

h_c^*/ h = 0.815; h_c^* = 0.815 x 6.05 = 4.93 m.

About 55% of liquid mass is excited in impulsive's mode while 43% liquid mass contributes in convective's mode. Sum of impulsive's and convective's mass is 913430.560 kg which is about 2% less than the whole mass of liquid.

weight of empty container + one third weight of staging,

m_s = (4744.4210 + 1702.19 / 3) x (1,000 / 9.81) = 541465.47 kg.

Time Period's

Time period of impulsive's mode,

$T_i = 2\pi\sqrt{\frac{m_i+m_s}{K_s}}$ [IS Code 1893 part 2 pn 16 fig 5

$T_i = 2\pi\sqrt{\frac{447394.56+541465.47}{132.32 x 10^5}}$

***Ti* = 1.70 sec**

Time period of convective's mode ,

$T_c = C_c\sqrt{\frac{D}{g}}$ [IS Code 1893 part 2 pn 16 fig 5]

$T_c = 3.20\sqrt{\frac{14}{9.81}}$ h/D =0.43 Cc= 3.20

Tc =3.82 sec

Design Horizontal Seismic Coefficient

Design horizontal seismic coefficient for impulsive mode ,

$(A_h)_i = \frac{Z \; x \; I}{2 \; x \; R}(\frac{S_a}{g})$

Where, Zone = V I = 1.5 R= 2.5

Z = 0.36 (IS 1893(Part 1): Table 2; Zone V)

***Ti* = 1.70 sec,**

Site has Medium soil,

Damping = 5%,

-:, $(S_a/g)_i$ = 0.9 (IS 1893(Part 1): Figure 2)

$(A_h)_i = \frac{0.36 \; x \; 1.5}{2 \; x \; 2.5}$x0.9

(*Ah*)*i* = 0.097

Design horizontal seismic coefficient for convective mode,

$(A_h)_c = \frac{Z \; x \; I}{2 \; x R}(\frac{S_a}{g})$

Where, I = 2.5, R = 2.5

Zone = V Z = 0.36(IS 1893(Part 1): Table 2; Zone V)

Tc =3.82 sec

Damping = 5%

$(S_a/g)_c$ = 0.45 x 1.75 = 0.787

Multiplying factor of 1.75 is used to obtain S_a/g values for 0.5%damping from that for 5%damping.

$(A_h)_c = \frac{0.36 \; x \; 1.5}{2 \; x \; 2.5}$x 0.787

(*Ah*)*c* = 0.0840

Base-Shear

Base-shear at the lowermost of staging, in impulsive mode,

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

V_i= 0.0970 x (447394.56 + 541465.47) x 9.81

V_i= 940.96 kN-m

Similarly, base shear in convective mode,

$V_c = (A_h)_c \; m_c \; g$

V_c = 0.0840 x 466036.00 x 9.81

V_c = 384.03 Kn

Whole base-shear at the lowermost of staging by SRSS

$V = \sqrt{V_c^2 + V_i^2}$

$V = \sqrt{940.96^2 + 384.03^2}$

V = 1016.16 kN.

Displacement of tank manual:-

Total displacement = Hs/500 = 16000/500 = 32 mm

https://dx.doi.org/10.22161/ijaers.5.10.18 **ISSN: 2349-6495(P) | 2456-1908(O)**

Table. 5: Displacement Manual

Node	Displacement (mm)
U5	32
U4	24
U3	16
U2	8
U1	0

III. SOFTWARE DESIGN INTZ WATER TANK

Design of intz water tank by using SAP2000 with fixed base:

The seismic presentation of RCC structures earlier and after the application of flexibility and stiffness-based elements method is to be studied in the present project. In this study we are presenting isolation system as a substitute of conventional technique to get improved performance of elevated water tank through the earthquake. This section offers model geometry evidence, including items such as joint coordinates, joint restraints, and element connectivity.

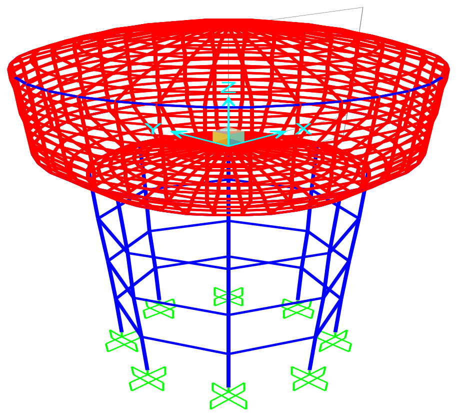


Fig. 6: Finite element model fixed base.

Seismic Data:

Seismic Zone: V; Soil Type: Medium soil

Beam Dead Load (UDL): 1.500 KN/m

Live load = 1.5000 KN/m; Water pressure: 0.60 KN/m

SAP2000 Analysis

1. Analysis of intz tank is to be performed using Sap2000 for Zone-V.

2.after the analysis is done for fixed base intz water tank is compared with the manual result obtained from manual design and sap2000 design.

3.in this study we have found that base shear and displacement result are equal.

4.but we cannot go for the further manual design of base isolation.

5 So software design by using sap2000 we design structure and compare it with fixed base intz water tank.

Total base shear at the bottom of staging by SRSS

V = 1016.36 kN.

Displacement of tank sap2000: -

Table.6: Displacement sap2000 fixed base.

Node	Displacement (mm)
U5	32
U4	23.4
U3	15.5
U2	8
U1	0

Design for intz water tank with base isolation: -

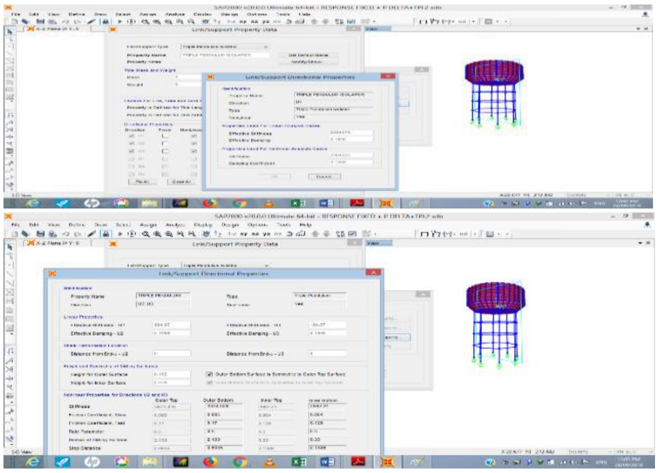


Fig. 7: Finite element model TFPI

SAP2000 Analysis

1. Analysis of intz overhead tank is to be performed using Sap2000 with base isolation for Zone-V.

2.we have used the triple friction pendulum isolator at the support at ground level

3.Response analysis is performed for the intz water tank and design is analyzed.

4. So software design by using sap2000 we design structure and compare it with fixed base intz water tank and base isolation.

Total base shear at the bottom of staging by SRSS

V = 894.69 kN.

Displacement of tank sap2000 with base isolation: -

Table 7: Displacement Base isolation

Node	Displacement (mm)
U5	5
U4	4
U3	2
U2	1
U1	0

Base shear and displacement analysis are performed with manual and SAP both for both fixed and triple friction pendulum support.

Intz water tank is never been considered under research by researchers with triple friction pendulum

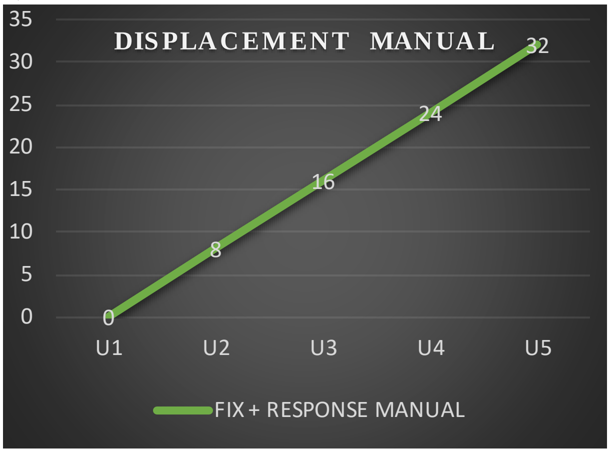
Also, Manual deign is not having option for defining base isolation, still we software defined triple pendulum support in sap2000 to compare with manual fixed base elevated tank.

https://dx.doi.org/10.22161/ijaers.5.10.18 ISSN: 2349-6495(P) | 2456-1908(O)

IV. RESULT PARAMETER

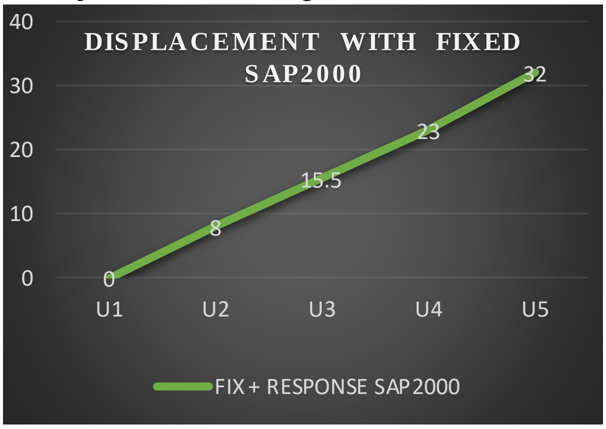
Parameter for manual: - In this manual calculation of intz water tank with earthquake resistant parameter. We have Design the parameter of base shear and displacement are as follows

A) Base shear value = 1016.16 kn



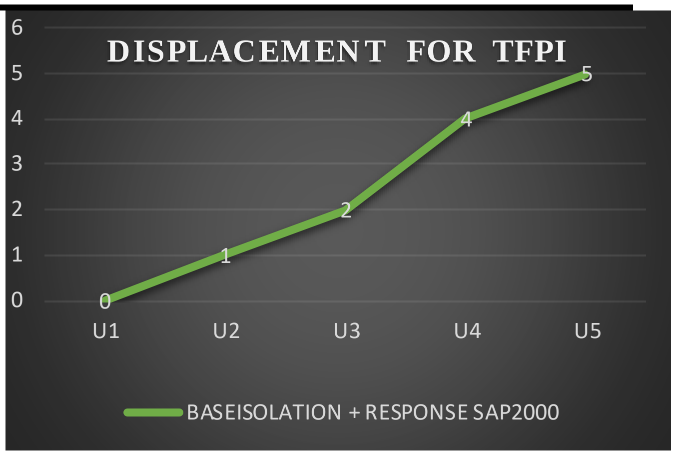
Graph. 1: Displacement

Result parameter for fixed sap2000: - In the sap2000 with fixed base of intz water tank. We have compared the design. we have found Same base shear and displacement as compared to manual design of intz water tank.



Graph. 2: Displacement sap2000

Result parameter for isolated base sap2000: -In the sap2000 with isolated base of intz water tank. We have found that base shear has been reduced to 12.00 % as compared to manual and base isolation. And also, we have found that there is less displacement as compared to manual with fixed base.

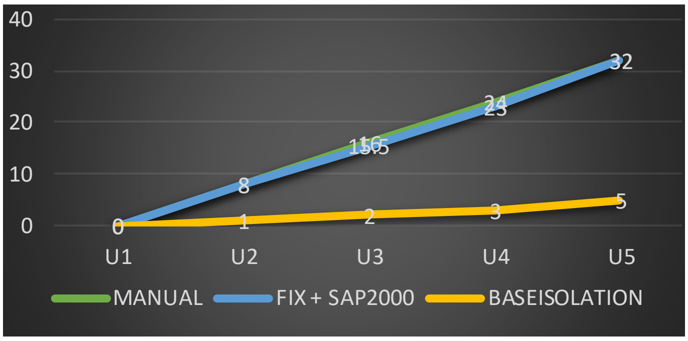


Graph. 3: Displacement base isolation

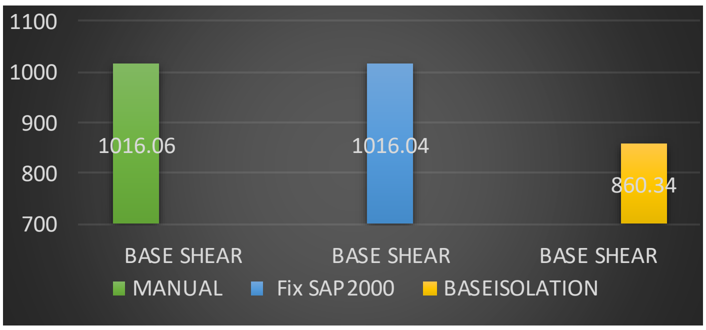
V. CONCLUSION

Elevated Water tank is never been considered under research by researchers with triple pendulum isolator. The result which we have obtained for manual fixed base and sap2000 fixed base we found that design for base shear and displacement are quite same. But for the base isolation elevated water tank we have found 2% decrease in base shear and in the displacement up to 90% is decrease with base isolator

Base shear of Zone V because of zone factor same for manual and fixed response reduction factor etc. while considering seismic analysis. And decrease in base shear for base isolation and displacement.



Graph. 4: Comparison of Displacement



Graph. 5: Comparison of Base Shear

https://dx.doi.org/10.22161/ijaers.5.10.18 ISSN: 2349-6495(P) | 2456-1908(O)

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