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

Dhaval V. Shankhpal¹, Mr. Ankit Pal²

¹M. tech Scholar – Department of Civil Engineering Department, Oriental University, Indore
Email : dhaval.shrine25a@gmail.com
² Assistant Professor Civil Engineering Department, Oriental University, Indore
Email : ankit.5792@gmail.com

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 analyse 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.
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. Base isolation 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, of Conical Dome = 2.00 m

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Suppose Rise h₁=1.800 m ; Rise h₂ = 1.600 m
The Radius R₂ of lowest dome is given by
h₂ * (2 * R₂ - h₂) = R₀
1.6 * (2 * R₂ - 1.6) = 50
Radius of lowest dome R₂=8.610 m
sinØ₂ = 5/8.610 = 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) = R₀
1.600 * (2 * R₁ - 1.600) = 7.00°
Radius of lowest dome R₁=14.510 m
SinØ₁ = 5.00/14.510 = 0.348240
Ø₁ = 28.84°
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
= 4000.00 * 14.510 / 1.00+0.8760 = 30938.00 N/m²
Meridional’s Stress’s per metre = T₁/t x d
= 30938.00 / 100.00 x 1000.00
= 309.38 N/mm²
Meridional’s Stress’s arises at the centre and its
magnitude’s = P*R₁/1+ CosØ₁
= 4000.00 x 14.510/2.00 x 0.10 = 290200.00 N/m²

Extremes hoops Stress’s = 00.29 N/mm²
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
Ø : 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 0.8760 =27102.00 N/m
Whole tension’s tendence’s to ruptures the beams = P₁ x D2
Ac = 27102.00 x 14.00 / 2.00 =189712.00 N
Whole’s tensions tendence’s to rupture’s the beam =
189712.00 N
Permissible Stress’s in HYSD bars = 150.00 mm²[ IS 456
:2000]
Ash = Whole tension tendency to rupture’s the beam
Ash = 189712.00 / 150.00 = 1265.00 mm²
Ash actual = 1265.00 mm²
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 mm²

The areas of cross sections of rings beams is given by

\[ A_c = \frac{\text{Whole tension tending to rupture the beam}}{A + (m-1) \times \text{Ashp}} \]

\[ \text{Ashp} = \frac{1897.12}{A + (13.0-1.0) \times 1257.00} = A = 143014.00 \text{ mm}^2 \]

Offer ring beam of 360.0 mm depth and 400 mm width.
Topmost Ring Beam =400.0x360.0 =144000.0 mm²
Safe

4 DESIGN OF CYLINDRICAL’S WALLS

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 :

\[ P = \frac{w \times h \times D}{2} \]

\[ P = 9800.00 \times 5.00 \times 14.00 / 2.0 \]

\[ P = 34300.00 \text{ N/m} \]

Area of Steel Ash = P / Permissible Stress’s
Ash = 34300.00 / 150.00
\[ \text{Ash} = 2286.00 \text{ mm}^2 \text{ per metre height} \]

Provided that ring’s on both the faces, Ash on each-face = 2286.00/2.0
Ash on each-face = 1143.00 \text{ 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 \text{ mm}^2 on each-face

Permiting 1.200 N/mm² Stress’s on composite section
1.2= \[ \frac{At}{1000.00 \times t + (m-1) \times \text{Ash offerd}} \]
2.0= 343000.00 / 1000.00 x t + (13.0-1.0) x 1190.00 x 2.0
\[ t = 257.330 \text{ mm} \]
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 \text{ mm}^2

Area of steel on each-face = 325.00 \text{ mm}^2
Space of 8 mm Ø bar = 1000.0 x 0.785 x 8² / 325 = 155. mm
\[ \text{ -: offer 8.00 mm Ø bars @ 150.00 mm c/c on both faces} \]

To resist the hoop tension at 2 m below top
Ash =2.00 x 2286.00 /5.00
Ash =914.400 \text{ 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
\[ \text{ -: offer Space of 12.00 mm Ø rings 240.00 mm c/c in the top 2.00 m height’s} \]
At 3.00 m below’s the top
Ash =3.00 x 2286.00 /5.00
Ash =1372.00 \text{ 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
\[ \text{ -: offer Space of 12.00 mm Ø rings 160.00 mm c/c in the next 1.00 m height’s} \]
At 4 m below’s the top
Ash =4.00 x 2286.00 /5.00
Ash =1829.00 \text{ 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
\[ \text{ -: offer Space of 12.00 mm Ø rings 120.00 mm c/c in the next 1.00 m height’s} \]

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 offred to take up this hoop’s tension’s refer fig 2 the load W transmitted through tank wall at the top of conical dome consist of the following

\[ \text{Fig. 2: Load Transmitted} \]

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= 45.00°
\[ \sin \theta = \cos \theta = 0.70710 \quad \tan \theta = 1.00 \]

\[ P_{w} = W \times \tan \theta = 58474.00 \times 1.00 \]

\[ P_{w} = 58474.00 \text{ N/m} \]

\[ P_{w1} = w \times h \times d^{3} = 9800.00 \times 5.00 \times 0.600 \]

\[ P_{w1} = 29400.00 \text{ N/m} \]

- Hoops tension's in the rings beams is given by

\[ P_{3} = \left( \frac{P_{w} + P_{w1}}{2} \right) D/2 \]

\[ P_{3} = \left( \frac{58474.00 + 29400.00}{2} \right) \times 14.00 / 2.00 \]

\[ P_{3} = 615118.00 \text{ N} \]

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

\[ A_{sh} = \frac{P_{3}}{\text{permissible Stress}'} \]

\[ A_{sh} = \frac{615118.00}{150.00} = 4100.00 \text{ mm}^{2} \]

- offer 7 rings of 28 Ø bars

\[ A_{Ash} = 0.785 \times 28^{2} \times 7 \]

\[ A_{Ash} = 4310.26 \text{ mm}^{2} \]

Stress's in equal section = \[\frac{P_{3}}{d \times h + (m-1) \times A_{sh}}\]

\[ \text{Stress's in equal section} = \frac{615118.00}{(1000.00 \times 6000.00) + (13.0 - 1.0) \times 4310.260} \]

\[ \text{Stress's in equal section} = 0.940 \text{ N/mm}^{2} < 1.200 \text{ N/mm}^{2} \text{ safe} \]

8.00 mm Ø distribution’s bars offered 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)

\[ W_{w} = \frac{\pi}{4} D^{2} \times h + \frac{\pi}{12} \times h_{0} \left( D^{2} + D_{o}^{2} + D \times D_{o} \right) - \frac{\pi}{3} h \times \frac{1}{2} (3D_{2}h) \]

\[ W_{w} = 4392368.00 \text{ N} \]

Suppose the thickness of conical slab be 400.00 mm

- Total Self Weight \( W_{s} \) is given by

\[ W_{s} = \left[ \pi \times \left( \frac{D + D_{o}}{2} \right) \times t \times t_{0} \right] \times \gamma c \]

\[ W_{s} = 25000.00 \times 3.140 \times (14.00+10.00/2) \times 2.820 \times 0.40 \]

\[ W_{s} = 1066131.00 \text{ N} \]

Weight \( W \) at \( B_{3} = P_{w} \)

\[ W = 58474.00 \text{ N/m} \]

- Perpendicular load \( W_{2} \) per metre run is given by

\[ W_{2} = \frac{\pi D \times W + W_{w} + W_{s}}{\pi D_{o}} \]

\[ W_{2} = \frac{255613.00}{10.00} \]

\[ W_{2} = 255613.00 \text{ N/m} \]

Meridional’s thrusts To in the conical dome

\[ \text{To} = W_{2} \cos \theta_0 \]

\[ \text{To} = 255613.00 \times 1.4140 \]

\[ \text{To} = 361437.00 \text{ N/m} \]

Meridional’s Stress’s = \( \text{To} / b^d \)

Meridional’s Stress’s = 361437.00 / 1000.00 \times 400.00

Meridional’s Stress’s = 0.900 \text{ N/mm}^{2} 

\[ 0.90 \text{ N/mm}^{2} < 1.200 \text{ N/mm}^{2} \text{: safe} \]

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 \( P = (5.00+2-h') \times 9800.00 \text{ N/m}^{2} \)

Hoop’s tensions \( P' \) is given by

\[ P' = \left( \frac{P}{\cos \theta_0 + q \times \tan \theta_0 } \right) D' / 2 \]

\[ P' = \left( (5.0+2-h') \times 9800.00 \times 1.4140 + 1000 \times 1 \{10.0 + 2h'}/2.0 \right) \]

\[ P' = 553075.00 + 37720.00 \times h' - 13859h'^{2} \]

The value of \( P' \) 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 Maxim, \( d \times P' / dh' = 0 \)

\[ 37720.00 - 2.0 \times 13859.00 \times h' = 0 \]

From Which \( h' = 1.3610 \text{ m} \)

Max \( P' = 535075.00 + 37720.00 \times (1.3610 - 13859.00) \times (1.3610)^{2} \)

Max \( P' = 560739.00 \text{ N} \)

C) Design of Walls: -

Meridional’s Stress’s = 0.900 \text{ N/mm}^{2} 

Max Hoop Stress’s = 560739.00 \text{ N}

Whole of Which is to be resisted by steel

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

\[ As = 560739.00 / 150.00 = 3738.00 \text{ mm}^{2} \]

Area of Each-face = 3738.00 / 2.00 =1869.00 \text{ mm}^{2} 

Space of 16.0 mm Ø bars=1000.0 x0.7850x 16.00² /100.0

=107.05

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

Actual Ash = 1000.0 x0.7850 x 16.0² / 100.0 =2010.00 \text{ mm²}

Max. tensile Stress’s in composite section = 1.3850 \text{ N/mm²} 

This tensile Stress’s is more than the permissible, value 1.20 \text{ N/mm²}:- increase the thickness \( 420.0 \text{ mm} \), this will reduce the tensile Stress’s to 1.1980 \text{ N/mm²} - safe

In the meridional direction offer reinforcement @

\[ (0.30 - [420.0-100.0/420.0-100.0] \times 0.10) =0.210\% \]

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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 = 1000.0 x 0.785 x 10/441.0 = 178 mm
- offer 10.00 mm bars @ 175.00 mm c/c on each face

7 DESIGN OF LOWE ST CIRCULAR BEAM B2:

Let the thickness of bottom dome be 250.00 mm

Total Weight W = Ww + Wo

Vertical load on beam per metre run = 255613 + 290093 x 0.81410
Vertical load on beam per metre run = 5292241 / 3.140 x 10.0 x 0.785 = 9800.00

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 = 442070 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 radius of lowest curved beam R = 5.00 m

2θ = 45°; θ = 22.5°

C₁ = 0.0660; C₂ = 0.0300; C₃ = 0.0050 [IS CODE TABLE 20.1]

θₘₐ = 9.50°

MO = SUPPORT'S MOMENT's B.M - VE = C₁ X WR² X 2θ
MO = 0.066 x 442070.0 x 5.0° x 0.7580

MO = 572882 Nm

Extreme + ve B.M at support = Mc = C₂ X WR² X 2θ
Mc = 0.030 x 442070 x 5° x 0.785

Mc = 260401 Nm

Extreme Torsional moment Mm = 3 X WR² X 2θ

Extreme Torsional moment Mm = 0.005 x 442070 x 5° x 0.785

Extreme Torsional moment Mm = 43400 Nm

For M=20 concrete [IS 456:2000]
σcbc = 7.00 N/mm² HYSD bars est = 150.00 N/mm²
We have K=0.3780: j = 0.8740: R=1.1560

- effective depth = \[ \frac{572882 \times 1000}{600 \times 2.156} \]
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
\[ Fo = 868002.00 \text{ N} \]

SF at any point is given by \( F = WR(\theta - \phi) \)

At \( \theta = \phi_m \); \( F = 442070 \times 5 \times (22.5^\circ - 9.5^\circ) \times 3.14 / 180 \)

\[ F = 501512.00 \text{ N} \]

BM at the point of extreme torsional’s moment’s \( \phi = \phi_m = 9.5^\circ \) is given by

\[ M = WR^2 \left( \theta \times \sin \phi + \theta \times \cot \phi \times \cos \phi - 1 \right) \]

\[ M = 442070 \times 5^2 \left( \Pi/8 \times \sin 9.5^\circ + \Pi/8 \times \cot 22.5^\circ \times \cos 9.5^\circ - 1 \right) \]

\[ M = -1421.00 \text{ Nm (sagging)} \]

At the support \( \phi = 0 \)

\[ Mo = WR^2 \left( \theta - \phi \right) = 0 \]

At mid span \( \theta = 22.5^\circ = \frac{\pi}{8} \) radians

\[ M = WR^2 \left( \theta \cos \phi - \phi \frac{\cos \theta}{\sin \phi} \sin \phi \right) = 0 \]

At the support \( Mo = WT \)

\[ Mo = 572882 \text{ Nm} \]

At the mid span

\[ Mc = 260401 \text{ Nm} \] sagging +ve

At the point of max torsion \( (\phi = \phi_m = 9.5^\circ) \)

\[ M = 1421 \text{ Nm} \]

\[ M_h = 43400 \text{ Nm} \]

**Foremost and Longitudinals Reinforcements**

a) **Section at point of extreme torsion**

\[ T = M_{\phi = \max} = 43400 \text{ Nm} \]

\[ M_\theta = M = 1421 \text{ Nm} \]

\[ Me_1 = 78009 \times 100 / 150 \times 0.874 \times 1160 \]

\[ Ast_1 = 1325.000 \text{ mm}^2 \]

\[ Ve = 501512.00 + 1.60 \times \frac{43400.00}{0.60} \]

\[ Ve = 617245.00 \text{ N} \]

\[ \tau_v = \frac{Ve}{b \times d} \]

\[ \tau_v = 617245.00 / 0.60 \times (1200.00 - 400.00) \]

\[ \tau_v = 887.00 \text{ N/mm}^2 \]

This is less than \( \tau_{\max} = 1800.00 \text{ N/mm}^2 \)

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

\[ \tau_c = 0.230 \text{ N/mm}^2 \]

**Since \( \tau_v > \tau_c \), Shear reinforcement is necessary**

The area of cross section \( A_{sv} \) of the stirrups is given by

\[ A_{sv} = \frac{\pi}{4} \times d \times \rho_{sv} + \frac{V}{b \times d} \times \frac{2}{\pi} \times d \times \rho_{sv} \]

Where \( b_1 = 600 - (40 \times 2) - 25 = 495.00 \text{ mm} \)

\[ A_{sv} = 43400 \times 452 / 442070 + 501512 / 1095 = 1.755 \]

Minimum Transverse’s reinforcements is governed by

\[ A_{sv} \geq \frac{\tau_v - \tau_c}{\sigma_{sv}} \times b \]

\[ A_{sv} \geq \frac{0.887 - 0.23}{150} \times 600 = 2.628 \]

:- Depth \( A_{sv} / S_v = 2.628 \)

Using \( s = 12.0 \text{ mm} \) \( \phi = 4 \) legged stirrups, \( A_{sv} = 4 \times \frac{\pi}{4} \times 12^2 = 452.00 \text{ mm}^2 \)

\[ Or \text{ Sv} = \frac{452}{2.628} = 172.00 \text{ mm} \]

But, the space should not exceed the least of \( X_1, X_1 + Y_1 \)

and 300 mm


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

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

At support , Fo = 868002 N
\[ \tau v = \frac{868002}{600 \times 1160} = 1.25 \text{ N/mm}^2 \]

At Support ,
\[ \frac{100 \times Ax}{bd} = \frac{100 \times 8 \times 491}{600 \times 1160} = 0.564 \]

FC = 0.0310 N/mm² , Shear Reinforcement is necessary

VC = 0.0310 x 600.00 x 1 1160.00 = 215760.00 N

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
\[ Sv = \left( \frac{314 x Ax x d}{Vs} \right) \]
\[ Sv = \frac{150 x 314 x 1160}{652242} = 83.80 \text{ mm} \]

- offer 12 mm Ø 4 legged stirrups

Asv = 4 x 0.7 x 12² = 452.39 mm² at space
\[ Sv = \frac{150 x 452.39 x 1160}{652242} = 120.0 \text{ mm} \]

C) At Mid span

At the mid span, SF is Zero -: offer minimum shear reinforcement given by
\[ \frac{Ax}{Bsv} \geq \frac{0.4}{fy} \]
\[ \frac{Ax}{Asv} = \frac{0.4 x b}{fy} \quad \text{for HYSD bar fy =} 415 \text{ N/mm}² \]
\[ \frac{Ax}{Asv} = \frac{0.4 x 600}{415} = 0.578 \]

Choosing 10 mm Ø 4 legged stirrups Asv =314 mm²
\[ Sv = \frac{314}{0.578} = 543 \text{ mm} \]

Max. permissible 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 \[ \frac{0.1}{100} \times 600 \times 1200 = 720 \text{ mm}² \]

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 x 14 = 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 = \[ \frac{\pi}{4} x 0.7² x 1 x 25000 = 9620 \text{ N} \]

Supposing the bracing be of 300 mm x 600 mm size

Length of Each Brace = L = \[ R x \sin \frac{\pi}{8} \]
\[ = 5 x \sin \frac{\pi}{8} \]
\[ = 3.83 \text{ m} \]

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 x 1.9) + (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 = \((4 x0.7x8) x 15000.07 + (0.6 x 10.6) x 1500 \]

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
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     |

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

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

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

| 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         |

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 = \( \pi \times \frac{28}{4} \times 13 \)

Asc = 8482 mm²

Equal area of column = \( \pi \times \frac{700}{4} \times (13 - 1) \times 8482 \)

Equal area of column = 486629 mm²

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

Where d = 700.00 mm d' = 700.00 - 40.0 x 2.0 = 620.00 mm

Ic = \( \frac{\pi}{64} x 700.00^4 + (13.0 - 1.0) \frac{8482.00}{8.00} \times 620.00^2 \)

Ic = 1.6676600000 x 10⁻¹⁰ mm⁴

Direct Stress’s in column = \( \sigma_{cc'} = 1941795.0 / 486629.0 = 3.990 \) N/mm²

Bendings Stress’s in column = \( \sigma_{cbc'} = \frac{153550 \times 1000}{1.66766 \times 10}\frac{10}{10} = 3.22 \) N/mm²

For the safety of column’s, we have the condition

\[ \frac{\sigma_{cc'}}{\sigma_{cc}} + \frac{\sigma_{cbc'}}{\sigma_{cbc}} \leq 1 \]

3.99 + 3.22 \( \frac{1.33 \times 1}{1.33 \times 7} \) \leq 1

0.95 < 1

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
10 DESIGN OF BRACES

The bending moment $m_1$ and extreme value in a brace is governed by step 9

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

We get $\theta = 24.8^\circ$

$$m_1 = \frac{q_{w1} \times b + q_{w2} \times c}{8 \times \sin \frac{\pi}{8}} \cos^2 \theta \sin \left( \theta + \frac{\pi}{8} \right)$$

For the lowest junction $C: h_1=5.00 \text{ m and } h_2 = 4.00$

$$m_1 = \frac{245660 \times 5 + 212600 \times 4}{8 \times \sin \frac{\pi}{8}} \cos^2 \theta \sin \left( \theta + \frac{\pi}{8} \right)$$

$m_{1 \text{ max}} = 222540.00 \text{ Nm}$

The max. shear force $S_{\text{max}}$ in a brace is given by $\sigma = \frac{\pi}{8}$

$$S_{\text{max}} = \frac{245660 \times 5 + 212600 \times 4}{8 \times \sin \frac{\pi}{8}} \cos^2 \frac{\pi}{8} \sin \left( \frac{\pi}{8} + \frac{\pi}{8} \right)$$

$m_{\text{max}} = 221786.00 \text{ Nm}$

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

Thus, the bracing will be exposed to a combination of $m_1\text{ max}$ shear forces and a twisting moments when wind blow parallel to it $= \frac{\pi}{8}$

Use M-20 Concrete For Which

$\sigma_{\text{c}} = 7.00 \text{ N/mm}^2 \quad \sigma_{\text{cc}} = 5.00 \text{ N/mm}^2$

For Steel

$\sigma_{\text{st}} = 230.00 \text{ N/mm}^2$

$\sigma_{\text{cbc}} = 7.00 \text{ N/mm}^2 \quad \sigma_{\text{cc}} = 5.00 \text{ N/mm}^2$

Use $M$ to find the depth of the section, compare the moment of resisting of the section to the external moment $b \times n \times c/2 [d.n/3] + (m-1) \cdot \text{Asc.C} \cdot (d-dc) = m_1$

$C = 1.330 \times 7.00 = 9.310 \text{ N/mm}^2$

$\text{mc} = 1.50$

$m = 1.50 \times 13.00 = 19.5$

$C' = 9.310 \times (0.230-0.10)/0.2830 = 6.020 \text{ N/mm}^2$

$m_{\text{max}} = 242850.00 \times 10^3$

$d = 680.00 \text{ mm}$

Approve $D = 700.00 \text{ mm so that } d = 700 - 25 - 10 = 665.00 \text{ mm}$

$\text{Asc} = \text{Ast} - \text{pbd} = 0.0056 \times (300.0 \times 700.0) = 1176.00 \text{ mm}^2$

No. of $20 \text{ mm} \phi$ bars each at top and bottom

$100 \times \text{As} / \text{bd} = \frac{100 \times 4 \times 491}{300 \times 700} = 0.94\%$

Maximum Shear $= 112870.00 \text{ N}$

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

$V_e = 112870.00 + \frac{1.6 \times 11090}{0.3} = 17201.00 \text{ N}$

$t_{\text{ve}} = 17201.00 / 300.00 \times 700.00$

$t_{\text{ve}} = 0.820 \text{ N/mm}^2$

This is smaller than $t_{\text{max}}$ but more than $t_{\text{c}} = 0.37 \text{ N/mm}^2$ - transpose reinforcements is necessary

$\text{Asc} = \text{Ast} = \text{pbd} = 0.0056 \times 300.0 \times 700.0 = 1176.00 \text{ mm}^2$

$\text{Asv} = \frac{12.0 \times 2 \times 300}{230.00 \times 700.0} = 1.360 \text{ %}$

$\text{Asv} = \frac{0.645}{0.645} \times 300.0 = 210 \text{ mm}^2$

$\text{Asv} = \frac{12.00 \times 2 \times 300}{230.00 \times 700.0} = 1.360 \text{ %}$

$\text{Asv} = \frac{0.645}{0.645} \times 300.0 = 210 \text{ mm}^2$

$\text{Asv} = \frac{12.00 \times 2 \times 300}{230.00 \times 700.0} = 1.360 \text{ %}$

$\text{Asv} = \frac{0.645}{0.645} \times 300.0 = 210 \text{ mm}^2$

Minimum transverse reinforcement is given by

$\frac{A_{\text{sv}}}{S_v} = \frac{v_e - v_c}{\sigma_{\text{sv}}}$

$\frac{A_{\text{sv}}}{S_v} = \frac{0.820 - 0.37}{230.0} \times 300.00$

$A_{\text{sv}} = 0.5870 \times 230.0 = 138.170 \text{ mm}^2$

$S_v = 350.00 \text{ mm}$

However, the space should not exceed the least of $X_1 \cdot \frac{1}{X_1} + \frac{1}{X_1} \text{ and } 300 \text{ mm}$

$X_1 = \text{Short dim stirrup}= 230.0 + 20.0 + 12.0 = 262.0 \text{ mm}$

$\text{Y}_1 = \text{Long dim stirrup} = 630.0 + 20.0 + 12.0 = 662.0 \text{ mm}$

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

- 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 %

$A_1 = 0.1/100 \times 300 \times 700 = 210 \text{ mm}^2$

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

$A_1 = 4 \times 78.5 = 314 \text{ mm}^2$

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
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: 250 to 280 thick
- Conical Dome: 250 thick; Braces: 300 x 700
- Circular Ring Beam: 700 diameter

Constraints of Spring Mass Model

- Total weight of water = 9143.627 KN.
- Inner diameter of tank = 14.00 m.
- For outcome parameters of spring mass model, the inner diameter equal to diameter of tank at top level of liquid will be measured.
- \( h = \frac{\pi}{2} \times \frac{D}{2} \times \frac{h}{D} = 932.072 \) m
- \( m_i / m = 0.48; m_i = 0.48 \times 932072 = 447394.56 \) kg
- \( mc / m = 0.50; mc = 0.50 \times 932072 = 466036 \) kg
- \( hs = 18.20 \) m
- \( hi / h = 0.395; hi = 0.395 \times 6.05 = 2.38 m \)
- \( hi*/h = 0.90; hi* = 0.9 \times 6.05 = 5.445 \) m
- \( hc / h = 0.60 \) m
- \( hc = 0.60 \times 6.05 = 3.63 \) m
- \( hc*/h = 0.815; hc* = 0.815 \times 6.05 = 4.93 \) m

55% of liquid mass is excited in impulsive mode while 43% liquid mass contributes in convective mode.

\( Ti = 2\pi \sqrt{\frac{m_i + ms}{Ks}} \) [IS Code 1893 part 2 pn 16 fig 5]

**Time period of impulsive’s mode**

\( Ti = 1.70 \) sec

**Time period of convective’s mode**

\( Tc = \frac{7}{R} \sqrt{\frac{f}{g}} \) [IS Code 1893 part 2 pn 16 fig 5]

\( Tc = 3.82 \) sec

Design Horizontal Seismic Coefficient

Design horizontal seismic coefficient for impulsive mode,

\( (Ah)_i = \frac{2 \times 1.81}{2 \times R} \)

\( (Ah)_i = 0.97 \)

Design horizontal seismic coefficient for convective mode,

\( (Ah)_c = \frac{2 \times 1.81}{2 \times R} \)

\( (Ah)_c = 0.084 \)

Base-Shear

**Base-shear at the lowest of stage, in impulsive mode**

\( Vi = (Ah)_i \times (m_i + ms) \) g

\( Vi = 0.0970 \times (447394.56 + 541465.47) \times 9.81 \)

\( Vi = 940.96 \) kN

Similarly, base shear in convective mode,

\( Vc = (Ah)_c \times mc \) g

\( Vc = 0.0840 \times 466036.00 \times 9.81 \)

\( Vc = 384.03 \) kN

**Whole base-shear at the lowest of stage by SRSS**

\( V = \sqrt{Vc^2 + Vi^2} \)

\( V = \sqrt{940.96^2 + 384.03^2} \)

\( V = 1016.16 \) kN.

Displacement of tank manual: -

Total displacement = \( Hs / 500 = 16000 / 500 = 32 \) mm
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.

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

Table 5: Displacement Manual

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

Design for intz water tank with base isolation: -

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 \( \approx 894.69 \) kN.

Displacement of tank sap2000 with 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 design 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.
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

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.

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.

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.
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