DESIGN OF ELEVATED SERVICE RESERVOIR

1.0 INTRODUCTION:

For storage of large quantities of liquids like water, oil, petroleum, acid and sometime gases also, containers or tanks are required. These structures are made of masonry, steel, reinforced concrete and pre stressed concrete.

Out of these, masonry and steel tanks are used for smaller capacities. The cost of steel tanks is high and hence they are rarely used for water storages. Reinforced concrete tanks are very popular because, besides the construction and design being simple, they are cheap, monolithic in nature and can be made leak proof.

Generally no cracks are allowed to take place in any part of the structure of Liquid Retaining R-C.C. tanks and they are made water tight by using richer mix (not less than M 30) of concrete. In addition some times water proofing materials also are used to make tanks water tight.
1.1) **CLASSIFICATION OF R.C.C. TANKS:**

In general they are classified in three categories depending on the situation.

1. Tanks resting on ground.
2. Tanks above ground level (Elevated tanks).
3. Under ground tanks.

1.2. **TANKS RESTING ON GROUND:**

These are used for clear water reservoirs, settling tanks, aeration tanks etc. these tanks directly rest on the ground. The wall of these tanks are subjected to water pressure from inside and the base is subjected to weight of water from inside and soil reaction from underneath the base. The tank may be open at top or roofed.

Ground water tank is made of lined carbon steel, it may receive water from a water well or from surface water allowing a large volume of water to be placed in inventory and used during peak demand cycles.
1.3. **ELEVATED TANKS:** These tanks are supported on staging which may consist of masonry walls, R.C.C tower or R.C.C. column braced together. The walls are subjected to water pressure from inside. The base is subjected to weight of water, wt- of walls and wt. roof. The staging has to carry load of entire tank with water and is also subjected to wind loads.

Water tank parameters include the general design of the tank, choice of materials of construction, as well as the following.

1. Location of the water tank (indoors, outdoors, above ground or underground) determines color and construction characteristics.
2. Volume of water tank will need to hold to meet design requirements.

3. Purpose for which the water will be used, human consumption or industrial determines concerns for materials that do not have side effects for humans.

4. Temperature of area where water will be stored, may create concern for freezing and delivery of off setting heat.

5. Delivery pressure requirements, domestic pressures range from 35-60 PSI, the demand for a given GPM (gallons per minute) of delivered flow requirements.

6. How is the water to be delivered to the point of use, into and out of the water tank i.e. pumps, gravity or reservoir.

7. Wind and Earthquake design considerations allow a design of water tank parameters to survive seismic and high wind events.

8. Back flow prevention, are check valve mechanisms to allow single direction of water flow.

9. Chemical injection systems for algae, bacteria and virus control to allow long term storage of water.

10. Algae in water tanks can be mitigated by removing sunlight from access to the water being stored.
1.4. **UNDER GROUND TANKS**: These tanks are built below the ground level such as clarifiers filters in water treatment plants, and septic tanks. The walls of these tanks are subjected to water pressure from inside and earth pressure from outside. The base of the tanks is subjected to water pressure from inside and soil reaction from underneath. Always these are covered at top. These tanks should be designed for loading which gives the worst effect.
The design principles of underground tanks are same as for tanks resting on the ground. The walls of the underground tanks are subjected to internal water pressure and outside earth pressure. The section of wall is designed for water pressure and earth pressure acting separately as well as acting simultaneously. Whenever there is possibility of water table to rise, soil becomes saturated and earth.

**TYPE OF TANKS:**

From the design consideration storage tanks are further classified according to their shape and design principles as

(1) circular tanks.

(2) Rectangular tanks.

(3) Intze type tanks.

4) Spherical tanks.

5) conical bottom tanks.

6) PSC Tanks.

**1.6 CIRCULAR TANKS:** Generally circular tanks rest on the ground or are elevated ones. Under ground circular tanks are also constructed. The circular tanks may be designed either with flexible base connection with wall or with a rigid connection between walls and base. In the former case the expansion and
contraction of side walls are possible but in latter case the walls are monolithic with base. The walls of tanks are subjected to hydrostatic pressure which is maximum at base and zero at top. Usually for design of circular tanks, the theory of thin cylinders is applied for design of wall thickness and for calculation of maximum hoop tension.

![Circular Tank Image](image)

**FIG NO 1.3 : CIRCULAR TANK**

Maximum hoop tension is given by formula $P = \frac{WHD}{2}$
And area of steel required for this tension is given by

\[ A_{st} = \left( \frac{WHD}{2f} \right) \]

for the calculation of thickness of wall, the permissible tensile stress in concrete is equated to expression given below,

\[ c_t = \left( \frac{WHD}{2} \right) - 100t + (m-1) A_{st} \]

In all the above cases
- \( W \) = Wt. of water/Cu.m.
- \( D \) = Diameter of Tank,
- \( H \) = Height of the tank.
- \( A_{st} \) = Area of steel required.
- \( m \) = Modular ratio
- \( t \) = Thickness of wall

\( f \) and \( c_t \) are permissible tensile stresses in steel and concrete respectively. As the pressure is maximum at base and reduced to zero at the top, the reinforcement is also gradually reduced to a minimum requirement from bottom to top. The main reinforcement consists of circular hoops to take care of hoop tension and is placed on both faces of wall. The distribution steel is placed vertically and is tied to main reinforcement.

Though it is assumed that the connection between walls and base is flexible, in reality there is some moment at the joint- Hence it is impossible to get an ideal flexible joint. When the joint is not flexible and restricted up to certain
height from base, the wall acts as a cantilever and beyond that it acts as simply supported.

1.7. RECTANGULAR TANKS: For smaller capacities circular tanks are uneconomical and their form work is costly. Rectangular tanks are constructed when small capacity tanks are required. These may be resting on ground, elevated or under ground. Tanks should be preferably square in plan and it is desirable that larger side should not be greater than twice the smaller side and for Rectangular tanks.

FIG NO 1.4 : RECTANGULAR TANK
Walls of the tanks either resting on ground or elevated are subjected to water pressure from inside and when under ground they are subjected to internal water Pressure and outside earth pressure.

In rectangular tanks moments are caused in two directions, hence exact analysis is rather difficult, they are designed by approximate methods. For tanks in which ratio of length to breadth is less than 2, tank walls are designed as continuous frame subjected to pressure varying from zero at top to maximum at H/4 or 1 meter from the base which ever is more.

The bottom H/4 or 1 m whichever is more is designed as a cantilever. Besides, the walls are also subjected to direct pull or tension due to the hydrostatic force on the other side walls. The section is to be designed for bending and direct tension.

\[ h = \frac{H}{4} \text{ or } 1.0 \text{ m} \]

The direct tension in long walls \[ = W( H-h) L/2 \]

\[ = W( H-h) B/2 \]

where \[ W = \text{Wt. of water/Cum} \]
\[ H = \text{Height of tank in m.} \]
\[ B = \text{Breadth of the tank in m.} \]
For rectangular walls in which ratio of length to breadth is greater than 2 the long walls are designed as cantilevers for maximum moment of \( Wh^3 / 6 \) and short walls as slabs supported on long walls. The bottom height \( H/4 \) or 1 m. of short wall, whichever is more is designed as cantilever.

In this also the direct tension caused due to pressure on the other walls should be taken into account and the reinforcement is to be provided for.

When tanks are open at top, the walls of the tanks can also be designed as (a) All the walls spanning horizontally as slabs (b) All the walls as cantilevers.

1.8 **Intze tanks**: This is a special type of elevated tank used for very large capacities. Circular tanks for very large capacities prove to be uneconomical when flat bottom slab is provide.

   Intze type tank consist of top dome supported on a ring beam which rests on a cylindrical wall. The walls are supported on ring beam and conical slab. Bottom dome will also be provided which is also supported by ring beam.

The conical and bottom dome are made in such a manner that the horizontal thrust from conical base is balanced by that from the bottom dome. The conical and bottom domes are supported on a circular beam which is in turn, supported on a number of columns. For large capacities the tank is divided into two compartments by means of partition walls supported on a circular beam.
Following are the components

(1) Top dome.

(2) Ring Beam supporting the top dome.

(3) Cylindrical wall.

(4) Ring beam at the junction of the cylindrical wall and the conical shell.

(5) Conical shell.

(6) Bottom dome.

(7) The ring girder.

(8) Columns braces.

(9) Foundations.

1.9. PRESTRESSED TANKS: The pre-stressed water tanks are built to hold liquids in large quantities. In circular tanks circumferential pre-stress is provided to resist hoop tension produced by internal liquid pressure.

Pre-load Corporation of America has developed a system by which continuous pre-stressing can be done. It consists of a machine called marry-go-round which is
supported by a trolley that moves at the top of the tank. The marry-go-round releases wire from a drum, tensions it through a die and wraps it round the tank walls. The wire is anchored at the bottom of the tank and the wrapping is done after pre-stressing the wall, the tank is filled and steel is covered by guniting.

The concrete is fully hardened in tank before the tensioned wire is wrapped around to cause hoop compression. The tank is covered with a dome of small rise. The ring beam is provided to support the dome. The dome may be also pre-stressed.
2.0 MATERIALS USED AND THEIR DESIGN REQUIREMENTS

Following are the materials which are used in the construction of R.C.C. Water Tanks.

i) Concrete.

(ii) Steel.

iii) Water Proofing materials.

iv) Minimum Reinforcement.

2.1 CONCRETE: Design of liquid retaining structure is different from an ordinary R.C.C. Structure as it is required that the concrete should not crack and it should be of high quality and strength and should be leak proof.

The design of the concrete mix shall be such that the resultant concrete is sufficiently impervious. Efficient compaction preferably by vibration is essential.

The permeability of the thoroughly compacted concrete is dependent on water cement ratio. Increase in water cement ratio increases permeability, while concrete with low water cement ratio is difficult to compact.
Other causes of leakage in concrete are defects such as segregation and honey combing. All joints should be made water-tight as these are potential sources of leakage.

Use of small size bars placed properly, leads to closer cracks but of smaller width. The risk of cracking due to temperature and shrinkage effects may be minimized by limiting the changes in moisture content and temperature to which the structure as a whole is subjected.

The risk of cracking can also be minimized by reducing the restraint on the free expansion of the structure with long walls or slab founded at or below ground level, restraint can be minimized by the provision of a sliding layer. This can be provided by founding the structure on a flat layer of concrete with interposition of some material to break the bond and facilitate movement.

Generally concrete mix weaker than M-30 is not used. To get high quality and impervious concrete, the proportion of fine and coarse aggregate to cement is determined carefully and water cement ratio is adjusted accordingly. Depending up on the exposure conditions, the grade of concrete is decided.
2.2 **STEEL:** Steel used reinforcement should confirm to IS: 1786: 1985

Since steel and concrete are assumed to act together; it has to be checked whether the tensile stress in concrete is within limits, so as to avoid cracks in concrete. The tensile stress in steel will be limited by the requirement that the permissible tensile stresses in concrete is not exceeded.

The permissible stresses in steel reinforcement is as follows for calculation of strength.

(a) Permissible tensile stresses in member in direct tension

\[ = 1500 \text{ Kg/Cm}^2 \]

(b) Tensile stress in member in bending

on liquid retaining face of member \[ = 1500 \text{ Kg/Cm}^2 \]

On faces away from liquid for members less than 225 mm thick

\[ = 1500 \text{ Kg/Cm}^2 \]

(c) On faces away from liquid for members 225 mm. thick or more

\[ = 1900 \text{ Kg/Cm}^2 \]
2.3. **MINIMUM REINFORCEMENT:** The minimum reinforcement in each of two directions shall have an area of 0.24% of Cross-Sectional area of concrete up to 100 mm thick.

For section of thickness greater than 100 mm and less than 450 mm. The reinforcement in each direction in shall be linearly reducing from 0.24% cross-sectional area to 0.16% cross-sectional area. For section greater than 450 mm thick reinforcement in each direction should be kept at 0.16% cross sectional area.
2.4. WATER PROOFING MATERIALS: Primary considerations in water tanks, besides, strength, is water tightness of tank. Complete water-tightness can be obtained by using a high strength concrete. In addition water proofing materials can be used to further enhance the water tightness.

To make concrete leak proof or water tight, internal water proofing or water proof linings are frequently used. In the method of internal water proofing, admixtures are used. The object of using them is to fill the pores of the concrete and to obtain a dense and less permeable concrete. Some of the most commonly used admixtures are hydrated lime in quantity varying from 8 to 15 percent, by weight of cement of, powdered iron fillings, which expands upon oxidation and fills the pores of concrete. Other agents like powdered chalk or talc, Sodium silicate Zinc sulphate, Calcium chloride etc., are also most extensively used.

In waterproof linings, paints, asphalts, coaltar, waxes, resins and bitumens are used. These materials have a property to repel the water.
3.0 INTRODUCTION:

3.1 General

The tank is proposed of Circular type with Slabs to form the base and another Slab forming the roofing. Floor beams along the periphery of the Circular slab is proposed to transfer the loads to the Supporting Structure.

The following Primary Loads considered in the Design of the tank portion
a) Dead Load
b) Live Load acting on Roof slab
c) Water Load inside the tank up to top of the Rectangular wall including free board.
d) Combination of all the above loads.

3.2 DESIGN BASIS
(a) The Circular wall has been designed for Hoop Tension & Bending moment
(b) The Floor beam has been designed for bending between the supporting columns.
(c) The columns are designed for direct load and bending due to wind/seismic effect.

3.3 Material Specifications
a) Grade of Concrete M30
b) Grade of steel - High yield deformed bars with yield stress of 415 N/mm²

3.3.1) Strength parameters
a) Concrete For Container Portion = M 30

3.3.2) Permissible stresses
Direct tension stress $\sigma_{ct}$ = 15 kg/cm²
Direct compressive stress $\sigma_{cc}$ = 80 kg/cm²
Bending tensile stress $\sigma_{cht}$ = 20 kg/cm²
Bending comp stress $\sigma_{cbc}$ = 100 kg/cm²
Characteristic comp strength $f_{ck}$ = 30 kg/cm²
Shear = 22 kg/cm²
Average Bond = 10*1.4
Local Bond = 17*1.4

3.3.3) Strength parameters

a) Concrete For Staging Portion
The staging will be designed with M20 concrete and executed with M25.

3.3.4) Permissible stresses

Direct tension stress $\sigma_{ct}$ = 12 kg/cm²
Direct compressive stress $\sigma_{CC}$ = 50 kg/cm²
Bending tensile stress $\sigma_{Cbt}$ = 17 kg/cm²
Bending comp stress $\sigma_{cbc}$ = 70 kg/cm²
Characteristic comp strength $f_{ck}$ = 20 kg/cm²
Shear = 17 kg/cm²
Average Bond = 8*1.4
Local Bond = 13*1.4

(b) Steel HYSY Fe = 415 N/mm²

Permissible stresses
(Water Retaining members)

3.4) Tensile Stress in members under direct tension = 1500 kg/cm²
Tensile stress in members in bending:
   a) On Liquid retaining face of members = 1500 kg/cm²
   b) On face away from liquid for members less than 225 mm = 1500 kg/cm²
   c) On face away from liquid for members 225 mm or more = 1900 kg/cm²
Tensile stress in Shear Reinforcement:
   a) For members less than 225 mm = 1500 kg/cm²
b) For members 225 mm or more
Compressive stress in columns subjected to direct load

\[
\text{Compressive stress} = 1750 \text{ kg/cm}^2
\]

3.5) Min Area of Reinforcement for Walls:
Min Area of Reinforcement for 100 mm thick Wall
Min Area of Reinforcement for 450 mm thick or more

\[
\begin{align*}
\text{Min Area of Reinforcement} &= 0.24 \% \\
\text{Min Area of Reinforcement} &= 0.16 \%
\end{align*}
\]

3.6) For M30 Concrete ; Design Constants :

\[
\begin{align*}
\sigma_{st} &= 1500 \text{ kg/cm}^2 \\
\sigma_{cbc} &= 100 \text{ kg/cm}^2 \\
m &= \frac{280}{3 \times \sigma_{cbc}} = 9.33 \\
r &= \frac{\sigma_{st}}{\sigma_{cbc}} = 15 \\
K &= \frac{m}{m+r} = 0.38 \\
j &= 1 - \left( \frac{K}{3} \right) = 0.87 \\
Q &= 0.5 \times \sigma_{cbc} \times k \times j = 16.53
\end{align*}
\]

\[
\begin{align*}
\sigma_{st} &= 1900 \text{ kg/cm}^2 \\
\sigma_{cbc} &= 100 \text{ kg/cm}^2 \\
m &= \frac{280}{3 \times \sigma_{cbc}} = 9.33 \\
r &= \frac{\sigma_{st}}{\sigma_{cbc}} = 15 \\
K &= \frac{m}{m+r} = 0.33 \\
j &= 1 - \left( \frac{K}{3} \right) = 0.89 \\
Q &= 0.5 \times \sigma_{cbc} \times k \times j = 14.69
\end{align*}
\]

\[
\begin{align*}
\sigma_{st} &= 2300 \text{ kg/cm}^2 \\
\sigma_{cbc} &= 100 \text{ kg/cm}^2 \\
m &= \frac{280}{3 \times \sigma_{cbc}} = 9.33 \\
r &= \frac{\sigma_{st}}{\sigma_{cbc}} = 23 \\
K &= \frac{m}{m+r} = 0.29 \\
j &= 1 - \left( \frac{K}{3} \right) = 0.9 \\
Q &= 0.5 \times \sigma_{cbc} \times k \times j = 13.05
\end{align*}
\]
For Un Cracked Section:

Permissible Bending Tension = 20 kg/cm²

M.R = \( \frac{Qbd^2}{6} \times b \times d^2 \times f \)

\( Q = \frac{f}{6} \) = 3

Considering Reinforcement: \( Q = 3.33 \)

For M20 Concrete:

Design Constants:

\( \sigma_{st} = 1900 \) kg/cm²

\( \sigma_{cbc} = 70 \) kg/cm²

\( m = \frac{280}{3 \times \sigma_{cbc}} = 13.33 \)

\( r = \frac{\sigma_{st}}{\sigma_{cbc}} = 27.14 \)

\( K = \frac{m}{m+r} = 0.33 \)

\( j = 1 - \frac{K}{3} = 0.89 \)

\( Q = 0.5 \times \sigma_{cbc} \times k \times j = 10.28 \)

\( \sigma_{st} = 2300 \) kg/cm²

\( \sigma_{cbc} = 70 \) kg/cm²

\( m = \frac{280}{3 \times \sigma_{cbc}} = 13.33 \)

\( r = \frac{\sigma_{st}}{\sigma_{cbc}} = 32.86 \)

\( K = \frac{m}{m+r} = 0.29 \)

\( j = 1 - \frac{K}{3} = 0.9 \)

\( Q = 0.5 \times \sigma_{cbc} \times k \times j = 9.14 \)

3.7) Design Data:

Capacity = 150 KL

Staging = 12 M

Seismic Zone = III

R.C.C to be M30 for container.

Staging to be Designed with M20 Concrete and executed with M25 Concrete.
3.8) Hydraulic features:

Ground level = 340.725
Lowest water level (LWL) = 352.725
Max water level (MWL) = 355.725
Dead storage = 0.15
Free board = 0.3
Effective Water depth $H = 355.725 - 352.725 = 3.000$

3.9) Member sizes:

No of columns Supporting the ESR = 5
No of Columns inside Container = 1
No of Braces = 3 Levels
Size of Container = 8.00 $\Phi$
Size of Column Supporting ESR = 400 $\Phi$
Size of Column inside Container = 200 $\Phi$
Braces = 250x 350
Roof beam = 200x 300
Floor beam = 300x 600
Floor Ring beam = 300x 300
The of side wall = 200 mm

3.10) Loads

Wind pressure = 150 kg/m²
Live Load on Roof = 100 kg/m²
Live Load on Walkway Slab = 300 kg/m²
Density of concrete = 2500 kg/m³
Density of water = 1000 kg/m³
3.11) **Soil Parameters:**
- Safe bearing capacity of soil (SBC) (Assumed) = 6.5 t/m²
- Depth of foundation = 3 m
- Depth of Ground Water table is at = 7 m
- Seismic Zone = III

3.12) **Capacity Calculations:**
- Depth of Water between MWL & LWL (Live Storage) h = 3 m
- Required Capacity of Tank V1 = \( \frac{\pi}{4} D^2 h \) = 150 m³
- Inner Diameter of Tank Required = 7.979 m
- Inner Diameter of Tank Provided D = 8 m
- Volume of Tank V₁ = 150.8 m³
- Consider freeboard of the Cylindrical Portion (FB) = 0.3 m
- Volume of Freeboard Portion V₂ = 15.08 m³
- Height of Dead Storage Portion = 0.15 m
- Volume of Dead Storage Portion V₃ = 7.54 m³
- Total Volume = 172.62 m³
- Total Height of the Cylindrical Portion (h) = 3.45 m
- Volume of Internal Column = 0.108 m³
- Net Volume of Tank V₁ = 150.692 m³

3.13) **DESIGN OF ROOF SLAB:**
- Let the thickness of slab be = 120 mm
- Width of Panel = 4.1 m
- Effective Span of Slab = 3.63 m
- Density of Concrete = 2500 kg/m³
- Dead Wt of Slab = 0.12 x 2500 = 300 kg/m²
- Floor Finish = 100 kg/m²
- Live Load = 100 kg/m²
- Total = 500 kg/m²

\[ \frac{L_y}{L_x} = 1 \]

Refer Table: 27, IS 456 /2000, CASE - 4 - Two adjacent edges Discontinuous
Negative moment Co efficient at continuous edge = 0.047
Positive moment Co efficient at Mid span = 0.035

Negative B.M
=0.047*500*3.63^2 = 309.66 Kg-m

Positive B.M
=0.035*500*3.63^2 = 230.59575 Kg-m

Effective thickness = 9.1 cm

Area of Steel Required at Support
-ve Ast = (309.66 X100 )/(1900x0.89x9.1 ) = 2.01 cm²

Area of Steel Required at Span
-ve Ast = (230.5975 X100 )/(1500x0.87x9.1 ) = 1.94 cm²

Min. area of steel required ( Astmin ) = (0.12/100)x12x100 = 1.44 cm²

Max. dia. Of bar ( f_max = D / 8) = 15 mm

Min. area of steel required on each face ( Astmin ) = 0.72 cm²

Max. allowble spacing ( S_max ) ar per IS 456 = 27.3 mm

Dia. Of bar ( f ) = 8 mm

Area of Bar = 50 mm²

Required Spacing = 24.88 cm

Provided Spacing = 20 cm

Provided Area = 250 mm²

Provide 8 dia tor @ 200 c/c bothways at bottom & alternate bars bent
Provide 8 dia tor @ 400 c/c bothways at top

Check for Deflection :
L / d = 26
% of Compression Reinforcement P_c = 0

Multiplication Factor for Tension Reinforcement = 2.00

Multiplication Factor for Compression Reinforcement = 1

Modified L/ d Ratio = 52.00
Actual L/d Ratio = 39.89

Hence, SAFE
3.14 DESIGN OF ROOF BEAM :-

Length of Span = 4.1 m
Let Width of Beam = 20 cm
Let Depth of Beam = 30 cm
Clear Cover = 3 cm
Effective Depth = 26.4 cm

Load from Slab = ($\pi / 8$) x $4.1^2$ x 500 = 3300.64 kg
U.D.L = 90 kg/m
S.F = $(3300.64 + 90 \times 4.1) / 2$ = 1834.82 kg/m
- ve B.M = $(5/48) \times 3300.64 \times 4.1 + 90 \times 4.1^2 / 12$ = 1535.72 Kg-m
Net - ve B.M = $1535.72 - 1834.82 \times 0.2 / 3$ = 1413.4 Kg-m
+ ve B.M = $0.6 \times (5/48) \times 3300.64 \times 4.1 + 0.5 \times 90 \times 4.1^2 / 12$ = 908.83 Kg-m
Area of steel required for Support = $1413.4 \times 100 / (1900 \times 0.89 \times 26.4)$ = 3.44 cm²
Min Area of Steel = 0.85 bd / fy = 1.08 cm²

At Top :

<table>
<thead>
<tr>
<th>Dia of Bar</th>
<th>Straight Bars</th>
<th>Extra Bars</th>
</tr>
</thead>
<tbody>
<tr>
<td>Area of Bar</td>
<td>= 12 mm</td>
<td>16 Mm</td>
</tr>
<tr>
<td>Required no of Bars</td>
<td>= 1.13 cm²</td>
<td>2.01 cm²</td>
</tr>
<tr>
<td>Provide no of Bars</td>
<td>= 3.04 no's</td>
<td>0.59 no's</td>
</tr>
<tr>
<td>Provided Area of Steel</td>
<td>= 2.26 cm²</td>
<td>2.01 cm²</td>
</tr>
</tbody>
</table>

Provide 2-12tor through + 1-16 tor extra at Supports (4.27sqcm)
Area of steel required for Span = $908.83 \times 100 / (1500 \times 0.87 \times 26.4)$ = 2.64 cm²
At Bottom :

<table>
<thead>
<tr>
<th></th>
<th>Straight Bars</th>
<th>Extra Bars</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dia of Bar</td>
<td>12 mm</td>
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</tr>
<tr>
<td>Area of Bar</td>
<td>1.13 cm²</td>
<td>2.01 cm²</td>
</tr>
<tr>
<td>Required no of Bars</td>
<td>2.34 no's</td>
<td>0.19 no's</td>
</tr>
<tr>
<td>Provide no of Bars</td>
<td>2 no's</td>
<td>1 no's</td>
</tr>
<tr>
<td>Provided Area of Steel</td>
<td>2.26 cm²</td>
<td>2.01 cm²</td>
</tr>
</tbody>
</table>

through + 1-16 TOR extra at bottom
(4.27 sqcm) Provide 2-12 tor

% of Steel Provided = 100 x 4.27 / (20x26.4) = 0.81

Permissible shear stress in concrete (tc) = 4.092 kg/cm²
Nominal shear stress (tv) = Vu/bd = 1834.82 / (20*26.4) = 3.475 kg/cm²

Net Shear force = 1834.82 - (4.092*20*26.4) = -325.76 kg

Stirrup Dia = 8 mm

No of legs = 2

Area of Bar = 1.01 cm²

Spacing required is Min. of following
Max. 300 mm

0.75*d = 198 mm

Minimum Shear Reinforcement = \( \frac{A_{sv}}{b_{sv}} \geq (0.4/0.87f_y) \)

Sv <= 0.87f_yA_{sv} / 0.4b

Sv <= = 45.58 cm

0.75 d = 19.8 cm

= = 30 cm

Provided Spacing is Lesser of above two cases = 19.80 cm

Say = 18.00 cm

Provide 8 dia tor @ 180 c/c through out
3.15) DESIGN OF SIDE WALL:-

LWL = 352.725 m  
MWL = 355.725 m  
Free board = 0.3  
Dead Storage = 0.15  
Height of Wall = 3.45 m  
Let the thickness of Wall = 200 mm  
Design Depth of Water \( H \) = 3.45 m  
Diameter of Wall \( D \) = 8 m  
\[ \frac{H^2}{Dt} = 7.44 \]

Max Loading at Base = 3450 kg/m^2  
Loading at Top = 0 kg/m^2  
Refer Appendix  
From IS 3370 -PART -IV -1967  
Design for Fixed Condition :

Max Bending moment Coefficient for Water face = -0.0158  
Max Bending moment Coefficient for Outer face = 0.0042  
Max Bending Moment at Water face = 646.75 Kg-m  
Max Bending Moment at Outer face = 171.02 Kg-m  
Uncracked depth required = \( \sqrt{646.75/3} \) = 14.68 cm  
Considering Steel Contribution, thickness required = 13.94 cm  
How ever Provided Overall thickness is 20cm O.K

clear cover = 30 mm  
Effective thickness = 165 mm  
\[ \text{Min. area of steel required ( Astmin )} = \frac{(0.217/100)x20x100}{(1500x0.87x16.5)} = 4.34 \text{ cm}^2 \]

Area of Steel Required on Water face  
\[ \text{-ve Ast} = \frac{(646.75 \times 100)}{(1500 \times 0.87 \times 16.5)} = 3 \text{ cm}^2 \]

Dia. Of bar ( f ) = 10 mm  
Area of Bar = 79 mm^2  
Required Spacing = 26.33 cm  
Provided Spacing = 25 cm  
Provided Area = 316 mm^2
Area of Steel Required on Outer face
-ve Ast = \( \frac{171.02 \times 100}{1500 \times 0.87 \times 16.5} \) = 0.79 cm²
Dia. Of bar (f) = 10 mm
Area of Bar = 79 mm²
Required Spacing = 36.41 cm
Provided Spacing = 25 cm
Provided Area = 316 mm²

3.16 DESIGN OF FLOOR SLAB :-
Let the Thickness of Slab is = 250 mm
Width of Panel = 4100 mm
Effective Span = 3.7
Density of Concrete = 2500 kg/m³
Dead Wt of Slab = 0.25 x 2500 = 625 kg/m²
Water Load on Slab = 3450 kg/m²
Floor Finish = 50 kg/m²
Total = 4125 kg/m²

- ve B.M = 0.032 x 4125 x 3.7² = 1807.08 Kg-m
+ ve B.M = 0.75 x 1807.08 = 1355.31 Kg-m
\[ \sigma_{bt} = \frac{1807.08 \times 6}{25^2} = 17.35 \text{ kg/cm}^2 \]
Permissible Bending Tension = 18 kg/cm²
Provided Thickness is O.K

Effective thickness = 21.5 cm
-ve Ast = \( \frac{1807.08 \times 100}{1500 \times 0.87 \times 21.5} \) = 6.44 cm²
+ve Ast = \( \frac{1355.31 \times 100}{1500 \times 0.87 \times 6.44} \) = 4.83 cm²
Minimum Area of Steel = 5.15 cm²
Dia. Of bar (f) = 10 mm
Area of Bar = 79 mm²
Required Spacing = 12.27 cm
Provided Spacing = 12 cm
Provided Area = 658.33 mm²
Provide 10dia tor @ 12 cm c/c bothways at top
Dia. Of bar (f) = 10 mm
Area of Bar = 79 mm²
Required Spacing = 16.36 cm
Provided Spacing = 16 cm
Provided Area = 493.75 mm²

Provide 10 dia tor @ 16 cm c/c bothways at bottom
Provide 10 dia tor @ 160 c/c bothways at bottom & alternate bars bent
Provide 10 dia tor @ 160 c/c bothways at top

Check for Deflection:
L / d = 26
% of Compression Reinforcement P_c = 0
Multiplication Factor for Tension Reinforcement = 1.51
Multiplication Factor for Compression Reinforcement = 1
Modified L/ d Ratio = 39.28
Actual L/d Ratio = 19.07

Hence, SAFE

3.17) DESIGN OF FLOOR BEAM :-
Length of Span = 4.1 m
Let Width of Beam = 30 cm
Let Depth of Beam = 60 cm
Clear Cover = 3 cm
Effective Depth = 56.2 cm
Load from Slab = (π / 8) x 4.1² x 4125 = 27230.25 kg
U.D.L = 262.5 kg/m
S.F = (27230.25+262.5x4.1) / 2 = 14153.25 kg
- ve B.M = (5/48) x 27230.25x4.1+262.5x4.1²/12 = 11997.3 Kg-m
Net - ve B.M = 11997.3-14153.25x0.4/3 = 10110.2 Kg-m
+ ve B.M = 0.6x (5/48) x 27230.25x4.1+0.5 x262.5x4.1²/12 = 7161.61 Kg-m
Check for Non Cracking:
\[ b_f = \left( \frac{L_o}{6} \right) + b_w + 6D_f \]

\[ D_f = 25 \text{ cm} \]
\[ L_o = 0.7L = 287 \text{ cm} \]
\[ b_f = \left( \frac{L_o}{6} \right) + b_w + 6D_f = 227.8333 \text{ cm} \]

Area of flange Portion
\[ A_1 = 5695.8333 \text{ cm}^2 \]

Area of Web Beam
\[ A_2 = 1050 \text{ cm}^2 \]

Total Area
\[ A = 6745.8333 \text{ cm}^2 \]

Moment of Inertia
\[ I = 1278794.6 \text{ cm}^4 \]
\[ Z = 74478.429 \text{ cm}^3 \]

Bending Tension
\[ \sigma_{bt} = 13.57 \text{ kg/cm}^2 \]

Allowable Bending Tension
\[ = 18 \text{ kg/cm}^2 \]

Hence Safe

How ever Provided Overall depth is 60cm O.K

Area of steel required for Support
\[ = \frac{10110.2 \times 100}{(1500 \times 0.87 \times 56.2)} = 13.79 \text{ cm}^2 \]

Min Area of Steel = 0.85 bd / fy
\[ = 3.45 \text{ cm}^2 \]

At Top:

<table>
<thead>
<tr>
<th></th>
<th>Straight Bars</th>
<th>Extra Bars</th>
</tr>
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<tr>
<td>Dia of Bar</td>
<td>16 mm</td>
<td>20 Mm</td>
</tr>
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<td>Area of Bar</td>
<td>2.01 cm²</td>
<td>3.14 cm²</td>
</tr>
<tr>
<td>Required no of Bars</td>
<td>6.86 no's</td>
<td>2.47 no's</td>
</tr>
<tr>
<td>Provide no of Bars</td>
<td>3 no's</td>
<td>3 no's</td>
</tr>
<tr>
<td>Provided Area of Steel</td>
<td>6.03 cm²</td>
<td>9.42 cm²</td>
</tr>
</tbody>
</table>

Provide 3-16tor through + 3-20 tor extra at Supports (15.45sqcm)

Area of steel required for Span
\[ = \frac{7161.61 \times 100}{(1900 \times 0.89 \times 56.2)} = 7.54 \text{ cm}^2 \]
At Bottom:

<table>
<thead>
<tr>
<th></th>
<th>Straight Bars</th>
<th>Extra Bars</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dia of Bar</td>
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<td>12 Mm</td>
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<tr>
<td>Area of Bar</td>
<td>2.01 cm²</td>
<td>1.13 cm²</td>
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<tr>
<td>Required no of Bars</td>
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<td>2 no's</td>
</tr>
<tr>
<td>Provided Area of Steel</td>
<td>6.03 cm²</td>
<td>2.26 cm²</td>
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</tbody>
</table>

Provide 3-16tor through + 2-12 TOR extra at mid span (8.29sqcm)

% of Steel Provided = 100 x15.45 /
(30x56.2) = 0.92

Permissible shear stress in concrete ( tc ) = 3.872 kg/cm²

Nominal shear stress ( tv ) = Vu/bd
=14153.25/ (30*56.2) = 8.395 kg/cm²

Net Shear force =14153.25-
(3.872*30*56.2) = 7625.06 kg

Stirrup Dia = 8 mm

No of legs = 2

Area of Bar = 1.01 cm²

Spacing Required =
(1.01*1900*56.2)/7625.06 = 14.1 cm

Spacing required is Min. of following
(0.85*fy *n*π/4 * d² / ( tv - tc)B = 145 mm
Max. = 300 mm

0.75*d = 422 mm

Minimum Shear Reinforcement = Asv / bSv >= (0.4/0.87fy )

Sv <= 0.87fyAsv / 0.4b

Sv <= = 30.39 cm

0.75 d = 42.15 cm

= 30 cm

Provided Spacing is Lesser of above two cases = 14.14 cm

Say = 14.00 cm
3.18) **DESIGN OF FLOOR RING BEAM**: 

Length of Span = 6.44 m  
Let Width of Beam = 30 cm  
Let Depth of Beam = 30 cm  
Clear Cover = 3 cm  
Effective Depth = 26.4 cm  

Results from STAAD Pro  
Max Bending Moment at Support = 11.26 KN-m  
Max Bending Moment at Span = 11.25 KN-m  
Max Shear Force = 34.8 KN  
Max Bending Moment at face of Support = 4.485 KN-m  
Un-cracked depth required = \sqrt{44850 \times (30 \times 3.33)} = 21.19 cm  

However Provided Overall depth is 30cm O.K  

Area of steel required for Support = 11.26 \times 10000 / (1500 \times 0.87 \times 26.4) = 3.27 cm²  

Min Area of Steel = 0.85 bd / fy = 1.62 cm²  

**At Top:**  

| Dia of Bar | = | 12 | mm |
| Area of Bar | = | 1.13 | cm² |
| Required no of Bars | = | 2.89 | no's |
| Provide no of Bars | = | 3 | no's |
| Provided Area of Steel | = | 3.39 | cm² |

Provide 3-12tor through (3.39sqcm )  

Area of steel required for Span = 11.25 \times 10000 / (1900 \times 0.89 \times 26.4) = 2.52 cm²  

**At Bottom:**  

| Dia of Bar | = | 12 | mm |
| Area of Bar | = | 1.13 | cm² |
| Required no of Bars | = | 2.23 | no's |
| Provide no of Bars | = | 3 | no's |
| Provided Area of Steel | = | 3.39 | cm² |

Provide 3-12tor through at bottom (3.39sqcm )
% of Steel Provided = 100 x 3.39 / (30x26.4) = 0.43
Permissible shear stress in concrete (tc) = 4.594 kg/cm²
Nominal shear stress (tv) = Vu/bd = 3480 / (30x26.4) = 4.394 kg/cm²
Net Shear force = 3480 - (4.594 x 30 x 26.4) = -158.45 kg
Stirrup Dia = 8 mm
No of legs = 2
Area of Bar = 1.01 cm²
Spacing Required = (1.01**26.4) / -158.45 = -2.4 cm
Spacing required is Min. of following
(0.85*fy *n*π /4 * d² / (tv - tc)B = -3275 mm
Max. = 300 mm
0.75*d = 198 mm
Minimum Shear Reinforcement = Asv / bSv >= (0.4/0.87fy)
Sv <= 0.87fyAsv / 0.4b

<table>
<thead>
<tr>
<th>Spacing Required</th>
<th>30.39 cm</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.75 d</td>
<td>19.8 cm</td>
</tr>
<tr>
<td></td>
<td>30 cm</td>
</tr>
</tbody>
</table>

Provided Spacing is Lesser of above two cases = 19.80 cm
Say = 18.00 cm
Provide 8 dia tor @ 180 c/c through out

3.19) Design of one meter wide gallery slab:
Length of Walkway = 1 m
Depth of Slab Required = L / 7 = 142.857 mm
Provided thickness of at Support = 150 mm
Thickness at free end = 100 mm
Loading:
(1) Self Wt of Slab = 0.125 x 2500 = 312.5 kg/m²
(2) Live Load = 300 kg/m²
(3) Finishes = 50 kg/m²
Total Load W = = 662.5 kg/m²
Max B.M = WL² / 2 = 662.5 x 1 x 1² / 2 = 331.25 Kg-m
Clear Cover = 2.5 cm
Effective Depth = 12.5 cm
Ast Required = 33125 / (2300 x 0.9 x 12.5) = 1.28 cm²
Min Area of Steel = 0.12 %
Ast = (0.12/100) x15 x 100 = 1.8 cm²

Note: The Floor Slab Top Reinforcement Should Extend up to end of Walk way Slab.

3.20) Design of Column inside Container:
Size of Column = 200 фр
Clear Cover = 40 mm
Start IL = 356.025 m
Floor Level = 352.575 m
Total Height of Column = 3.45 m
Clear Ht of Column = 3.45 m
Effective Length of Column = 4.14 m
L/d = 20.7 >12
It will Designed as Long Column
Reduction factor = 1.25 – (20.7 /48 ) = 0.82
Axial load (Unfactored) = 7632 kgs
Max B.M (Unfactored) = 0 Kg-m
Max B.M (Unfactored) = 0 Kg-m
Design Load = 7632/0.82 = 9307.32 kg
Say = 10000 kg

Load carrying capacity =60( a-0.01a) +1900 x 0.01a
10000 = 60a -0.6a+19a

a = 127.55 cm²
Provided C/S Area = 314.16 cm²
Min % of Steel Required = 0.8 %
Min Area of Steel Required = 2.51 cm²
Provide dia of bar = 12 mm
Area of Bar = 1.13 cm²
Required no of Bars = 2.22 no's
Provided no of Bars = 6 no's
Provide 6 no's 12 tor = 6.78 cm²
Provide 8 tor @ 150 c/c links
Check in Working Stress Method:

Axial load (Unfactored) = 7632 kgs
Max B.M (Unfactored) = 0 kgm
Max B.M (Unfactored) = 0 kgm

\[ A = Asc + (1.5m-1)At \]
\[ (20^2 \times \frac{\pi}{4}) + (1.5*10.98-1)*6.78 \]
\[ I = \left( \frac{\pi d^4}{64} \right) + \left( (1.5m-1) \times At \times (\frac{d}{2} - x) \right)^2 \]
\[ \frac{d}{2} = 10 \text{ cm} \]
\[ X = 4.6 \text{ cm} \]
\[ I = \left( \frac{\pi}{4} \times 20^2 / 64 \right) + (1.5*10.98-1)*3.39*(10-4.6)^2 \]
\[ \frac{Z}{= \left( \frac{\pi}{4} \times 20^2 / 64 \right) + (1.5*10.98-1)*3.39*(10-4.6)^2} \]
\[ = 9383.23 \text{ cm}^4 \]
\[ \frac{Z}{d/2} = 938.32 \text{ cm}^3 \]
\[ \sigma_{cc \text{ Cal}} = \frac{P}{A} = 18.21 \text{ kg/cm}^2 \]
\[ \sigma_{cbc \text{ Cal}} = \frac{My}{Z} = 0 \text{ kg/cm}^2 \]
\[ \sigma_{cbc \text{ Cal}} = \frac{Mz}{Z} = 0 \text{ kg/cm}^2 \]
\[ \frac{(\sigma_{cc \text{ Cal}} / \sigma_{cc}) + (\sigma_{cbc \text{ Cal}} / \sigma_{cbc}) + (\sigma_{cbc \text{ Cal}} / \sigma_{cbc})}{= 0.36 < 1} \]

Hence Safe

Load Carrying Capacity = 60x(\left( \frac{\pi}{4} \times 20^2 \right) - 6.78)+ 1900x6.78
\[ = 31324.76 \text{ kgs} \]

Weight of Container:

Roof Slab = \( \frac{\pi}{4} \times 8.2^2 \times 500 \)
\[ = 26405.09 \text{ kgs} \]

Roof Beam = 4x0.2x0.18x4.1x2500
\[ = 1476 \text{ kgs} \]

Wall = 2\( \times \frac{\pi}{4} \times 4.1 \times 3.45 \times 0.2 \times 2500 \)
\[ = 44437.83 \text{ kgs} \]

Floor Slab = \( \frac{\pi}{4} \times 8.2^2 \times 675 \)
\[ = 35646.87 \text{ kgs} \]

Floor Beam = 4x0.3x0.35x4.1x2500
\[ = 4305 \text{ kgs} \]

Floor Ring Beam = 2\( \times \frac{\pi}{4} \times 4.1 \times 0.3 \times 0.3 \times 2500 \)
\[ = 5796.24 \text{ kgs} \]

Gallery = \( \frac{\pi}{4} \times (10.4^2 - 8.4^2) \times 662.5 \)
\[ = 19564.27 \text{ kgs} \]

Roof Slab Excluding Live Load Portion = \( \frac{\pi}{4} \times 8.2^2 \times 400 \)
\[ = 21124.07 \text{ kgs} \]

Gallery Excluding Live load portion = \( \frac{\pi}{4} \times (10.4^2 - 8.4^2) \times 362.5 \)
\[ = 10704.98 \text{ kgs} \]

Weight of Internal Column = \( \frac{\pi}{4} \times 0.2^2 \times 3.45 \times 2500 \)
\[ = 270.96 \text{ kgs} \]

Water = \( \frac{\pi}{4} \times 8^2 \times 3.45 \times 1000 \)
\[ = 173415.91 \text{ kgs} \]

Water in free board portion = \( \frac{\pi}{4} \times 8^2 \times 0.3 \times 1000 \)
\[ = 15079.64 \text{ kgs} \]
Total Wt of container in full condition Excluding Freeboard Portion = 296238.53 kgs
Total Wt of container Including Freeboard Portion = 311318.17 kgs
Wt of empty container = 123761.95 kgs
Wt of Container in full condition excluding Freeboard & Live load = 282098.22 kgs
Height of C.G of empty container from top of floor slab will be
\[
\frac{(21124.07 \times 3.51 + 1476 \times 3.6 + 44437.83 \times 1.725 + 270.96 \times 1.725 - 35646.87 \times 0.125 - 5796.24 \times 0.15 - 10704.98 \times 0.075)}{123761.95}
\]
C.G of Empty Container = 1.22 m
Height of C.G of container full from top of floor slab will be
\[
\frac{(26405.09 \times 3.51 + 1476 \times 3.6 + 44437.83 \times 1.725 + 270.96 \times 1.725 + 173415.91 \times 1.725 - 35646.87 \times 0.125 - 5796.24 \times 0.15 - 19564.27 \times 0.075)}{311318.17}
\]
C.G of container full = 1.5 m

3.21) Design of Staging:
No of Columns = 5
Column Size = 400 Φ
Brace Levels = 3
Size of Braces = 250x 350
Floor Ring Beam = 300x 300
Depth of foundation below G.L = 3 m
Height of Wall Portion h3 = (3+0.3+0.15) = 3.45 m
Height of IV Column Panel = 3.45 m
Depth of III Brace = 0.35 m
Height of III Column Panel = 3.6 m
Depth of II Brace = 0.35 m
Height of II Column Panel = 3.6 m
Depth of I Brace = 0.35 m
Height of I Column Panel = 2.5 m
Height of Column from top of footing to bottom of Floor beam = 14.2 M
3.22) WIND ANALYSIS:
Design Wind pressure \( Pz = 0.60 \times Vz^2 \)

Basic wind pressure \( Vb = 39 \) m/s

Probability factor \( k1 = 1.06 \)
for Category 2, class 'A' & height bet 20m - 50m
\( k2 = 1.056 \)

Topography factor \( k3 = 1 \)

Design Wind Speed \( Vz = Vb \times k1 \times k2 \times k3 \)
\( Vz = 39 \times 1.06 \times 1.056 \times 1 = 43.66 \) m/s

\( Pz = 0.60 \times Vz^2 \)
\( Pz = 0.60 \times 43.66^2 = 1143.72 \) N/m²
\( = 114.372 \) Kg/m²

Say Wind Pressure = 150 Kg/m²

From Appendix D, from Table 23, Circular plan shape, direction of wind pressure to diagonal
\( Cf = 0.7 \)

Cross sectional area of Column \( ao = 0.1257 \) m²

Column is in Circular in Shape, Shape factor = 0.7

Wind Load on Column = 150x0.7x0.4 = 42 kg/m
Wind Load on Brace Beams = 150x0.35 = 52.5 kg/m
Wind Load on Container = 150x0.7 = 105 kg/m

These Wind Loads Applied in Staad pro

3.23) EARTHQUAKE ANALYSIS

Seismic Zone = III
Type of Frame : S.M.R.F

DEAD & LIVE LOADS:

A) Container:
Load from Base (Floor) beam = 296238.53 kg
Container full = 296238.53 kg

Weight of Water = 158.34*1000 = 158340 kg
Container Empty = 137898.53 kg

B) Staging: (Above top of footing)
Columns = 5*14.2*π/4*0.4^2*2500 = 22305.31 kg
Inner Braces = 4*3*4.1*0.25*0.35*2500 = 10762.5 kg
Outer Braces = \(4 \times 3 \times 5.798 \times 0.25 \times 0.35 \times 2500\) = 15219.75 kg  
Weight of staging = 48287.56 kg

Lumped weights:
A) Tank empty  
\(\text{We} = 137898.53 + (48287.56/3)\) = 153994.38 kg

B) Tank full  
\(\text{Wf} = 296238.53 + (48287.56/3)\) = 312334.3833 kg

Stiffness of Staging (carried out in Staad pro 2007): Refer (Appendix -)
Delta = 3.34 mm  
load applied = 1000 Kg  
\(K = \text{Load applied} / \text{Delta}\) = 299401.2 kg/m

Fundamental period:
\(T = 2 \pi \sqrt{\frac{W}{gK}}\)
A) Tank Empty  
\(W = 153994.38\) kg  
\(G = 9.81\)  
\(K = 299401.2\)  
\(T = 2 \pi \sqrt{\frac{153994.38}{9.81 \times 299401.2}}\) = 1.439 sec

B) Tank full  
\(W = 312334.38\) kg  
\(G = 9.81\)  
\(K = 299401.2\)  
\(T = 2 \pi \sqrt{\frac{312334.38}{9.81 \times 299401.2}}\) = 2.049 sec

Average acceleration coefficient:
Assuming a damping of 5 percent of critical for the above periods the average acceleration coefficient from fig 2 of I.S. code 1893-2002 for Soft soils
A) Tank empty:
If \(0.67 \leq T \leq 4\), then \(\frac{S_a}{g} = \frac{1.67}{T}\) = 1.161

B) Tank full:
\(\frac{S_a}{g} = \frac{1.67}{T}\) = 0.815

Horizontal Seismic coefficient:
\(Ah = \frac{Z \times I \times S_a}{(2 \times R \times g)}\)
From IS 1893 (Part-1):2002, Table 2
\[ Z = 0.16 \]

From IS 1893 (Part-1):2002, Table 6
\[ I = 1.5 \]

From IITK-GSDMA Guidelines, Table 2
\[ R = 5 \]

These Parameters assigned in STAAD Modeling, and the Lumped mass for Tank Full Condition and Tank Empty Condition is applied at C.G of Tank.

The Analysis was done by Different Loading Combinations as Per IS 1893 - 2002.

The Different Loading Conditions for Tank Full condition are

- DL+LL+WATERLOAD, 1.5(DL+LL+WATERLOAD), 1.5(DL+WATERLOAD+E Q+X),
- 1.5(DL+WATERLOAD+EQ-X), 1.5(DL+WATERLOAD+EQ+Z),
- 1.5(DL+WATERLOAD+EQ-Z), 1.5(DL+WATERLOAD+WL+X),
- 1.5(DL+WATERLOAD+WL-X), 1.5(DL+WATERLOAD+WL+Z),
- 1.5(DL+WATERLOAD+WL-Z), 1.2(DL+WATERLOAD+LL+E Q+X),
- 1.2(DL+WATERLOAD+LL+EQ-X), 1.2(DL+WATERLOAD+LL+EQ+Z),
- 1.2(DL+WATERLOAD+LL+EQ-Z), 1.2(DL+WATERLOAD+LL+WL+X),
- 1.2(DL+WATERLOAD+LL+WL-X), 1.2(DL+WATERLOAD+LL+WL+Z),
- 1.2(DL+WATERLOAD+LL+WL-Z)

The Different Loading Conditions for Tank Empty condition are

- DL+LL, 1.5(DL+LL), 1.5(DL+E Q+X), 1.5(DL+E Q-X), 1.5(DL+E Q+Z),
- 1.5(DL+E Q-Z), 1.5(DL+WL+X), 1.5(DL+WL-X), 1.5(DL+WL+Z),
- 1.5(DL+WL-Z), 1.2(DL+LL+E Q+X), 1.2(DL+LL+E Q-X),
- 1.2(DL+LL+E Q+Z), 1.2(DL+LL+E Q-Z),
- 1.2(DL+LL+WL+X), 1.2(DL+LL+WL-X), 1.2(DL+LL+WL+Z), 1.2(DL+LL+WL-Z)

The Column & Brace Beams are Designed for Critical Loading Condition
3.24) Design of External Brace Beams:

Length of Span = 5.798 m
Let Width of Beam = 250 mm
Let Depth of Beam = 350 mm
Clear Cover to Main Reinforcement d' = 30 mm
Effective Depth d = 314 mm
d' / d = 0.0955

Critical Loading Condition: Refer Appendix

Grade of Concrete = M 20
Grade of Steel = Fe415
\( F_{ck} = 20 \text{ N/mm}^2 \)
\( F_y = 415 \text{ N/mm}^2 \)

Moment at end Support = 29.85 KN-m
Moment in Span = 13.60 KN-m
Shear Force = 16.69 KN
Torsion = 0.45 KN-m
Equivalent Moment due to Torsion at support (KN-m) = 35.63 KN-m
Equivalent Moment due to Torsion at Mid Span (KN-m) = 19.38 KN-m
Equivalent Shear due to Torsion at support (KN) = 16.73 KN-m
From Table D, \( M_{ulim} / bd^2 = 2.76 \)

\( M_{ulim} = 84.53 \text{ KN-m} \)

Actual moment is less than limiting moment, therefore, the section to be designed as Singly reinforced beam

For Referring to tables, we need the value of \( M_u / bd^2 \)
At Supports \( M_u / bd^2 \)
35.63x1000000 / 250x314^2 = 1.4
From SP -16, Table : 2 , \( P_t = 0.441 \)
\( A_{st} = (0.441/100) \times 250 \times 314 \)
\( = 346.2 \text{ mm}^2 \)
At Span \( M_u / bd^2 \)
19.38x1000000 / 250x314^2 = 0.8
From SP -16, Table : 3 , \( P_t = 0.229 \)
\( A_{st} = (0.229/100) \times 250 \times 314 \)
\( = 179.8 \text{ mm}^2 \)
At Supports \( P_c \) (from SP:16) (%) = 0.0 %
At Span \( P_c \) (from SP:16) (%) = 0.0 %
Asc required (mm²) at top = 0.0 mm²
Asc required (mm$^2$) at bottom = 0.0 mm$^2$
Minimum Area of Steel = $0.85 \times b \times d / \gamma_y = 160.78$ mm$^2$

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<tr>
<td>Area of Bar</td>
<td>113.1 mm$^2$</td>
</tr>
<tr>
<td>Required no of Bars</td>
<td>1.59 no's</td>
</tr>
<tr>
<td>Provide no of Bars</td>
<td>2 no's</td>
</tr>
<tr>
<td>Provided Area of Steel</td>
<td>226.2 mm$^2$</td>
</tr>
</tbody>
</table>

Provide 2-12tor through (226.2sqmm)

At Top:

<table>
<thead>
<tr>
<th>Straight Bars</th>
<th>Extra Bars</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dia of Bar</td>
<td>12 mm</td>
</tr>
<tr>
<td>Area of Bar</td>
<td>113.1 mm$^2$</td>
</tr>
<tr>
<td>Required no of Bars</td>
<td>3.06 no's</td>
</tr>
<tr>
<td>Provide no of Bars</td>
<td>2 no's</td>
</tr>
<tr>
<td>Provided Area of Steel</td>
<td>226.2 mm$^2$</td>
</tr>
</tbody>
</table>

Provide 2-12tor through+1-16tor extra at top (427.26sqmm)

Nominal shear stress ($\tau_v$) = $\frac{V}{b \times d} = 16.73 \times 100 / (25 \times 31.4) = 15.287$ kg/cm$^2$

% of Steel Provided Tension reinforcement (in supp) = $100 \times 427.26 / (250 \times 314) = 0.54$

Design Shear Strength of concrete ($\tau_c$) (From Table 19 of IS 456-2000) = 4.933 kg/cm$^2$

Maximum Shear stress $\zeta_{c_{\text{max}}}$ = 2.800 kg/cm$^2$

Minimum Shear Reinforcement Required

Net Shear force = $16.73 \times 100 - (4.933 \times 25 \times 31.4) = 1$ kg

Stirrup Dia = 8 mm
Stirrup legs = 2
Area of Bar = 100.53 mm$^2$
\[ V_{us} = 0.87 \times f_y A_{sv} \times d / S_v \]
Spacing Required = \((0.87 \times 415 \times 100.53 \times 314) / 21994.1\) = -518.19 mm

Spacing Required is Min. of following
Max Spacing of Shear reinforcement = 300 mm
0.75*d = 236 mm

Minimum Shear Reinforcement = \(\frac{A_{sv}}{b S_v} \geq (0.4 / 0.87 f_y)\)
\[ S_v = 0.87 f_y A_{sv} / 0.4 b \]

Provided Spacing is Lesser of above four cases = 235.50 mm

As per IS 13920, Spacing of Hoops for a distance of 2d from face of support shall be Min of below two conditions

Dia of Hoop Bar = 8 mm
No of Legs = 2
2d = 628 mm
Say = 650 mm

a.) \(d/4\) = 7.85 cm
b.) 8 times of Dia of Longitudinal Bar = 8 \(d_b\) = 9.6 cm
Provided Spacing is Lesser of above two cases = 7.85 cm
Say = 7.5 cm

As per IS 13920, Spacing of Hoops is for center portion should not Exceed (d/2)

\[ d/2 = 15.7 \text{ cm} \]
Say = 15 cm

Stirrups:
Provide Support to 650 mm , 8 dia tor 2L stirrups @ 7.5 cm c/c
650 to center , 8 dia tor 2L stirrups @ 15 cm c/c

3.25 Design of Internal Brace Beams:
Length of Span = 4.1 mt
Let Width of Beam = 250 mm
Let Depth of Beam = 350 mm
Clear Cover to Main Reinforcement \(d'\) = 30 mm
Effective Depth \(d\) = 314 mm
\[ d' / d = 0.0955 \]

Critical Loading Condition: Refer Appendix
Grade of Concrete = M 20
Grade of Steel = Fe415

\[ F_{ek} = 20 \text{ N/mm}^2 \]
\[ F_y = 415 \text{ N/mm}^2 \]

Moment at end Support = 39.66 KN-m
Moment in Span = 32.02 KN-m
Shear Force = 24.21 KN
Torsion = 1.04 KN-m
Equivalent Moment due to Torsion at support (KN-m) = 52.97 KN-m
Equivalent Moment due to Torsion at Mid Span (KN-m) = 45.33 KN-m
Equivalent Shear due to Torsion at support (KN) = 24.29 KN-m

From Table D, \( \frac{M_{ulim}}{bd^2} \) = 2.76
\( M_{ulim} = 84.53 \text{ KN-m} \)

Actual moment is less than limiting moment, therefore, the section to be
designed as Singly reinforced beam
For Referring to tables, we need the value of \( \frac{M_u}{bd^2} \)
At Supports \( \frac{M_u}{bd^2} \)
52.97 x 10^6 / 250 x 314^2 = 2.1
From SP -16, Table : 2 , \( P_t \) = 0.696
Ast = (0.696/100) x 250 x 314 = 546.4 mm^2

At Span \( \frac{M_u}{bd^2} \)
45.33 x 10^6 / 250 x 314^2 = 1.8
From SP -16, Table : 3 , \( P_t \) = 0.579
Ast = (0.579/100) x 250 x 314 = 454.5 mm^2

At Supports \( P_c \) (from SP:16) (%) = 0.0 %
At Span \( P_c \) (from SP:16) (%) = 0.0 %
Asc required (mm^2) at top = 0.0 mm^2
Asc required (mm^2) at bottom = 0.0 mm^2
Minimum Area of Steel = 0.85 x b x d / fy = 160.78 mm^2
### At Bottom:

<table>
<thead>
<tr>
<th></th>
<th>Straight Bars</th>
<th>Extra Bars</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dia of Bar</td>
<td>12 mm</td>
<td>16 mm</td>
</tr>
<tr>
<td>Area of Bar</td>
<td>113.1 mm²</td>
<td>6 mm²</td>
</tr>
<tr>
<td>Required no of Bars</td>
<td>4.02 no's</td>
<td>1.14 no's</td>
</tr>
<tr>
<td>Provide no of Bars</td>
<td>2 no's</td>
<td>2 no's</td>
</tr>
<tr>
<td>Provided Area of Steel</td>
<td>226.2 mm²</td>
<td>2 mm²</td>
</tr>
</tbody>
</table>

Provide 2-12tor through + 2-16tor extra at bottom (628.32 sqmm)

### At Top:

<table>
<thead>
<tr>
<th></th>
<th>Straight Bars</th>
<th>Extra Bars</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dia of Bar</td>
<td>12 mm</td>
<td>16 mm</td>
</tr>
<tr>
<td>Area of Bar</td>
<td>113.1 mm²</td>
<td>6 mm²</td>
</tr>
<tr>
<td>Required no of Bars</td>
<td>4.83 no's</td>
<td>1.59 no's</td>
</tr>
<tr>
<td>Provide no of Bars</td>
<td>2 no's</td>
<td>2 no's</td>
</tr>
<tr>
<td>Provided Area of Steel</td>
<td>226.2 mm²</td>
<td>2 mm²</td>
</tr>
</tbody>
</table>

Provide 2-12tor through + 2-16tor extra at top (628.32 sqmm)

Nominal shear stress (tv) = \( \frac{V_u}{bd} = \frac{24.29 \times 100}{(25 \times 31.4)} \) = 15.287 kg/cm²

% of Steel Provided Tension reinforcement (in supp) = \( \frac{100 \times 628.32}{(250 \times 314)} \) = 0.80

Design Shear Strength of concrete (tc) (From Table 19 of IS 456-2000) = 5.737 kg/cm²

Maximum Shear stress \( \zeta_{cmax} \) = 2.800 kg/cm²

Minimum Shear Reinforcement Required = 2074.5 kg

Net Shear force = 24.29 \times 100 - (5.737 \times 25 \times 31.4) = 5 kg

Stirrup Dia = 8 mm

Stirrup legs = 2

Area of Bar = 100.53 mm²
\[ V_{us} = 0.87 \times f_y x A_{sv} x d / S_v \]

Spacing Required = \((0.87 \times 415 \times 100.53 \times 314) / 20745.5\) = -549.37 mm

Spacing Required is Min. of following
Max Spacing of Shear reinforcement = 300 mm
0.75*d = 236 mm

Minimum Shear Reinforcement = \(A_{sv} / b_{sv} \geq (0.4 / 0.87f_y)\)
\[ S_v \leq 0.87f_y A_{sv} / 0.4b \]

Provided Spacing is Lesser of above four cases = 235.50 mm

As per IS 13920, Spacing of Hoops for a distance of 2d from face of support shall be Min of below two conditions

<table>
<thead>
<tr>
<th>Dia of Hoop Bar</th>
<th>= 8 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>No of Legs</td>
<td>= 2</td>
</tr>
<tr>
<td>2d</td>
<td>= 628 mm</td>
</tr>
<tr>
<td>Say</td>
<td>= 650 mm</td>
</tr>
</tbody>
</table>

a.) \(d / 4\)
b.) 8 times of Dia of Longitudinal Bar = 8 x \(d_b\) = 9.6 cm

Provided Spacing is Lesser of above two cases = 7.85 cm

Say = 7.5 cm

As per IS 13920, Spacing of Hoops is for center portion should not Exceed (d/2)

\[ d / 2 = 15.7 \text{ cm} \]

Say = 15 cm

Stirrups:
Provide Support to 650 mm, 8 dia tor 2L stirrups @ 7.5 cm c/c
650 to center, 8 dia tor 2L stirrups @ 15 cm c/c

3.26) Design of Columns:

<table>
<thead>
<tr>
<th>Size of Column</th>
<th>= 400 (\Phi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>GL</td>
<td>= 340.73 m</td>
</tr>
<tr>
<td>Start IL</td>
<td>= 353.48 m</td>
</tr>
<tr>
<td>Depth of foundation below G.L</td>
<td>= 3.00 m</td>
</tr>
<tr>
<td>Total Height of Column</td>
<td>= 15.00 m</td>
</tr>
<tr>
<td>Clear Ht of Column</td>
<td>= 3.60 m</td>
</tr>
<tr>
<td>Effective Length of Column</td>
<td>= 4.32 m</td>
</tr>
<tr>
<td>L/d</td>
<td>= 10.80 &lt;12</td>
</tr>
</tbody>
</table>
It will be Designed as Short Column

Critical Loading Condition: Refer Appendix

Column Depth \((D_c)\) = 0.4 m

Characteristic of concrete \((f_{ck})\) = 20 N/mm\(^2\)

Characteristic strength of steel \((f_y)\) = 415 N/mm\(^2\)

Partial safety factor for loads \((f)\) = 1.5

Clear cover for column \((d')\) = 0.04 m

Factor Load \((Pu)\) from STAAD results = 898.573 kN

Moment from analysis along Z-direction \((M_{uz})\) = 56.636 kN-m

Moment from analysis along Y-direction \((M_{uy})\) = 0.001 kN-m

Moment due to minimum eccentricity along Z-direction \((M_{uz})\)

\[ e_{min} = \frac{L}{500} + \frac{D}{30} = 0.021 \]

\[ M_{uz} = Pu \times ez = 18.9 \text{ kN-m} \]

Moment due to minimum eccentricity along Y-direction \((M_{uy})\)

\[ e_{min} = \frac{L}{500} + \frac{D}{30} = 0.021 \]

\[ M_{uy} = Pu \times ey = 18.87 \text{ kN-m} \]

Final Moment in Z-direction = 56.636 kN-m

Final Moment in Y-direction = 18.87 kN-m

Assume reinforcement percentage, \(p\) = 1.5

\[ \frac{p}{f_{ck}} = 0.075 \]

Uniaxial moment capacity about zz-axis:

\[ \frac{d'}{D} = 0.1 \]

\[ \frac{Pu}{f_{ck}} \times D^2 = 0.281 \]

\[ \frac{M_u}{f_{ck}} \times D^3 \text{ (from chart 56 of SP-16)} = 0.07 \]

Uniaxial moment capacity about zz-axis \((M_{uz1})\) = 89.6 KN-m

Uniaxial moment capacity about yy-axis:

\[ \frac{d'}{D} = 0.1 \]

\[ \frac{M_u}{f_{ck}} \times D^3 \text{ (from chart 56 of SP-16)} = 0.07 \]

Uniaxial moment capacity about yy-axis \((M_{uy1})\) = 89.6 KN-m

Calculation of \(P_{uz}\):

\[ P_{uz} / A_g \text{ (from chart 63 of SP-16)} = 13.5 \text{ N/mm}^2 \]

Capacity of section of under pure axial load \((P_{uz})\) = 1696.46 KN

\[ \frac{Pu}{P_{uz}} = 0.53 \]

\[ a_u \text{ (from SP-16)} = 1.55 \]

\[ \frac{M_{uz}}{M_{uz1}} = 0.632 \]

\[ \frac{M_{uy}}{M_{uy1}} = 0.211 \]
\[(M_{uy}/M_{uy1})^n + (M_{uz}/M_{uz1})^n = 0.581 < 1\]

Area of Steel Required \( A_{streq} \) = 1884.96 \( \text{mm}^2 \)

\[A_{streq} = p * B_c * D_c\]

Provide 20 nos

Hence, provide 8 no.s of 20 mm dia Fe-415 bars

Area of Steel \( A_{st} \) = 2513 \( \text{mm}^2 \)

Design of ties:

Minimum diameter of lateral ties = \( 1/4 \) Dia Of Long. Bars = 5 mm

Diameter Of Ties Provided = 8 mm

C/C Spacing Of Ties Shall Be Least Of The Following:

Least Lateral Dimension Of Column = 400

16 x Diameter Of Long. Bars = 16 x 20 = 320 mm

48 x Diameter Of Ties = 48 x 8 = 384 mm

C/C Spacing Of Ties Required = 320 mm

Provide spacing of ties = 320 mm

8 tor ties @ 200 mm c/c

Check in Working Stress Method:

Axial load (Unfactored) = 59905 kgs

Max. B.M (Unfactored) = 0 Kg-m

Max. B.M (Unfactored) = 3776 Kg-m

\[A = A_{sc} + (1.5m-1)A_{t}\]

\[(40^2 * \pi/4) + (1.5*13.33-1)*25.13 = 1733.98 \text{ cm}^2\]

\[I = (\pi d^4 / 64) + (1.5m-1) \times A \times (d/2 - x)^2\]

\[d/2 = 20 \text{ cm}\]

\[X = 5 \text{ cm}\]

\[I = (\pi \times 40^4 / 64) + (1.5*13.33-1)*12.565*(20-5)^2\]

\[Z = I / (d/2) = 8968.25 \text{ cm}^3\]

\[\sigma_{cc \text{ Cal}} = P / A = 34.55 \text{ kg/cm}^2\]

\[\Sigma \sigma_{cbc \text{ Cal}} = M_y / Z = 0 \text{ kg/cm}^2\]

\[\Sigma \sigma_{cbc \text{ Cal}} = M_z / Z = 42.1 \text{ kg/cm}^2\]

\[(\sigma_{cc \text{ Cal}} / \sigma_{cc}) + (\sigma_{cbc y \text{ Cal}} / \sigma_{cbc}) + (\sigma_{cbc z \text{ Cal}} / \sigma_{cbc}) = 1.29 > 1\]
If Consider Earthquake loads / Wind Loads , we have to increase Permissible Stresses By 33% ,then
$(\sigma_{cc \text{ Cal}} / 1.33x\sigma_{cc}) + (\sigma_{cbc \text{ Cal}} / 1.33x\sigma_{cbc}) + (\sigma_{cbcz \text{ Cal}} / 1.33x\sigma_{cbc}) = 0.97 <1$

Hence Safe

Load Carrying Capacity =50x((\pi/4 * 40^2) - 18.8496)+1900x25.13 = 109322.35 kgs

Confining Links :
Column of Section 400 \( \Phi \)
Grade of Concrete fck = 20 N/mm²
Grade of Steel fy = 415 N/mm²
Spacing of Circular Hoops :
Spacing should be lesser of the following :
1.) 1/4 of Minimum Member Dimension = 100 mm
2.) 100 = 100 mm
Spacing of Hoops S = 100 mm
Clear Cover = 40 mm
Dia of Circular Hoops = 10 mm
Core Diameter measured to the outside of hoop Dk =400- 2x40+ 2x10 = 340 mm
Area of Concrete Core Ak =((\pi/4)*340^2) = 90792.03 mm²
Gross area of Column C/S Ag =((\pi/4)*400^2) = 125663.71 mm²
Area of C/S of bar forming circular hoop is Ash = 0.09 SDkfck/fy ((Ag/Ak)-1)
Ash = 0.09*100*340*(20/415)*((125663.71/90792.03)-1) = 56.64 mm²
Provided C/S of bar forming circular hoop = 78.54 mm²
Provided Circular Hoop bar of dia10mm is O.K
Thus Circular Hoops of Dia 10mm at a spacing of 100mm c/c will be adequate
Provided C/S of bar forming Tie h/6 = 600.000 mm
Say = 600 mm
Provided hoop bar of dia 10mm is O.K
Thus Ties of Dia 10mm at a spacing of 100mm c/c will be adequate for a height of (h/6) i.e. 600mm
3.27) **DESIGN OF RAFT FOUNDATION:**

- Load coming on to the Foundation = 432311 kgs
- Let Self weight of foundation (15%) = 64846.65 kgs
- Total load coming from Foundation = 497157.65 kgs
  - Say = 500000 kgs
- Depth of foundation below G.L = 3 m
- Safe Bearing Capacity of Soil = 6500 Kg/m²
- Area of Raft Required = 76.92 m²
- Side of Raft Required = 8.77 m
- Side of Raft to be provided = 8.8 m
- Area of Raft Provided = 77.44 m²
- Upward Pressure = 6456.61 kg/m²
- Net upward Pressure = 5582.53 kg/m²

3.28) **Check for Uplift:**

- Depth of foundation below ground level = 3 m
- Uplift Pressure on Foundation of Structure should be considered as per available water table at site in rainy season. However, minimum uplift up to 50% of depth of foundation below ground level for safety purpose may be considered.
- Depth of Water table Below G.L = 7 m
- So, Depth of Water table is far below Foundation Level
- For Uplift, 50% of depth of foundation below ground level should be considered
- Unit Wt of Water = 1000 kg/m³
- Uplift Pressure = 1.5*1000 = 1500 kg/m²
- Upward Load = 8.8² * 1500 = 116160 kgs
- Self Wt of Raft = 38720 kgs
- Self Wt of Raft Beam = 12896.4 kgs
- Weight of P.C.C = 18585.6 kgs
- Total Dead Wt of Structure Including staging = 186186.09 kgs
- Total Upward Load = 116160 kgs
- Total Downward Load = 256388.09 kgs

  - Factor of Safety = Total Downward load / Total Upward load = 2.21 >1.25
Design of Raft Slab :-

Let the Thickness of Slab is = 200 mm
Grade of Concrete = 25 kg/cm²
Grade of Steel = 415 kg/cm²

Triangular Slab
Size
b = 4.1 m
h = 5.798 m

Diameter of Inscribed Circle
\[ d = \frac{2bh}{b + \sqrt{b^2 + 4h^2}} \]
\[ d = 2.899 \text{ m} \]

-Ve B.M = \( \frac{Wd}{30} \)
\[ = \frac{1563.89}{100} \text{ Kg-m} \]

Thickness required = \( \sqrt{1563.89/11.09} \)
\[ = 11.88 \text{ cm} \]

Provided Uniform thickness = 20 cm

clear cover = 5 cm

Effective thickness = 15 cm

Min. area of steel required (Astmin) = (0.12/100)x20x100
\[ = 240 \text{ mm}^2 \]

Max. dia. Of bar (f_max = D / 8)
\[ = 25 \text{ mm} \]

Min. area of steel required on each direction (Astmin)
\[ = 120 \text{ mm}^2 \]

Max. allowable spacing (S_max) ar per IS 456
\[ = 45 \text{ mm} \]

Area of Steel Required = 1563.89x100 / 2300x0.9x15
\[ = 5.04 \text{ cm}^2 \]

Dia. Of bar (f)
\[ = 10 \text{ mm} \]

Area of Bar
\[ = 79 \text{ mm}^2 \]

Required Spacing
\[ = 15.67 \text{ cm} \]

Provided Spacing
\[ = 15 \text{ cm} \]

Provided Area
\[ = 526.67 \text{ mm}^2 \]

Provide 10 tor @ 150 c/c bothways at top & bottom as Square mesh

Design of Raft Slab :-

Let the Thickness of Slab is = 200 mm
Grade of Concrete = 25 kg/cm²
Grade of Steel = 415 kg/cm²
Span = 1.501 m
Clear Span = 1.351 m
Shear Force = 3771 kg
Bending Moment = 849.1 Kg-m

Thickness required = \sqrt{849.1/11.09} = 8.75 cm
Provided Uniform thickness = 20 cm
clear cover = 5 cm
Effective thickness = 15 cm

Area of Steel Required = \frac{849.1 \times 100}{2300 \times 0.9 \times 15} = 2.73 cm^2
Dia. Of bar (f) = 10 mm
Area of Bar = 79 mm^2
Required Spacing = 28.94 cm
Provided Spacing = 15 cm
Provided Area = 526.67 mm^2

Provide 10 tor @ 150 c/c bothways at top & bottom as Square mesh

Note: At Corners Provide 5 no's 10 tor at top & bottom

3.30) DESIGN OF RAFT BEAM FB1:

Length of Span = 4.1 m
Let Width of Beam = 300 mm
Let Depth of Beam = 400 mm
Clear Cover to Main Reinforcement d' = 30 mm
Effective Depth d = 350 mm
Triangular Load = 5582.53 \times 4.1^2/6 = 15640.39 kgs
Shear Force = 7820.195 kgs

\begin{align*}
-\text{Ve B.M} &= (5/48)WL - SF \times (X/3) \\
-\text{Ve B.M} &= 5637.06 \quad \text{Kgm} \\
+\text{Ve B.M} &= 0.6x (5/48)WL \\
+\text{Ve B.M} &= 4007.8499 \quad \text{Kgm}
\end{align*}

\begin{align*}
\text{Area of Steel Required} &= \frac{5637.06 \times 100}{2300 \times 0.9 \times 35} = 7.78 \quad \text{cm}^2 \\
\text{Area of Steel Required} &= \frac{4007.8499 \times 100}{2300 \times 0.9 \times 35} = 5.53 \quad \text{cm}^2
\end{align*}
At Top:

<table>
<thead>
<tr>
<th></th>
<th>Straight Bars</th>
<th>Extra Bars</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dia of Bar</td>
<td>16 mm</td>
<td>16 mm</td>
</tr>
<tr>
<td>Area of Bar</td>
<td>201.06 mm²</td>
<td>201.06 mm²</td>
</tr>
<tr>
<td>Required no of Bars</td>
<td>2.75 no's</td>
<td>0.75 no's</td>
</tr>
<tr>
<td>Provide no of Bars</td>
<td>2 no's</td>
<td>1 no's</td>
</tr>
<tr>
<td>Provided Area of Steel</td>
<td>402.12 mm²</td>
<td>201.06 mm²</td>
</tr>
</tbody>
</table>

Provide 2-16tor through+1-16tor extra at Mid span (603.18sqmm)

At Bottom:

<table>
<thead>
<tr>
<th></th>
<th>Straight Bars</th>
<th>Extra Bars</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dia of Bar</td>
<td>16 mm</td>
<td>16 mm</td>
</tr>
<tr>
<td>Area of Bar</td>
<td>201.06 mm²</td>
<td>201.06 mm²</td>
</tr>
<tr>
<td>Required no of Bars</td>
<td>3.87 no's</td>
<td>1.87 no's</td>
</tr>
<tr>
<td>Provide no of Bars</td>
<td>2 no's</td>
<td>2 no's</td>
</tr>
<tr>
<td>Provided Area of Steel</td>
<td>402.12 mm²</td>
<td>402.12 mm²</td>
</tr>
</tbody>
</table>

Provide 2-16tor through+2-16tor extra over supports at bottom (804.24sqmm)

Nominal shear stress (tv) = Vu/bd
= 7820.195/ (30*35) = 7.448 kg/cm²

% of Steel Provided Tension reinforcement (in supp) = 100 x 603.18/(300x350) = 0.57

Design Shear Strength of concrete (tc) (From Table 23 of IS 456-2000)
= 3.24 kg/cm²

Maximum Shear Stress \( \zeta_{c_{\text{max}}} \) = 2.500 kg/cm²

Minimum Shear Reinforcement Required

Net Shear force = 7820.195-(3.24*30*35) = 4418.20 kg

Stirrup Dia = 8 mm

Stirrup legs = 2

Area of Bar = 100.53 mm²

\[ V_{us} = 0.87 \times f_y \times A_{sv} \times d/ S_v \]

Spacing Required =
\[ (0.87 \times 415 \times 100.53 \times 350)/44182 = 287.53 \text{ mm} \]
Spacing Required is Min. of following
Max Spacing of Shear reinforcement  300  mm
0.75*d  =  637  mm
Minimum Shear Reinforcement  =  \( \frac{Asv}{bSv} \geq (0.4/0.87f_y) \)
\( Sv \leq 0.87fyAsv / 0.4b \)
\( Sv \leq = 302.47 \) mm
Provided Spacing is Lesser of above four cases  =  300.00  mm
Say  =  180.00  mm
As per IS 13920, Spacing of Hoops for a distance of 2d from face of support shall be Min of below two conditions
Dia of Hoop Bar  =  8  mm
No of Legs  =  2
2d  =  700  mm
Say  =  700  mm
a.)  \( \frac{d}{4} \)  =  8.75  cm
b.) 8 times of Dia of Longitudinal Bar  =  8 x 12.8  cm
Provided Spacing is Lesser of above two cases  =  8.75  cm
Say  =  8.5  cm
As per IS 13920, Spacing of Hoops is for center portion should not Exceed (d/2)
\( \frac{d}{2} \)  =  17.5  cm
Say  =  17  cm

Stirrups :
Provide Support to 700 mm, 8 dia TOR 2L stirrups @ 85 mm c/c
700 to center, 8 dia TOR 2L stirrups @ 170 mm c/c

DESIGN OF RAFT BEAM FB2 :-
Length of Span  =  5.798  mt
Let Width of Beam  =  300  mm
Let Depth of Beam  =  800  mm
Clear Cover to Main Reinforcement  d'  =  30  mm
Effective Depth  d  =  770  mm
Triangular Load  =  5582.53x5.798^2/12  =  15638.9  kgs
UDL  =  5582.53X1.501  =  8379.38  Kg/m
Shear Force  =  15638.9+8379.38x5.798/2  =  32111.27  kgs
\[-Ve \text{ B.M} = \frac{5}{48}WL + \frac{WL^2}{12} - SF \left(\frac{X}{3}\right)\]
\[-Ve \text{ B.M} = \frac{5}{48}WL + \frac{WL^2}{12} - SF \left(\frac{X}{3}\right) = 28637.74 \text{ Kgm}\]
\[\text{Area of Steel Required} = \frac{28637.74 \times 100}{2300 \times 0.9 \times 77} = 17.97 \text{ cm}^2\]
\[+Ve \text{ B.M} = 0.6x \left(\frac{5}{48}WL + 0.5 \times \frac{WL^2}{12}\right) = 17404.15 \text{ Kgm}\]
\[\text{Area of Steel Required} = \frac{17404.15 \times 100}{2300 \times 0.9 \times 77} = 10.92 \text{ cm}^2\]

**At Top:**

<table>
<thead>
<tr>
<th></th>
<th>Straight Bars</th>
<th>Extra Bars</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dia of Bar</td>
<td>16 mm</td>
<td>20 mm</td>
</tr>
<tr>
<td>Area of Bar</td>
<td>201.06 mm²</td>
<td>314.16 mm²</td>
</tr>
<tr>
<td>Required no of Bars</td>
<td>5.43 no's</td>
<td>1.56 no's</td>
</tr>
<tr>
<td>Provide no of Bars</td>
<td>3 no's</td>
<td>2 no's</td>
</tr>
<tr>
<td>Provided Area of Steel</td>
<td>603.18 mm²</td>
<td>628.32 mm²</td>
</tr>
</tbody>
</table>

Provide 3-16tor through+2-20tor extra at Mid span (1231.5 sqmm)

**At Bottom:**

<table>
<thead>
<tr>
<th></th>
<th>Straight Bars</th>
<th>Extra Bars</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dia of Bar</td>
<td>16 mm</td>
<td>20 mm</td>
</tr>
<tr>
<td>Area of Bar</td>
<td>201.06 mm²</td>
<td>314.16 mm²</td>
</tr>
<tr>
<td>Required no of Bars</td>
<td>8.94 no's</td>
<td>3.80 no's</td>
</tr>
<tr>
<td>Provide no of Bars</td>
<td>3 no's</td>
<td>4 no's</td>
</tr>
<tr>
<td>Provided Area of Steel</td>
<td>603.18 mm²</td>
<td>1256.6 mm²</td>
</tr>
</tbody>
</table>

Provide 3-16tor through+4-20tor extra over supports at bottom (1859.82 sqmm)

Nominal shear stress (tv) =Vu/bd = 32111.27/(30*77) = 13.901 kg/cm²

% of Steel Provided Tension reinforcement (in supp) = 100 x 1859.82/(300x770) = 0.81

Design Shear Strength of concrete (tc) (From Table 23 of IS 456-2000) = 3.696 kg/cm²

Maximum Shear stress $\zeta_{cmax}$ = 1.900 kg/cm²

Minimum Shear Reinforcement Required

Net Shear force = 32111.27 - (3.696*30*77) = 23573.51 kg

Stirrup Dia = 8 mm

Stirrup legs = 2
Area of Bar = 100.53 mm²

\[ V_{us} = 0.87 \times f_y x A_{sv} \times d / S_v \]

Spacing Required =
\[ (0.87 \times 415 \times 100.53 \times 770) / 235735.1 = 118.56 \text{ mm} \]

Spacing Required is Min. of following:

Max Spacing of Shear reinforcement = 300 mm

\[ 0.75 \times d = 578 \text{ mm} \]

Minimum Shear Reinforcement = \[ A_{sv} / b_{sv} \geq (0.4 / 0.87f_y) \]

\[ S_v \leq 0.87f_y A_{sv} / 0.4b \]

\[ S_v \leq 302.47 \text{ mm} \]

Provided Spacing is Lesser of above four cases = 118.56 mm

Say = 115.00 mm

As per IS 13920, Spacing of Hoops for a distance of 2d from face of support shall be Min of below two conditions:

Dia of Hoop Bar = 8 mm

No of Legs = 2

\[ 2d = 1540 \text{ mm} \]

Say = 1550 mm

a.) \[ d/4 = 19.25 \text{ cm} \]

b.) 8 times of Dia of Longitudinal Bar = 8 x \[ d_b = 12.8 \text{ cm} \]

Provided Spacing is Lesser of above two cases = 12.8 cm

Say = 10 cm

As per IS 13920, Spacing of Hoops is for center portion should not Exceed (d/2):

\[ d/2 = 38.5 \text{ cm} \]

Say = 20 cm

Stirrups:

Provide Support to 1550 mm, 8 dia tor 2L stirrups @ 100 mm c/c

1550 to center, 8 dia tor 2L stirrups @ 200 mm c/c

DESIGN OF RAFT BEAM FB5:

Length of Span = 1.351 mt

Let Width of Beam = 300 mm

Let Depth of Beam = 800 mm

Clear Cover to Main Reinforcement d' = 30 mm

Effective Depth d = 770 mm
Triangular Load = 5582.53x1.351^2/4 = 2547.31 kg/m
Shear Force = 2547.31 kgs
-Ve B.M = 2547.31x1.351/2 = 1720.71 kgm
Area of Steel Required = 1720.71x100 / 2300x0.9x77 = 1.08 cm²
Note: Extend Reinforcement of FB4 up to end

DESIGN OF RAFT BEAM FB4 :-
Length of Span = 5.798 mt
Let Width of Beam = 300 mm
Let Depth of Beam = 800 mm
Clear Cover to Main Reinforcement d' = 30 mm
Effective Depth d = 762 mm
From Slab S2 = 5582.53x1.351/2 = 3771 kg/m
Shear Force = 3771x5.798/2 = 10932.129 kgs
-Ve B.M at Ends = 1720.71 Kg-m
+Ve B.M at center = 3771x5.798^2/8 - 1720.71 = 14125.41 Kg-m
Area of Steel Required = 14125.41x100 / 2300x0.9x76.2 = 8.96 cm²
Area of Steel Required = 1720.71x100 / 2300x0.9x76.2 = 1.09 cm²
Min Area of Steel = 0.85 bd / fy

At Top:

  Straight          Extra
  Bars             Bars

  Dia of Bar = 16   Mm   16 mm
  Area of Bar = 201.06 Mm²  201.06 mm²
  Required no of Bars = 4.46 no's   1.46 no's
  Provide no of Bars = 3 no's    2 no's
  Provided Area of Steel = 603.18 Mm²  402.12 mm²

Provide 3-16tor through +2-16tor extra at Mid span (1005.3sqmm)

At Bottom:

  Straight
  Bars

  Dia of Bar = 16 mm
  Area of Bar = 201.06 mm²
  Required no of Bars = 2.33 no's
  Provide no of Bars = 3 no's

57
Provided Area of Steel = 603.18 mm²
Provide 3-16tor through (603.18sqmm)

Nominal shear stress \((t_v) = \frac{V_u}{bd}\)
\[= \frac{10932.129}{(30*76.2)} = 4.782\text{ kg/cm}^2\]

% of Steel Provided Tension reinforcement (in supp) = 100 x 1005.3/(300x762) = 0.44

Design Shear Strength of concrete \((tc)\)
(From Table 23 of IS 456-2000) = 2.908 kg/cm²

Maximum Shear stress \(\zeta_{cmax}\) = 4.000 kg/cm²

Minimum Shear Reinforcement Required

Net Shear force = 10932.129-(2.908*30*76.2) = 4284.44 kg

Stirrup Dia = 8 mm
Stirrup legs = 2

Area of Bar = 100.53 mm²

\[V_{us} = 0.87 \times f_y A_{sv} \times d / S_v\]

Spacing Required =
\[= \frac{(0.87 \times 415 \times 100.53 \times 762)}{42844.4} = 645.54\text{ mm}\]

Spacing Required is Min. of following

Max Spacing of Shear reinforcement = 300 mm

\[0.75 \times d = 572\text{ mm}\]

Minimum Shear Reinforcement = \(\frac{A_{sv}}{bS_v}\) >= \(0.4/0.87 f_y\)
Sv <= \(0.87 f_y A_{sv} / 0.4 b\)

Provided Spacing is Lesser of above four cases = 300.00 mm

Say = 180.00 mm

As per IS 13920, Spacing of Hoops for a distance of 2d from face of support shall be Min of below two conditions

Dia of Hoop Bar = 8 mm
No of Legs = 2
2d = 1524 mm
Say = 1550 mm

a.) \(d/4\)
b.) 8 times of Dia of Longitudinal Bar = 8 x \(d_b\) = 12.8 cm

Provided Spacing is Lesser of above two cases = 12.8 cm

Say = 12 cm

As per IS 13920, Spacing of Hoops is for center portion should not Exceed (d/2)

\[d/2 = 38.1\text{ cm}\]
Say = 20 cm

Stirrups:
Provide Support to 1550 mm, 8 dia tor 2L stirrups @ 120 mm c/c
1550 to center, 8 dia tor 2L stirrups @ 200 mm c/c

DESIGN OF RAFT BEAM FB3 :-
Length of Span = 1.351 mt
Let Width of Beam = 300 mm
Let Depth of Beam = 800 mm
Clear Cover to Main Reinforcement d' = 30 mm
Effective Depth d = 770 mm
From FB4 & FB5 = 10932.129 + 2547.31 = 13479.439 Kg
Triangular load From FS3 = 2547.31 Kg
Shear Force = 13479.439 + 2547.31 = 16026.749 kgs
Max B.M = 13479.439 x 1.351 + 2547.31 x 1.351 / 2 = 19931.43 kgm
Area of Steel Required = 19931.43 x 100 / 2300 x 0.9 x 77 = 12.5 cm²
Note: Extend Reinforcement of FB2 up to end

(3.31) Appendix

From STADD Pro, The Results are as follows:
For Stiffness Calculation:

<table>
<thead>
<tr>
<th>Node</th>
<th>L/C</th>
<th>X-Trans mm</th>
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<tr>
<td>531</td>
<td>1</td>
<td>4.282</td>
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<tr>
<td>2</td>
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<tr>
<td>6</td>
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<tr>
<td>8</td>
<td>1</td>
<td>4.26</td>
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</table>
For Floor Ring Design:

<table>
<thead>
<tr>
<th>Max</th>
<th>Beam</th>
<th>L/C</th>
<th>Node</th>
<th>Fy kN</th>
<th>Mx kNm</th>
<th>Mz kNm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fy</td>
<td>222</td>
<td>DL+WATERLOAD+LL 4</td>
<td>10</td>
<td>34.804</td>
<td>-3.42</td>
<td>11.261</td>
</tr>
<tr>
<td>Min</td>
<td>230</td>
<td>DL+WATERLOAD+LL 4</td>
<td>19</td>
<td>34.805</td>
<td>3.415</td>
<td>11.261</td>
</tr>
<tr>
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<td>221</td>
<td>DL+WATERLOAD+LL 4</td>
<td>9</td>
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<td>34.804</td>
<td>-3.42</td>
<td>11.261</td>
</tr>
<tr>
<td>Mx</td>
<td>230</td>
<td>DL+WATERLOAD+LL 4</td>
<td>19</td>
<td>34.805</td>
<td>3.415</td>
<td>11.261</td>
</tr>
<tr>
<td>Min</td>
<td>230</td>
<td>DL+WATERLOAD+LL 4</td>
<td>18</td>
<td>28.194</td>
<td>3.415</td>
<td>11.251</td>
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</tbody>
</table>
For Outer Braces Design:

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<th>Beam</th>
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<th>L/C</th>
<th>Fy kN</th>
<th>Mx kNm</th>
<th>Mz kNm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Max</td>
<td>943</td>
<td>1.5(DL+WL+WL-Z)</td>
<td>517</td>
<td>16.68</td>
<td>28.00</td>
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<tr>
<td>Min</td>
<td>944</td>
<td>1.5(DL+WL+WL+X)</td>
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<td>16.68</td>
<td>28.00</td>
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<td>Max</td>
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<td>1.5(DL+WL+WL+X)</td>
<td>518</td>
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<td>-11.74</td>
</tr>
<tr>
<td>Min</td>
<td>943</td>
<td>1.5(DL+WL+WL+Z)</td>
<td>517</td>
<td>2.34</td>
<td>-11.74</td>
</tr>
<tr>
<td>Max</td>
<td>944</td>
<td>1.5(DL+WL+WL+Z)</td>
<td>518</td>
<td>16.68</td>
<td>29.85</td>
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For Inner Braces Design:

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<th>Beam</th>
<th>Node</th>
<th>L/C</th>
<th>Fy kN</th>
<th>Mx kNm</th>
<th>Mz kNm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Max</td>
<td>949</td>
<td>1.5(DL+WL+WL+X)</td>
<td>518</td>
<td>24.21</td>
<td>0</td>
</tr>
<tr>
<td>Min</td>
<td>950</td>
<td>1.5(DL+WL+WL+X)</td>
<td>516</td>
<td>-24.21</td>
<td>0</td>
</tr>
<tr>
<td>Max</td>
<td>950</td>
<td>1.5(DL+WL+WL-Z)</td>
<td>520</td>
<td>6.268</td>
<td>1.044</td>
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<tr>
<td>Min</td>
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<td>1.5(DL+WL+WL-X)</td>
<td>520</td>
<td>6.268</td>
<td>-1.044</td>
</tr>
<tr>
<td>Max</td>
<td>949</td>
<td>1.5(DL+WL+WL+X)</td>
<td>518</td>
<td>24.21</td>
<td>0</td>
</tr>
<tr>
<td>Min</td>
<td>950</td>
<td>1.5(DL+WL+WL-X)</td>
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<td>10.757</td>
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### Columns Design:

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<tr>
<th></th>
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<th>Fx kN</th>
<th>My kNm</th>
<th>Mz kNm</th>
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<td>19</td>
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<td>-0.001</td>
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<td>Min</td>
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<td>898.573</td>
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### Column inside Container Design:

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<th>Beam</th>
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<th>Node</th>
<th>Fx kN</th>
<th>My kNm</th>
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</tr>
</thead>
<tbody>
<tr>
<td>Max</td>
<td>Fx</td>
<td>959 DL+WATERLOAD+LL</td>
<td>253</td>
<td>76.318</td>
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<tr>
<td>Min</td>
<td>Fx</td>
<td>446 DL+WATERLOAD+LL</td>
<td>446</td>
<td>73.608</td>
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<td>Max</td>
<td>My</td>
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<td>253</td>
<td>76.318</td>
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<tr>
<td>Min</td>
<td>My</td>
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<td>73.608</td>
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<td>Max</td>
<td>Mz</td>
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<td>Min</td>
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</tbody>
</table>
DIAGRAMS:

FOOTING

Fig no 4.1: Shear bending

Fig no 4.2: Deflection

COLUMN

Fig no 4.3: Shear bending
Fig no4.4: deflection

Brace beam

Fig no4.5: Shear bending

Fig no4.6: deflection
Fig no4.7: our service reservoir
STATIC LOAD/REACTION/EQUILIBRIUM SUMMARY FOR
CASE NO. 7
LOADTYPE SEISMIC TITLE EQ-X

CENTER OF FORCE BASED ON X FORCES ONLY (METE).
( FORCES IN NON-GLOBAL DIRECTIONS WILL
INVALIDATE RESULTS)

\[ X = 0.313209544E-16 \]
\[ Y = 0.272499992E+01 \]
\[ Z = 0.215331561E-15 \]

***TOTAL APPLIED LOAD ( KN METE ) SUMMARY
(LOADING 7)
SUMMATION FORCE-X = -51.26
SUMMATION FORCE-Y = 0.00
SUMMATION FORCE-Z = 0.00

SUMMATION OF MOMENTS AROUND THE ORIGIN-
MX= 0.00 MY= 0.00 MZ= 139.69

TOTAL REACTION LOAD ( KN METE ) SUMMARY (LOADING 7)
SUMMATION FORCE-X = 51.26
SUMMATION FORCE-Y = 0.00
SUMMATION FORCE-Z = 0.00

SUMMATION OF MOMENTS AROUND THE ORIGIN-
MX= 0.00 MY= 0.00 MZ= -139.69

MAXIMUM DISPLACEMENTS ( CM /RADIANS) (LOADING 7)
MAXIMUMS AT NODE
X = -1.76057E+00 226
Y = 2.95134E-02 235
Z = -3.18099E-04 4
RX= 2.58050E-05 340
RY= -1.17629E-05 56
RZ= 1.26520E-03 517
EXTERNAL AND INTERNAL JOINT LOAD SUMMARY (KN METE)-

JT     EXT FX/     EXT FY/     EXT FZ/     EXT MX/     EXT MY/     EXT MZ/
INT FX     INT FY     INT FZ     INT MX     INT MY     INT MZ

SUPPORT=1

526    0.00    0.00    0.00    0.00    0.00    0.00       0.00    0.00
       -11.73   -87.13   0.00   0.00   0.00   23.80   111111

527    0.00    0.00    0.00    0.00    0.00    0.00        0.00    0.00
       -7.73    0.00    0.00   0.00   0.00   20.06   111111

528    0.00    0.00    0.00    0.00    0.00    0.00        0.00    0.00
       -11.73   87.13    0.00   0.00   0.00   23.80   111111

529    0.00    0.00    0.00    0.00    0.00    0.00        0.00    0.00
       -7.73    0.00    0.00   0.00   0.00   20.06   111111

530    0.00    0.00    0.00    0.00    0.00    0.00        0.00    0.00
       -12.36   0.00    0.00   0.00   0.00   24.39   111111

FOR LOADING -   8
APPLIED JOINT EQUIVALENT LOADS
JOINT FORCE-X     FORCE-Y     FORCE-Z     MOM-X     MOM-Y     MOM-Z

109 0.00000E+00 0.00000E+00 1.42398E+00 0.00000E+00
       0.00000E+00 0.00000E+00

110 0.00000E+00 0.00000E+00 1.42398E+00 0.00000E+00
       0.00000E+00 0.00000E+00

111 0.00000E+00 0.00000E+00 1.42398E+00 0.00000E+00
       0.00000E+00 0.00000E+00

112 0.00000E+00 0.00000E+00 1.42398E+00 0.00000E+00
       0.00000E+00 0.00000E+00

113 0.00000E+00 0.00000E+00 1.42398E+00 0.00000E+00
       0.00000E+00 0.00000E+00

114 0.00000E+00 0.00000E+00 1.42398E+00 0.00000E+00
       0.00000E+00 0.00000E+00

115 0.00000E+00 0.00000E+00 1.42398E+00 0.00000E+00
       0.00000E+00 0.00000E+00

116 0.00000E+00 0.00000E+00 1.42398E+00 0.00000E+00
       0.00000E+00 0.00000E+00
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MEMBER LOAD
221 TO 256 UNI GY -3
LOAD 10 LOADTYPE Wind TITLE WL+X
ELEMENT LOAD
1 TO 9 28 TO 45 64 TO 81 100 TO 117 136 TO 153 172 TO 189 208 TO 216 PR -1.05
MEMBER LOAD
915 916 918 920 921 923 960 961 963 973 974 976 UNI GX 0.42
935 939 943 948 965 970 UNI GX 0.45
LOAD 11 LOADTYPE Wind TITLE WL-X
ELEMENT LOAD
10 TO 27 46 TO 63 82 TO 99 118 TO 135 154 TO 171 190 TO 207 PR -1.05
MEMBER LOAD
916 TO 918 921 TO 923 961 TO 973 974 976 UNI GX -0.42
936 940 944 945 966 967 UNI GX -0.45
LOAD 12 LOADTYPE Wind TITLE WL+Z
ELEMENT LOAD
1 TO 18 37 TO 54 73 TO 90 109 TO 126 145 TO 162 181 TO 198 PR -1.05
MEMBER LOAD
915 TO 917 920 TO 922 960 TO 962 973 TO 975 UNI GZ -0.42
935 936 943 944 965 966 UNI GZ -0.45
LOAD 13 LOADTYPE Wind TITLE WL-Z
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NUMBER OF JOINTS/MEMBER+ELEMENTS/SUPPORTS = 530/665/5

SOLVER USED IS THE IN-CORE ADVANCED SOLVER

TOTAL PRIMARY LOAD CASES = 11, TOTAL DEGREES OF FREEDOM = 3150

LOADING 6 LOADTYPE SEISMIC TITLE EQ+
LOADING 7 LOADTYPE SEISMIC TITLE EQ-X
LOADING 8 LOADTYPE SEISMIC TITLE EQ+Z
LOADING 9 LOADTYPE SEISMIC TITLE EQ-Z

LOADING 1 LOADTYPE DEAD TITLE DL

SELFWEIGHT Y -1.000

ACTUAL WEIGHT OF THE STRUCTURE = 1574.849 KN

ELEMENT LOAD (UNITS ARE KN METE)

ELEMENT PRESSURE

694 -1.000000
695 -1.000000
696 -1.000000
697 -1.000000
698 -1.000000
699 -1.000000
700 -1.000000
FOR LOADING -  1
APPLIED JOINT EQUIVALENT LOADS
JOINT  FORCE-X  FORCE-Y  FORCE-Z  MOM-X  MOM-Y  MOM-Z

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STAAD SPACE -- PAGE NO. 55

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**JOINT DISPLACEMENT (CM  RADIANS)  STRUCTURE TYPE**

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Various softwares used for the design of tank

http://softwaretopic.informer.com/software-for-water-tank-design://
**TSOLexpress:** Simulation program for design and calculation of solar thermal systems.

**Daikin Altherma Simulator:** A static calculation tool for dimensioning Daikin Altherma heat pump systems.

**GeoDesigner:** GeoDesigner is the most complete residential earth loop design.

**RainTank2:** RainTank 2 is an interactive software tool which assists rural communities.

**Storage Tank Design Software:**
- [www.techtarget.com/Data-Backup](http://www.techtarget.com/Data-Backup)  

**Wadiso:** Wadiso is an application built for the analysis of water distribution systems.

**HYDROFLO:** Hydro-flow is a powerful software tool that assists piping system designers.

**TriTank650:** TRI*TANK650 software for the design and rating of welded steel oil storage tanks.
CONCLUSIONS

Elevated Service Reservoir of 150 K.L. capacity with 12 m staging has been designed considering M30 concrete for the Container and M20 for staging. However, M25 concrete is used for staging.

Detailed structural drawings have been prepared.

Abstract estimate is prepared for the elevated service reservoir including pipe connections considering the current standard schedule of rates, issued by government of Andhra Pradesh. The estimate works out to Rs:27,17,550.
REFERENCES


2. SP :16  Design aids.


4. IS :875-1987

5. IS:1893 -2002

6. IS:13920 -1993

7. IS:11682 -1985


9. Reinforced concreter design by Ashok Kumar Jain and Arun Kumar Jain.

10. Reinforced concrete design by N. Krishna Raju and R.N. Pranesh