ANALYSIS AND DESIGN OF TRESTLE TYPE WATER TOWER WITH DIAGONAL BRACING USING FEM AND DRAFT CODE PROVISIONS

Major Project

Submitted in partial fulfillment of the requirements

For the degree of

Master of Technology in Civil Engineering
(Computer Aided Structural Analysis & Design)

By

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CERTIFICATE

This is to certify that the Major Project entitled "**Analysis and Design of Trestle type Water Tower with Diagonal bracing using FEM and Draft Code Provisions**" submitted by **Ms. Priyadarshini Vijay Sonar (04MCL018)**, towards the partial fulfillment of the requirements for the degree of **Master of Technology in Civil Engineering** (**Computer Aided Structural Analysis and Design**) of Nirma University of Science and Technology, Ahmedabad is the record of work carried out by her under my supervision and guidance. In my opinion, the submitted work has reached a level required for being accepted for examination. The results embodied in this major project, to the best of my knowledge, haven't been submitted to any other university or institution for award of any degree or diploma.

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ABSTRACT

Water Tower is a special liquid retaining structure which is designed as crack free structure using Working Stress Method. There are basic discrepancies observed where the design of container is made with Working Stress Method and the supporting structure is designed by Limit Stress Method. As all engineering structures undergo Earthquakes with their distinct behaviour, Water tower also behaves quite differently than ordinary building structures.

In earlier codes provisions were given considering Water tank as Single Degree of Freedom system but in Draft Code impulsive and convective water masses of Container water are considered.

The Bhuj Earthquake occurred on 26th January 2001, most of the Water tanks built in Gujarat state which were designed by *IS 1893 (1984*) Code have survived the earthquake. The functioning of Water tanks during recent earthquake is at variance with conservative design approach adopted by provisions of Proposed Draft Code. There are varying opinions regarding provisions for elevated tanks seismic analysis in Proposed Draft Code *'IS: 1893 (2002) (part II)'*. Comments regarding Proposed Draft are also mentioned here. In this study comparison between design forces for Water tanks obtained by the provisions of Old Code and those of the Proposed Drat Code is carried out.

The dissertation also deals with the concept where we can achieve economical water tank staging by introducing diagonal bracings thus reducing the design column and horizontal bracing moments considerably and providing lighter sections. Considerable increase in cost results due to adoption Proposed Code, for Water tanks resting on column supports staging or shaft supports.

Generally in concrete design, Diagonal bracings are not preferred due to congestion of reinforcement at joints and difficulties in detailing and construction. If these problems are not addressed adequately, there is a potential danger which may lead to the failure of water tank supporting staging. In present dissertation steel diagonal Bracings are considered and the forces in columns and bracings due to conventional peripheral bracing system with and without Diagonal Bracings are compared.

The lateral load carrying capacity of structure is based on its stiffness and its load distribution mechanism. The trestle is found to be flexible compared to the

shaft and thus subjected with lower lateral forces. For the stiffness calculation STAAD Pro software has been used which gives more accurate stiffness as it incorporates 3D behaviour of frame staging.

The provisions regarding Response Reduction Factors adopted for Shaft supports in Proposed Draft Code appears to be based on the thought that shafts are more vulnerable than the Trestle support. Although in Gujarat most of the tanks are Shaft supported and they have performed well during recent Earthquake.

The seismic analysis is carried out manually using old code provisions and draft code provisions. The Dynamic analysis of Case study problem is done using STAAD Pro.

First chapter includes the introduction part.

Second chapter includes the literature survey where the literature available for Water tanks regarding Modeling, design, Draft Code Provisions, Various Bracing System is abstracted. Scope of work is also included in the same chapter.

Third chapter includes comparison of Proposed Draft code with *IS 1893-1984* and the discussion for provisions of draft code are reviewed.

Fourth chapter consists of Analysis and design of Water tank supported on trestle.

Fifth chapter contains the dynamic analysis of water tank carried out using STAAD Pro software which also consists of the time period variation of water tank for various modes of vibrations for various decreasing order capacities.

Sixth chapter includes estimation and cost analysis for trestles with and without Diagonal bracings additional to the Peripheral bracings for improving effectiveness of supporting structure with graphical representations.

Seventh chapter concludes with graphical representations and discusses future scope of the project.

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ABBREVIATION NOTATION AND NOMENCLATURE

Notation	Meaning
γ	Density
(Ah)c	Design horizontal seismic coefficient for convective mode
(Ah)i	Design horizontal seismic coefficient for impulsive mode
Δbt	Drift
Ac	Concrete area
Ah	Horizontal seismic coefficient
As	Area of steel reinforcement
Cc	Coefficient of convective mode
Cp	Coefficients for bending moment
D	Diameter o water container
E	Joint efficiency factor
Eb	Modulus of elasticity of beam
Ec	Modulus of elasticity of Column
Es	Modulus of Elasticity
F	Allowable unit stress
F	Known force applied at CG of container
G	Specific capacity
g	Acceleration due to Gravity
H	Depth from top of tank
h	Height of panel
h	Panel height
h'	Free Board
h1	Rise of Top dome of water tank
h2	Rise of bottom dome of water tank
hcg	Dist of container cg from ground level in Ground supported tanks
hcg	Height of center of gravity of empty container measured from base of staging
hi*	Height of impulsive mass above bottom of tank wall (considering base pressure
hs	Structural height of staging, measured from top of footing of staging to the bottom of tank wall
I	Importance factor
Ib	Moment of Inertia of beam
Ic	Moment of inertia of Column
K	Stiffness of Trestle
Kb	Stiffness of Beam
Kbg	Stiffness of bracings with equal panel height at top and bottom

Kc	Stiffness of Column
Ks	Stiffness of staging
L	Length
L	Span of bracing girder
m	Modular ratio
mb	Mass of base slab/plate
mi	Impulsive mass of water
ms	Equivalent mass
mt	Mass of roof slab
mw	Mass of tank wall
Nc	No. of Column in periphery of staging
NÞ	Meriodonal thrust
P	Allowable soil pressure
q	Design Load
Q	Required capacity
q1	Uniform radial pressure
Qd	Volume Occupied
r	Radius of the container
R	Response reduction factor
R_1	Radius of Bottom dome
S	Surface Area
Sa/g	Average response acceleration coefficient
T	Hoop tension
T	Shell plate thickness
tb	Thickness of base slab/plate.
T_c	Time period in convective mode of container
Ti	Time period in impulsive mode of container
V	Base shear
V	Shear force
Vb	Base shear
W1	Total domed weight
Z	Zone factor
Δ	Displacement resulted
θ	Semicircle angle
σcb	Allowable Stress in compression bending
σt	Allowable stress in tension

1. INTRODUCTION

1.1 GENERAL

The storage reservoirs are part of the essential urban amenities. For the drinking purposes and industrial requirement Water is necessarily stored in large capacity reservoirs. They are special civil engineering structures in which, containers are designed as crack free, in order to ensure them to be leak proof. Also in these structures, the solid liquid interaction comes into picture. Storage containers are generally designed by Working stress method and the staging, i.e. supporting structure for storage containers are designed with Limit state method. Thus the different methodologies create some natural discrepancy in the Water tanks, especially if the entire structure is designed as one unit.

Due to Inverted Pendulum geometry, the design of Overhead Water tank is governed by Lateral Forces that may be Earthquake Forces or Wind Forces. In case of India, Water tanks are designed as per *IS: 3370 (Part I to V), IS: 11682*, and *IS: 1893*. After the event of Bhuj Earthquake of M 7.7 on Richter scale, the state government of Gujarat sponsored GSDMA (Gujarat Disaster Management Authority) project, which examined the necessity for revision of earthquake codes with the help of IITK (INDIAN INSTITUTE OF TECHNOLOGY, KANPUR), which suggests increase in earthquake forces as per Proposed Code.

It is not clear to what extent this sponsored assignment studied the performance of existing Water tanks of Gujarat during the past earthquake. The existing tanks were designed as per IS: 1893-1984. They are large in numbers of various capacities; constructed on different type of soils condition and that sense are truly representative of all possible situations.

The Draft Code suggests for Elevated Water tanks the response reduction factors of half the value then that for the building frame. Adoption of Draft code will not only result in increase of cost but will also raise serious doubts regarding the design adequacies of existing Water tanks which have survived earthquake and are functioning well.

1.2 TYPES OF WATER TANKS

Depending upon support type, Water tanks are classified as Ground supported, partially or Fully Under ground Water tanks. If storage reservoir is situated above ground by some supporting structure like Trestle or Shaft then the Water tank is known as Overhead Water tank.

1.3 MATERIALS OF WATER TANKS

In general Water tanks are constructed using Reinforced Concrete and Steel. Uses of advanced materials like Prestressed Concrete and Polymerized concrete, Ferrocement for constructing tank can also be made.

The various forms of Water tanks components that are generally accepted are; for

* **Containers:** Cylindrical, Rectangular, Square, Conical, elliptical
* **Staging:** Trestle supported and Shaft supported
* **Bottom:** Depending on Function, Construction, Maintenance costs & Aesthetics

For Concrete towers i) Curved shaped bottom, ii) Flat bottom

For Steel tanks i) Spherical or Dome shaped bottom.

1.4 WATER TANKS STAGING

It is the supporting structure for container, which transfers loads from container to footing below. Staging types are Shaft and Trestle.

FIGURE 1.1 TYPES OF TANK STAGING

1.4.1 Shaft type staging [1]

Shaft is hollow column, but when its thickness is very small compared to its diameter, it assumes a special identity as membrane structure. Sliding formwork speeds up the construction of shaft. But when the construction is done with conventional formwork then special care for the verticality, circularity and uniformity of thickness is to be taken. The variation in geometry results in additional stresses in shaft. It attracts more wind and seismic forces due to its higher obstruction area and higher stiffness. The construction joints of shaft are weak points thus after earthquake some damage is observed to these construction joints. [1]

1.4.2 Trestle type staging [1]

When a group of columns, say 4, 6, 8 etc. in nos. are so on used to support Water tank container then it is known as Trestle staging. As the individual column in a group of columns becomes slender, in order to make columns safe against buckling additional ties in the form of bracings are provided. The columns are subjected to direct and bending stresses, whereas bracings take reversible bending due to lateral forces like Earthquake and Wind.

As leakage through container joints results in corrosion of reinforcing steel, this at times weakens the joints of trestle resulting in risk of instability. Thus inspection and maintenance of beam column joints is very necessary. Also during design proper ductility should be provided to joints to avoid brittle failure of such joints. The major failure of trestles occurs due to deficient detailing and construction of beam column joints, poorly designed staging and lack of maintenance.

TABLE 1.1 FUNCTIONS OF TRESTLE ELEMENTS

Sr. No.	Member	Purpose	Load subjected
1	Column	Transfer vertical loads	Direct & Bending stresses
2	Bracings	Reduction in effective length of column and participate in resisting lateral forces	Reversible Bending and Axial stresses
3	Foundation	Transfer loads to earth below	Bending stresses and shear

1.5 INNOVATIVE WATER TANKS STAGING

Some of the innovative Water tanks are used as monuments, pent houses, shopping malls, and structure to fascinate people. Also there are examples where the supporting structures are also modified so as to improve overall lateral stiffness of Water tank. Following diagrams show few of such innovative structures.

FIGURE 1.2 INNOVATIVE WATER TANKS

1.6 OBJECTIVE OF STUDY

Regular updating and revisions of codes are necessary for improved design and construction practices in the country. The process of revision must reflect the refinement attained in the design process due to further research and further development. It must also reflect the experience gained during the interim period between the revisions. Greater participation of the code users will result in more balanced outcome. However it is often seen that the codes fall short meeting the expectations of the users.

The structures designed as per *IS: 1893-2000* and constructed well have served the purpose well. The existing Water tanks in Gujarat appear to have survived well the disastrous Bhuj Earthquake M 7.7 Richter scale. Even though the Proposed Draft code *IS: 1893-2000 (Part II)* have suggested lower Response Reduction factors resulting in increase in lateral loads of 2 to 3 times in case of Shaft supported tanks and increase in case of Trestle supported tanks.

In this context it is considered of interest to study the provision of Proposed Draft code.

Thus it is necessary to study the Proposed Draft code and existing codes which mainly includes *IS: 1893* code for design of earthquake resisting structures and give comparative conclusions between them. The main objective of the dissertation is to study the analytical and designing clauses of Proposed Draft code *IS: 1893-2002 (Part II)* and compare them with the existing code of IS*: 1893-2000* and *IS: 1893-1984.*

To study the Trestle supported Water tank and to acquaint with the effects of Steel Diagonal Bracings on column moments, and verify whether Diagonal Bracings are more effective than traditional peripheral ties in reducing column moments.

1.7 OUTLINE OF DISSERTATION

First chapter deals with the introduction of Trestle type staging supported Water tank.

Second chapter gives the summary of the various research papers referred for the dissertation work, which includes literature related with modeling, analysis and design of elevated Water tanks, and the alternative bracings systems for improving lateral stiffness of Water tank.

Third chapter includes review of Proposed Draft code and discussion regarding some issues that are of interest. The chapter consists of general design clauses for the analysis and design of trestle supported Water tanks mentioned in Proposed Draft Code.

Fourth chapter includes Analysis and Design of case study problem according to *IS: 1893 (1984)* and Proposed Draft Code for liquid retaining structures and their comparison in terms of base moments and base shear.

Fifth chapter is related with dynamic analysis of considered case study problem using STAAD Pro and Sixth chapter deals with the estimation of Trestle supported tank with Peripheral Bracings and with Diagonal Bracings.

The Seventh chapter consists of graphical representation of all the work related with detailed study of effects of Steel Diagonal bracings in case of column moments and their comparison with traditional peripheral braced staging with respect to column and bracing moment. It also includes conclusions and future scope of present dissertation.

2. LITERATURE SURVEY

The Bhuj, Gujarat earthquake (Bhuj earthquake) of January 26, 2001 was a major event both in terms of its seismological characteristics and in terms of its economic, life loss and social consequences. After the large damages took place due to Bhuj Earthquake, the State government started disaster management project through GSDMA (Gujarat State Disaster Management Authority). One step of managing disasters is to make civil engineering structures more compatible with the requirement offered by large damaging natural hazards like Earthquakes, Floods and Cyclones. For that the codes for designing structures are being revised. *IS: 1893-2002* is being revised in five parts. Second part pertains Liquid Retaining Structures such as Water tanks. In present Project the Literature is studied as per following topics related with Water tanks.

- 2.1 Analysis
- 2.2 Design
- 2.3 Modeling
- 2.4 Staging configuration with alternate Bracing Systems
- 2.5 Column Stiffness and Lateral Load Distribution on Staging of Water tank
- 2.6 Effects of Earthquakes on Liquid Retaining Structures
- 2.7 Effects of Winds on Liquid Retaining Structures
- 2.8 Soil structure interaction

2.1 ANALYSIS

G. Tripathi & et al, simulated finite element model of Water tank and carried out seismic analysis of the same model built with four nodded plate and shell elements, taking care of appropriate distribution of Water mass so that hydrodynamic forces can be easily accounted. The author and others suggest alternative technique to simulate Water tank, to deal with seismic analysis problem. In which they considered mass lumping method which is less calculative and easy to understand than the approach used conventionally by Housner (1957) and *IS: 1893*, who considered Water to be filled inside Water container. Here as per author lumped mass technique, which requires less computational efforts, is better representation of hydrodynamic forces than that of conventional two mass representations. [02]

V. Verma & et al suggests another 'Lumped mass beam model' technique which is less computational, accurate method for the modeling of liquid retaining structure.

This method is based on 'Strain Energy Equivalence', in combination with 3D model found to be simpler, economic and gives conservative results. In this paper the formulation to calculate stiffness of beam model is given also 'Method of Energy Equivalence', 'Method of Averages' are discussed. [03]

2.2 DESIGN

For design of Water tank the Books by renowned authors are available which mainly includes **Jai Krishna and Jain** [04]**, P. Dayaratnam** [05]**, and Krishnaraju** [06].

2.3 MODELING

In the Proposed Draft Code Water tank model is simulated assuming dual nature behaviour of Water in elevated Water tank container. G.W. Housner first suggests such two mass representations. R. Shepherd also briefly discusses it in his technical paper. Two mass representations includes first mass which retain itself along with container produces hydrostatic pressures on container walls is known as 'Impulsive mass' and the other which sloshes resulting in wave pressure is known as 'Convective mass'.

In computer programmes to simulate model of Water tank with idealized discrete mass system limited number of degree of freedom are used in prototype. Thus for simulation of Water tank model two mass representation which is easy and useful.

As per **R Shepherd**, Earlier studies show one degree of freedom system for Water tank gives satisfactory results but in application of computer simulation and numerical integration techniques, for getting accurate earthquake prediction it is necessary to have valid model of the system. In two mass representations mass m_1 is the convective mass and m_0 is the impulsive mass, the effective spring stiffness K_1 may be determined by considering geometric properties of tank and tower stiffness K_0 is derived by standard structural analysis techniques.

Author used computer numerical integration techniques and compared the results obtained with the results of dynamic analysis carried on the prestressed concrete tubular tower.

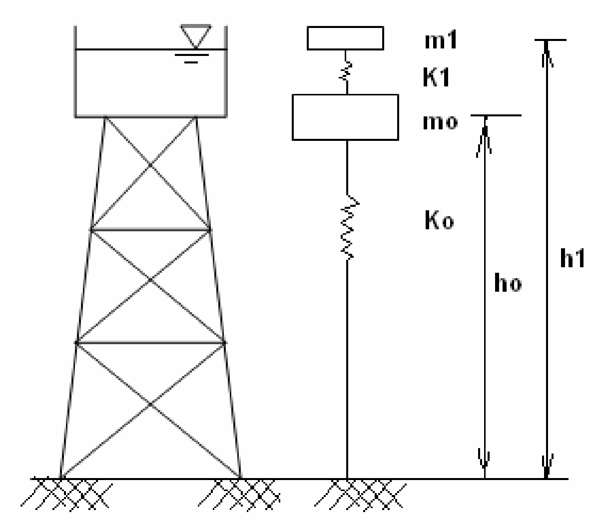


FIGURE 2.1 WATER TANK AND TWO MASS REPRESENTATIONS

The considered tower model supports a cylindrical container. Impulsive movements of container walls, because of inertia forces give rise to impulsive pressures and these pressures developed are directly proportional to acceleration of container walls. Convective pressures are produced by the oscillations of fluid and are consequences of the impulsive pressures.

For a liquid storage tank, assuming m1 and m0 masses can use the equivalent dynamic system. Above figure shows an oscillating liquid surface and next one shows mass equivalent to those produced by the liquid.

The mass m_1 exerts a maximum horizontal force directly proportional to the maximum acceleration of the tank bottom, at a height h_1 and thus contributes to the overturning moment in the tank. The mass m_0, acting as a solid oscillating mass flexibly connected to the walls and located at a height h_0, also contributes to the overturning moment acting on the tank. [07]

2.4 STAGING CONFIGURATION WITH ALTERNATE BRACING SYSTEMS

R.K. Ingle & S. S. Kulkarni describes the method by which we can reduce lateral drift of Water tower by studying proper design aspects. Design of staging is carried out for the gravity and lateral loads like wind, earthquake forces. Author states it is necessary to consider additional forces due to P – Delta effect for the stability of column and stability of structure as a whole.

The Water containers are designed as per uncracked theory whereas staging columns are designed by limit state method, which leads to the natural discrepancy between two components of same structure. Author suggests P - Delta effects should be included in computation of story shears, story drift and member end forces when stability index exceeds value of 0 .1 where,

Stability index = (Story drift X Vertical load)/ (Height X Horizontal shear)

Due to bending of columns and beams, and due to axial deformations of columns, drift occurs in framed structures. As height to width ratio increases, effect of the column axial deformations is significant. The major portion, nearly 20 – 70 % of drift in case of rigid frames is caused due to end rotations of beams and columns concept known as 'Bent action'.

As per Naeim's (1986) suggestion bent action displacements can be calculated as follows

$$\Delta_{bt} = \sum (V_i) h_i^2 \left(\frac{\frac{1}{\sum (K_b)_i} \frac{1}{\sum (K_c)_i}}{(12E)} \right)$$

Where,

K_b is the stiffness of beam,

K_c is stiffness of column,

E is modulus of elasticity,

V is the shear force,

h is the height of panel

This expression gives reasonable sizes of columns and braces. Important to note that for tall Water structures designed to satisfy drift limitations braces required are more than the brace requirement observed in design against wind and seismic loading. [08]

Frames are laterally stiffened to control drift either by following ways,

- Increasing size of columns or braces or both
- Rigid brace introduction
- Increasing number of panels of braces
- Introduction of plan bracing

In above mentioned alternatives, introduction of rigid braces or plan braces is economical alternative. [08]

- As per the conclusions derived by **R. K. Ingle**, intermediate bracing sizes can be increased for controlling drift up to stability index is less than 0.1 but at the same time by selecting such economical alternative will go against the 'weak beam – strong column system'. This fact should be taken into account,

- Increase in size of central brace and second lowest brace gives better results,
- Increasing size of braces increases stiffness of Water tank at the same time reducing moments in columns and braces,
- Plan bracing doesn't have significant effect on reduction of drift so plan bracings are not recommended.

In Another paper by **R. K. Ingle** discusses proportioning of columns for Water tank supporting structures.

As per *IS: 456* it is necessary to include effects of final deformations i.e. P – Delta effect in buildings design forces. Shape of the column, placing like tangentially or radially, plays an important role in reducing drift leading to economical design. [09]

Author studied above-mentioned effects for Water tower design and for modelling made following assumptions:

- Considering the Water tower as space frame, also being connected by container walls the top braces are considered to be axially rigid, carries out static analysis of Water tower skeleton.
- The interaction between soil and structure on account of great stiffness of foundation is disregarded.
- By provision of tangential arrangement of columns twisting moments in braces decreases considerably.

The conclusions made on going through series of experiments on Water tower having various configurations. [13]

- As the number of panels increases, percent increase in stiffness goes down for tangential disposition of columns in comparison with square columns,
- Percent increase in stiffness of the structure goes up as the number of columns is increased along the periphery. This means that the structure tends to become a cylinder of minimum thickness,
- Tangential arrangement of columns shows less time period, which means more acceleration from the response spectrum of *IS: 1893*. However the increase in forces is less in comparison with the increase in stiffness,
- Radial arrangement of columns shows decrease in stiffness and increase in stability index. This arrangement is not recommended.

Tangential arrangement of rectangular columns can be used for tank supporting structure, which will reduce secondary moments without increase in cost of the structure.

Raymond H. Plaut & Rac Hak Yoo [10] in his paper writes regarding elastic response of columns after sudden loss of bracing.

Author discusses the elastic response of braced columns after sudden loss of bracings due to explosion or any other accidents, which make column to undergo dynamic loading. These loadings result into significant amount of oscillations leading to collapse.

Effects of axial forces, bracings position, and bracing stiffness also taken into account which gives some important conclusions like

- As bracing stiffness increases, optimal bracing location corresponding to critical maximum load, moves towards base of column.
- Stiffer brace tends to store more energy, thus when such brace is not near the ends, maximum response increases as the bracing stiffness K increases.
- When such brace is located near the ends of the columns maximum response is obtained at t = 0 sec when brace suddenly get removed then maximum deflection does not necessarily increase as brace stiffness increase.

2.5 COLUMN STIFFNESS AND LATERAL LOAD DISTRIBUTION ON STAGING OF WATER TANK

As per the studies carried out by **Sajjad Sameer U. and Sudhir K. Jain** the column stiffness given by 12 EI/L^3 is inaccurate, and the column axial deformation contributes 5 to 15 % of the total lateral displacement. The seismic force for the structure depends on the flexibility structure has. [11]

The assumption that Bracings attached to the columns are rigid is not true but depending upon that the columns stiffnesses are calculated by Indian standards. For tank staging whenever approximate methods are used they should be carefully applied due to three-dimensional behaviour of frame. Three approximate methods namely Portal method, Moment Distribution method and Simplified portal method are given by author for the calculation of staging stiffness.

The formulations for these methods are as follows [11]

Method 1 Portal Method

For columns of intermediate panels

$$Kcolumn = \frac{12EcIc}{h^3}\left[\frac{\sum Kbg}{\sum Kbg + 2Kc}\right]$$

For columns of uppermost and bottom most panels

$$Kcolumn = \frac{12EcIc}{h^3}\left[\frac{\sum Kbg}{\sum Kbg + Kc}\right]$$

Method 2 Moment Distribution Method

For columns of intermediate panels

$$Kcolumn = \frac{12EcIc}{h^3}\left[\frac{\sum Kbg}{\sum Kbg + 2Kc}\right]$$

For columns of uppermost and bottom most panels

$$Kcolumn = \frac{12EcIc}{h^3}\left[\frac{\sum Kbg + Kc}{\sum Kbg + 2Kc}\right]$$

Method 3 Simplified Portal Method

For intermediate panels

$$Kpanel = \frac{12EcIcNc}{h^3}\left[\frac{\left(\frac{EbIb}{L}\right)}{\left(\frac{EbIb}{L} + \frac{2EcIc}{h}\right)}\right]$$

For the uppermost and the bottommost panels

$$Kpanel = \frac{12EcIcNc}{h^3}\left[\frac{\left(\frac{EbIb}{L}\right)}{\left(\frac{EbIb}{L} + \frac{EcIc}{h}\right)}\right]$$

Where,

Eb	Modulus of elasticity of beam
Ib	Moment of Inertia of beam
L	Span of bracing girder
Ec	Modulus of elasticity of Column
Ic	Moment of inertia of Column
h	Panel height

Kbg	Stiffness of bracings with equal panel height at top and bottom
Kc	Stiffness of Column
Nc	No. of Column in periphery of staging

Another good paper on lateral load analysis of frame staging for Elevated Water tanks written by **Sajjad Sameer U. and Sudhir K. Jain** in which they have described overall seismic forces that are coming in the design of bracings, columns and other important component of elevated Water tank.

By the approximate method axial force in columns is obtained assuming it is proportional to the distance from bending axis of staging, and taking into account the shift in the inflexion point location. [12]

2.6 EFFECTS OF EARTHQUAKES ON LIQUID RETAINING STRUCTURES

After Bhuj Earthquake some reports gave observations regarding Elevated Water tanks damages. Some of them are as follows;

2.6.1 Ravi mistry, Weimin Dong, Haresh Shah took survey of the earthquake damaged areas and gave following observations:

- It was noted that all government-designed and operated overhead Water tanks with capacities ranging from 10,000 liters to 1,000,000 liters appeared to have withstood the earthquake without any apparent damage, regardless of their proximity to the epicenter. An example of one of these structures, which is located near Gandhidham, is shown in Figure 2.2.
- However, we understand that five elevated tanks failed in the area surrounding Morbi and Malia (approximately 60 km SE of the epicenter). Unfortunately, the reconnaissance team was unable to visit these tanks to ascertain the cause or severity of the damage. [13]

FIGURE 2.2 ELEVATED WATER TANK AT GANDHIDHAM, NO APPARENT DAMAGE

Durgesh C Rai describes various kinds of damages and overall causes of damages in shaft as well as trestle supported Water tanks after Bhuj earthquake on 26th January 2001 in Gujrat and surrounding area.

Durgesh C Rai [14] following are the observations and conclusions:

- Within the radius of 125 km from epicenter many overhead Water tanks suffered severe damages. In which *Shaft supported overhead tanks* shows tension cracks at bottom portion of shaft, whereas *RC framed staging tanks* located in regions of the highest intensity of shaking collapsed while a few developed cracking near brace-column joint regions.
- Staging diameter of shaft increases with increase in the capacity but thickness of the staging section is usually kept 150 and 200 mm. The flexure cracks in staging were observed from first to third lift portions. These cracks are mostly circumferential and throughout out perimeter of shaft. Cracks also observed in construction joints.
- The ESRs are behaving like *inverted pendulum* structures which resist lateral forces by the flexural strength and stiffness of their *circular hollow shaft type staging*. The section close to the ground is subjected to the maximum flexural

demand. Damage to shaft is critical as it affects load-bearing capacity seriously.

- For the *Frame type staging Water tanks* author writes that Frame type staging are superior to shaft type staging for lateral resistance as their *large redundancy* and *greater capacity to absorb seismic energy* through inelastic actions.
- RC frameworks can be designed to perform in a ductile fashion under lateral loads with greater reliability. The sections near the beam-ends can be designed and detailed to sustain inelastic deformation and dissipate seismic energy.
- In frame supported Water tanks, frame members and the brace column joints if not designed and detailed for inelastic deformations, a collapse of the staging may occur under seismic overloads.
- The current designs of RC shaft type circular staging (supporting structure) for elevated Water tanks are extremely vulnerable to lateral loads such as earthquakes. [14] Extreme damages can occur to tanks within 125 m from epicenter. This is despite the fact that most staging could withstand the seismic forces greater than those specified by *IS: 1893-1984*. Under seismic loads frame staging behaves quite differently than in normal framed structure.
- Also the staging does not have much redundancy and hence toughness (a desirable feature for earthquake-resistance which is present in the multiple bays and frame lines of a building framing system). This lack of redundancy is extremely serious in circular shaft type staging where lateral stability of the structure depends on only a single element, i.e., shaft, failure of which would severely affects the lateral stability of the entire structure. Shaft also has lack of ductility, which can be used during earthquake to resists lateral loads. [14]
- *IS: 1893-1984* code underestimates the design forces by at least a factor of 3 for Water tanks.
- The slender staging that results from the low design forces is a very unfavorable feature for seismic areas. Also, there are no provisions in IS codes for ductile detailing of shaft type (thin shell) tank staging generally shaft behave in a brittle manner, therefore, should be avoided.

- As Frame staging of Water tanks can be detailed as per provisions of *IS: 13920-1993* and *IS: 11682-1985* which refers to the ductility requirements of *IS: 4326-1976*.
- The failures of framed staging are lesser than shaft type.

After publishing this paper, the state professionals are in disagreement with to the conclusions of Durgesh Rai. As per many consultants and professionals, the paper is deficient in collecting experimental data, and extent of the survey.

Following are some of the conclusions that are derived by Professionals against **Durgesh Rai's** technical Paper.

- As far as Shaft and Frame staging are concerned, both are the alternative systems for supporting Water tank. Both are reasonably different structures so comparison between them should be avoided. Properly designed, well constructed, good maintained structural systems always behave better in resisting gravity and lateral loads.
- Damages to any structure needs to be examined very carefully. More often construction deficiencies are overlooked and emphasis shifts to design deficiencies. These aspects should be considered before drawing above-mentioned conclusion.
- As in the state of Gujarat, in most of the cases the construction is of shaft supported Water tanks, and so obviously the damages are observed more in such Water tanks compared to the frame supported Water tanks which are constructed on much lower scale.
- Regarding this issue data from GWSSB which is having records of more than 40 years construction of Water tanks in Gujarat shows Water tanks have survived Bhuj and other earthquakes even in areas near the epicenter, which were designed by *IS: 1893-1984* and functioning well till date. [15]

Regarding both shaft supported and trestle supported tank comparison some of the points are briefly studied and stated by **Prof. Y. T. Vani** in his letter to GSDMA. In the compiled letter author gave illustrations and note regarding the report of GWSSB where state government data shows about 2000 ESR's of capacity above 5000 liters which are shaft supported tanks which have performed well during Earthquake, even though these tanks were designed as per previous *IS: 1893 (1984)*. (Refer 5.0 of report to GSDMA)

Also author states

Any comparison in the two systems (Shaft and Trestle) of design has to be based on total design comparisons rather than comparison of isolated numbers. The load factors, assessment of limit state strength, various other design parameters, and the final comprehensive 'safety factor', all will affect the design. [16]

Prof C. H. Shah also gave explanation of how we can not blame only shaft as supporting system responsible for failures of tank.

As per Author we have to consider how the cracks in Water tank develops, which may be of following type due to

- Corrosion of reinforcement
- Honeycombing at the construction joints
- Inadequate strength

Similarly the causes of failures of structures like lack of ductile detailing, poor quality construction, and poor maintenance should be given proper importance before coming to any conclusion regarding any supporting system to be better than other structural system. [16]

2.7 EFFECTS OF WINDS ON LIQUID RETAINING STRUCTURES

B. Tansel, M. ASCE, and N. Ahmed writes about Structural stability of Elevated Water Reservoirs under Hurricane force wind conditions.

It is necessary to provide uninterrupted supply of electricity, telecommunication facilities and Water supply during natural disasters. Elevated Water reservoirs are designed to serve two basic purposes that are equalizing Water storage volume and emergency Water storage.

Elevated Tank Configurations

Design of elevated tanks depends on capacity, required elevation, size and shape of structural members, stability of structure and foundation, type and installation of appurtenances for operation. Towers supporting elevated tanks are designed to act as a unit with the tank so that they should able to resist the combined stresses due to weight and bending. Elevated storage tanks' configurations are based on the shape of storage tank and the riser structures, which include double ellipsoidal, spherical, and ped-cone (spherical with conical base and top). Towers configurations includes fluted pillar, composite or multi-legged.

The height of the elevated storage tank depends on allowable bearing capacity of the soil on which the tank will be built. Author suggests Water tanks to be kept at full capacity level to avoid damages due to high winds during storm.

2.8 SOIL STRUCTURE INTERACTION

Somnath Datta, S. C. Datta & et al, discusses the Soil structure interaction in dynamic analysis of frame staging Water retaining structure briefly.

The paper aims to observe the effect of soil–structure interaction on two dynamic characteristics namely, the impulsive lateral period, which regulates lateral seismic behaviour and the impulsive torsional-to-lateral period ratio that regulates torsional vulnerability of the structure.

A parametric study with limited example tanks based on these formulations shows that the frame staging with all kinds of alternate configurations having less panel heights, more number of columns, larger column diameter and stiffer circumferential beams compared to columns encounters the strongest influence of soil–structure interaction effect. [18]

The staging types considered for study are as follows

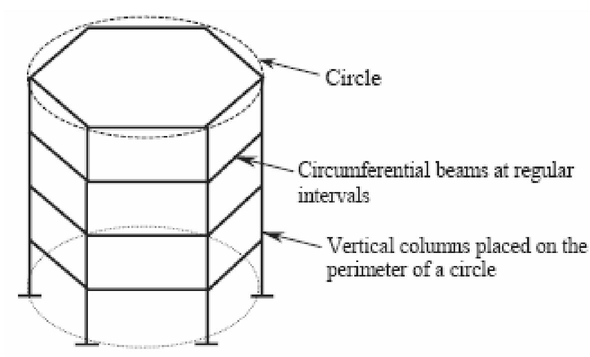


FIGURE 2.3 USUAL TRESTLE TOWER CONFIGURATIONS

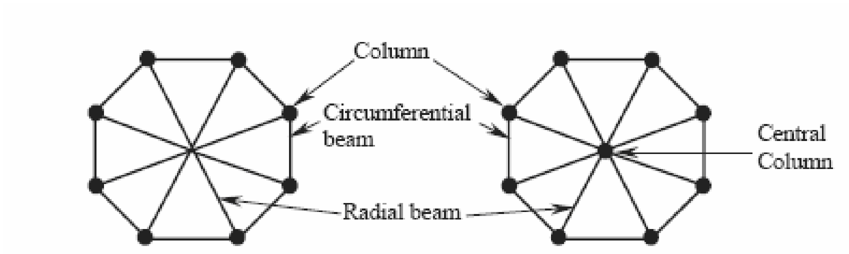


FIGURE 2.4 STAGING WITH RADIAL BEAMS & WITH RADIAL BEAMS AND CENTRAL COLUMN

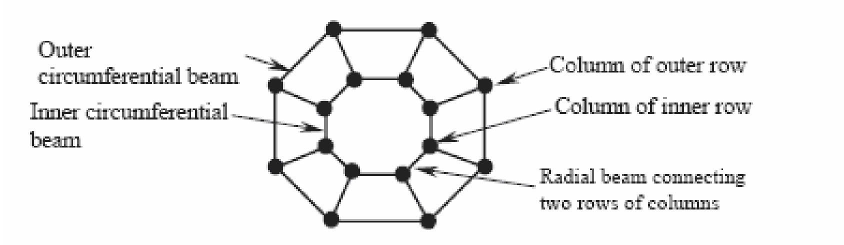


FIGURE 2.5 STAGING WITH TWO CONCENTRIC ROWS OF COLUMNS CONNECTED BY RADIAL AND CIRCUMFERENTIAL BEAMS

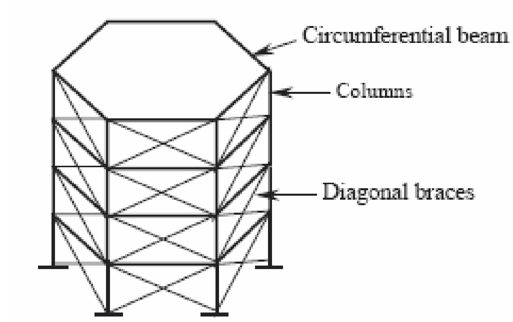


FIGURE 2.6 DIAGONAL BRACES

Author gives formulation for calculation of time period of various staging configuration. The study of effect of soil–structure interaction is mainly done on two important dynamic characteristics of elevated tanks supported by frame staging with a few alternate configurations. Analytical formulations are validated and employed in the present study so that influential parameters can be well identified and can be varied within their feasible range of variations. Following conclusions can be arrived from the study:

Soil–structure interaction [18]

- Considerably, increases the impulsive lateral period and decreases the impulsive torsional-to-lateral period ratio.
- Having stronger effect in case of elevated tanks supported by alternate frame staging configurations with panels of small heights, and larger number of columns, large column diameter and stiffer circumferential beams compared to the columns.
- Analysis with fixed base assumption may lead to underestimation or overestimation of seismic base shear of elevated tanks with any alternate staging configuration at both tank-full and tank-empty conditions. Soil-flexibility may cause tension in some of the staging columns at tank-empty

condition resulting risk for the performance of elevated tanks resting on various staging.

- If elevated tanks designed on the basis of a fixed base assumption. Ignoring soil–structure interaction may also lead to wrong assessment of torsional vulnerability.
- Considerable effect of soil–structure interaction on dynamic characteristics of both of them due to their stiffer structural configurations should be accounted for seismic design.

3. REVIEW OF PROPOSED DRAFT CODE

3.1 GENERAL

As far as Water Tower is concerned it shows high susceptibility to lateral forces due to its structural form in case of Elevated Water tanks. Also the amount of lateral forces subjected on structure depends on structures rigidity. Rigid the structure, attracts more forces. By the use of provisions of *IS: 1893-2005(Part II)* Proposed Draft code there is large increase of design forces on Water tanks approximately three to four times as per case study considered in this project where the stiffness of the staging is calculated considering it as flexible structure. It's necessary to review the provisions of Proposed Draft, which is being circulated before its acceptance by BIS. Some of the modifications are done in Proposed Draft are quite admirable but some ambiguity are also present in some of the provisions. Here one by one some provisions are reviewed.

Some of the major modifications included in Proposed Draft are as follows

a) Analysis of ground supported tanks.

b) For elevated tanks two-degree of freedom idealization is used for analysis.

c) Bracing beam flexibility is included for calculation of lateral stiffness of tank staging.

d) The effect of convective hydrodynamic pressure inclusion.

e) Hydrodynamic pressure distribution suggestion stress analysis of tank wall.

f) Effect of vertical ground acceleration on hydrodynamic pressure is considered.

3.1.1 SPRING MASS MODEL FOR SEISMIC ANALYSIS

The IS 1893-1984 assumes Elevated Water tank behaves as Single degree of freedom but research work [19] shows the Water inside Water container can be separated for the purpose of analysis as some portion of Water on application of lateral forces tries to be retained with the container whereas above mass of Water sloshes producing wave pressure. This dual nature behaviour is accepted in Proposed Draft making Elevated Water tank as Two degree of freedom system, which is also rational one.

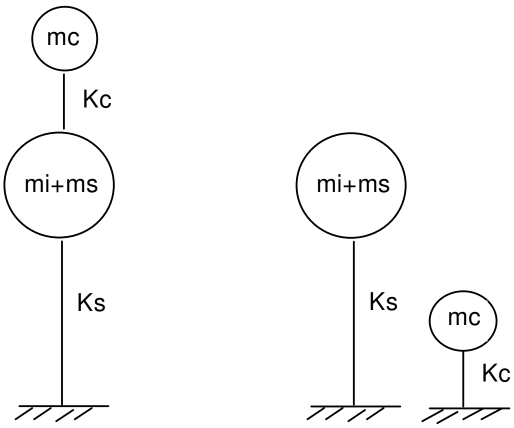


FIGURE 3.1 TWO MASS IDEALISATION AND EQUIVALENT UNCOUPLED SYSTEM

Here it is necessary to note that for consideration of equivalent uncoupled system behaviour both times period of impulsive and convective mode as well separated preferably Ti/ Tc > 2.5.

Where,

m_c = Convective mass of liquid

m_i: Impulsive mass of liquid

m_s: Mass of container of elevated tank and one-third mass of staging

K_c: Spring stiffness of convective Mode

K_s: Lateral stiffness of elevated tank staging

Ti: Impulsive mode time period

Tc: Convective mode time period

3.1.2 TIME PERIOD

$$T_i = 2\pi\sqrt{((m_i + m_s)/K_s)} \tag{3.1}$$

Time period for impulsive mode is given by above Equation (3.1)

Where, Ks is the stiffness of staging.

And for convective mode time period following equation is used.

$$T_c = C_C\sqrt{\frac{D}{g}} \tag{3.2}$$

Where,

Cc is the coefficient of convective mode is found out by following graph shown.

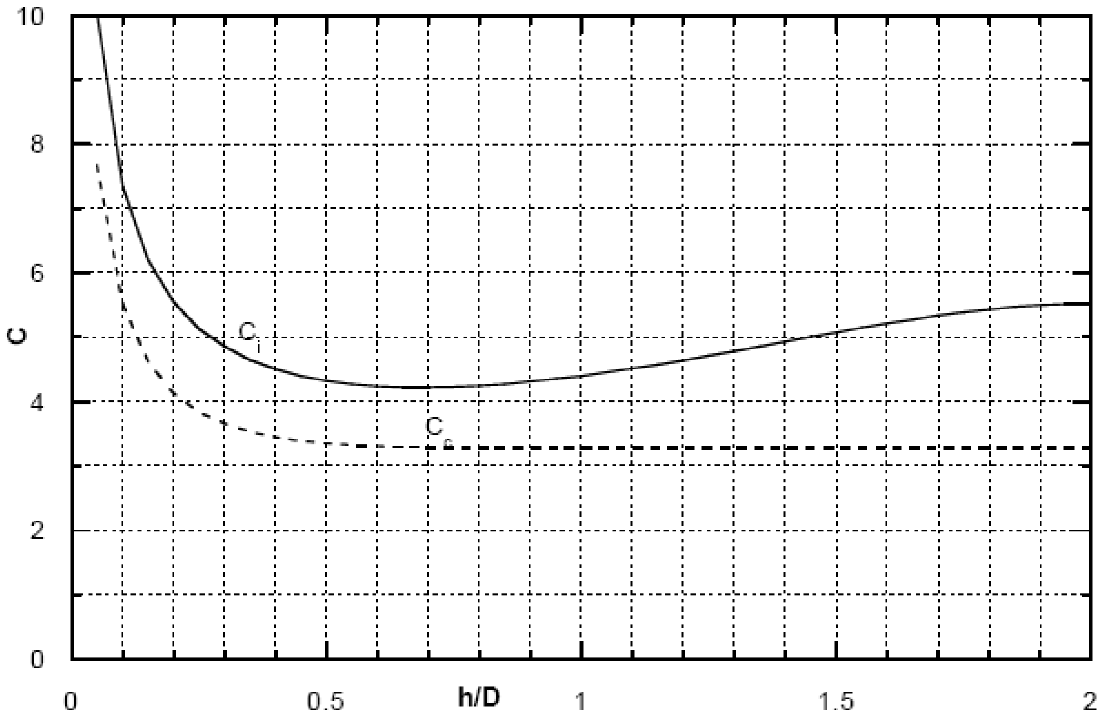


FIGURE 3.2 COEFFICIENT OF IMPULSIVE (Ci) AND CONVECTIVE (Cc) MODE TIME PERIOD FOR CIRCULAR TANK.

Design of Water tank is carried out for Tank Empty and Tank full condition.

On general observations Tank full condition governs design in the higher earthquake prone areas.

3.1.3 DAMPING

Damping in the convective mode for all types of liquids and for all types of tanks is mentioned to take as 0.5% of the critical. Damping in the impulsive mode shall be taken as 2% of the critical for steel tanks and 5% of the critical for concrete or masonry tanks.

3.1.4 DESIGN HORIZONTAL SEISMIC COEFFICIENT

For the seismic analysis on general scale only horizontal seismic coefficient is considered and it is assumed that the mass of structure and hydrodynamic forces will be sufficient to deal with for the vertical acceleration component.

The formula for calculating design horizontal seismic coefficient is as follows

$$A = \left[\frac{Z}{2} \frac{I}{R} \frac{S_a}{g} \right]$$

Where,

Z = Zone factor given in Table 2 of *IS 1893-2002 (Part 1),*

I = Importance factor given in Table 1 of Proposed Draft code,

R = Response reduction factor given in Table 2 of Proposed Draft code,

Sa/g = Average response acceleration coefficient as given by Figure 2 and Table 3 of *IS: 1893-2002 (Part 1)* and subject to Clauses 4.5.1 to 4.5.4 of Proposed Draft code.

3.1.5 BASE SHEAR

For Ground Supported Tank

Base shear in impulsive mode, at the bottom of tank wall is given by

$$V_i = (A_h)_i (m_i + m_w + m_t) g$$

And base shear in convective mode is given by

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

Where,

$(Ah)_i$ = Design horizontal seismic coefficient for impulsive mode,

$(Ah)_c$ = Design horizontal seismic coefficient for convective mode,

mi = Impulsive mass of Water

mw = Mass of tank wall

mt = Mass of roof slab, and

g = Acceleration due to gravity.

Similarly for Elevated Water tanks

Base shear at the top of footing is

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

And the convective base shear is given by

$$V_C = (A_h)_C m_s g$$

Where,

ms = Mass of container and one-third mass of staging.

mi= Impulsive mass of water.

The resultant base shear is found out by using SRSS i.e. Square root of Sum of Squares method.

Thus base shear equation becomes,

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

3.1.6 BASE MOMENT

For Ground supported tanks

Bending moment in impulsive mode, at the bottom of wall is given by

$$M_i = (A_h)_i (m_i h_i + m_w h_w + m_t h_t) g$$

and bending moment in convective mode is given by

$$M_c = (A_h)_c m_c h_c g$$

Where,

hw = Height of center of gravity of wall mass, and

ht = Height of center of gravity of roof mass.

Overturning moment in impulsive mode to be used for checking the tank stability at the bottom of base slab/plate is given by

$$M_i^* = (A_h)_i \left[m_i (h_i^* + t_b) + m_w (h_w + t_b) + m_t (h_t + t_b) + \frac{m_b t_b}{2} \right] g$$

and overturning moment in convective mode is given by

$$M_c^* = (Ah)_c m_c (h_c^* + t_b) g$$

Where,

mb = mass of base slab/plate, and

tb = thickness of base slab/plate.

hi* Height of impulsive mass above bottom of tank wall (considering base pressure)

For the Elevated Water tanks

Overturning moment in impulsive mode, at the base of the staging is given by

$$M_i^* = (A_h)_i [m_i (h_i^* + h_s) + m_s h_{cg}] g$$

and overturning moment in convective mode is given by

$$M_c^* = (A_h)_c m_c (h_c^* + h_s) g$$

Where,

hs = Structural height of staging, measured from top of footing of staging to the bottom of tank wall, and

hcg = Height of center of gravity of empty container, measured from base of staging.

Total moment shall be obtained by combining the moment in impulsive and convective modes through Square root of Sum of Squares (SRSS) and is given as

$$M_i^* = \sqrt{M^{*2}_i + M^{*2}_c)}$$

3.1.7 DIRECTION OF EARTHQUAKE FORCES

For Ground supported rectangular tanks

These tanks to be analyzed for horizontal earthquake force acting non-concurrently along each of the horizontal axes of the tank for evaluating forces on tank walls.

For elevated tanks,

Staging components should be designed for the critical direction of seismic force. Different components of staging may have different critical directions.

The alternative to above clause is to consider following load combination for designing trestle i.e. staging and its components

Load combination rules:

i) 100% + 30% Rule:

$$\pm ELx \pm 0.3ELy \text{ and } \pm 0.3ELx \pm ELy$$

ii) SRSS Rule:

$$\sqrt{EL_x^{\ 2} + EL_y^{\ 2}}$$

Where, ELx is response quantity due to earthquake load applied in x-direction and Ely is response quantity due to earthquake load applied in y-direction.

3.2 REVIEWS

As far as Proposed Draft provisions are concerned there are some admirable inclusions like two mass representations, but in case of design values, Proposed Draft clauses increases design forces by 1.2 times approximately on Trestle supported Water tanks. The solved example in this project also shows the same amount of increase in design forces.

The following issues need to be clarified in Proposed Draft like

Importance factor

The importance factor mentioned in Draft for drinking Water tanks is 1.5 which is same for hazardous fluid containing tanks.

Response reduction Factor

The response reduction factor in case of building particularly for Special Moment Resisting Frames is 5 in *IS: 1893-2002(PART I)* but for frame staging supporting Water tanks the value sudden reduces to 2.5 without giving any explanation regarding R value reduction.

As mentioned in Proposed Draft R values are taken directly from international codes. This is not very convincing. Just taking R-values from foreign codes without examining its implication on the final design may result in an approach, which is very conservative.

Regarding shaft-supported tanks also, codes recommend lesser R-value than frame support. This is based on the belief that Shafts are more vulnerable to earthquakes even though most of the shaft-supported tanks appear to have performed well during earthquakes in Gujarat. [15]

Additionally the tremendous increase in base shear in case of Shaft supported tanks will result in very large raft foundations without proportionate increase in shaft diameter. In normal soils, this may result in unacceptable increase in cost of Shaft supported tanks, making them unviable in comparison to Trestle supported tanks.

Sloshing height

Elevated Water tanks are usually covered at the top. Water within the free board space is also considered for design for Full tank condition. As a result there is very little space above. Additionally there are obstructions such as inner supporting columns, inner shafts etc. preventing sloshing. In this circumstance when earthquake occurs then the water body will mainly behave in impulsive mode. The proposed code does not address this aspect when it recommends use of dual mass i.e. impulsive and convective mass of water.

4. ANALYSIS AND DESIGN OF TRESTLE SUPPORTED WATER TANK

4.1 STIFFNESS OF STRUCTURE

More accurate we calculate the natural time period of structure, more realistic we can predict the behaviour of the structure for Lateral force. As far as water tank trestles are concerned no specific formula for calculation of stiffness is mentioned in Indian Standard *IS: 11682-1985* Criteria for Design of RCC staging for Overhead Water Tanks which allows the designer flexibility in the use of formula for stiffness as per his perception of frame supported trestle.

The trestle is the space frame, which behaves quite differently than normal building frame. Thus for calculating stiffness we had to consider overall no.of bracings which may be in horizontal or vertical plane or may be in inclined plane and the amount of rigidity the joints have, because it also affects the stiffness.

In present project the stiffness is calculated by the method mentioned in IS 1893-2005 (Part II) Proposed Draft code, which is more accurate than other formulations as 3D behaviour of the trestle, is incorporated in it. In *SP 22 (S & T) 1982*, Explanatory Handbook on Codes for Earthquake Engineering, Bureau of Indian Standards examples of trestle supported water tanks are solved considering frame staging as rigidly connected by bracings which is too conservative. Thus in this work flexibility of frame staging is incorporated as per Proposed draft code provisions.

The two arrangements of bracing systems are considered, first one with Peripheral bracing or polygonal bracing and second one with steel diagonal bracing in addition to peripheral bracings.

For the calculation of both kinds of arrangements of trestle STAAD Pro software is used where we find out displacement for the node simulated at CG level of container for any known force. As the stiffness is the force required for unit displacement of structure.

$$K = \frac{F}{\Delta} \tag{5.1}$$

Where,

K is the stiffness of structure

F is the known force applied at CG of the container

Δ is the displacement observed

We can easily decide the stiffness of structure. For both arrangements following models were prepared and stiffness calculated as per previously mentioned procedure.

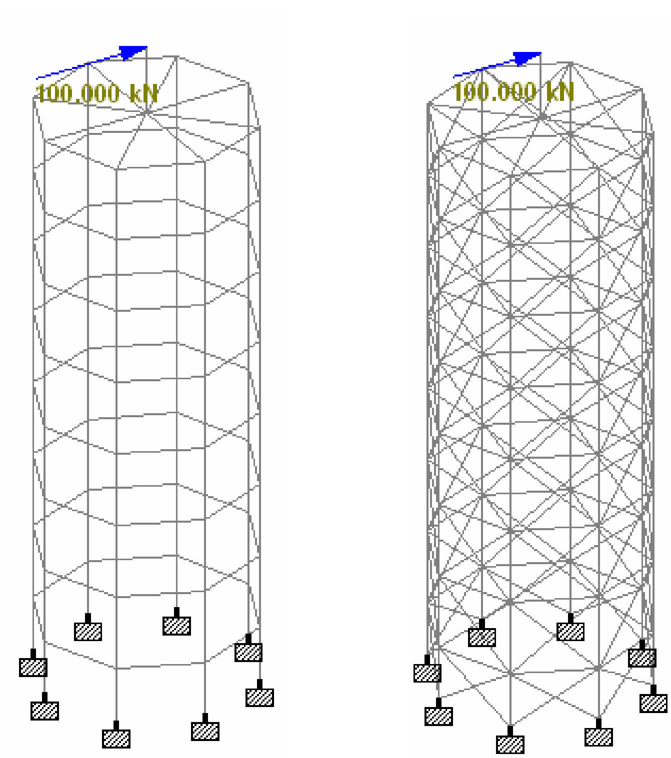


FIGURE: 4.1 STIFFNESS CALCULATIONS FOR TRESTLE

On application of lateral force 100 kN, we get the displacement of the node at CG of container as 24.535 mm and 12.223 mm respectively for Peripherally braced, and Peripherally with Diagonally braced trestle.

Thus for Peripherally braced trestle

Stiffness = 100000/ 24.535 = **4075.81** N/mm

And for Peripherally with diagonally braced trestle

Stiffness = 100000/ 12.223 = **8181.29** N/mm

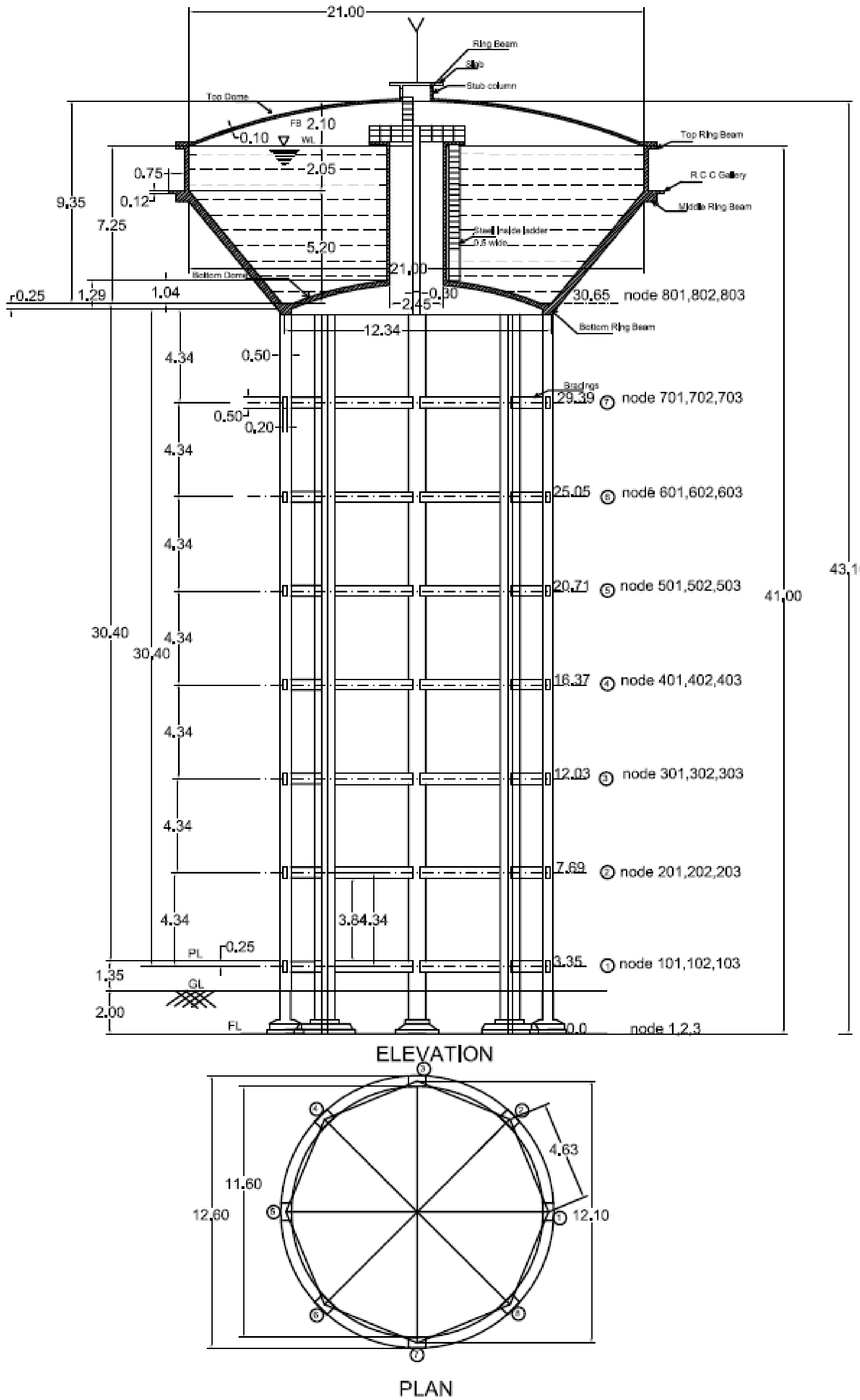


FIGURE 4.2 ELEVATION AND PLAN OF CASE STUDY PROBLEM

4.2 INTZE TANK CONTAINER DESIGN

Data

Grade of Concrete	20	N/mm^2
Grade of Steel	415	N/mm^2
Concrete Density	24.53	kN/m^3
Capacity of the tank in Liters	1800000	liters
Capacity of the tank in m^3	1800	m^3
Height of Plinth level	1.35	m
Height of the staging above ground level	30	m
Basic Wind Pressure	1.5	kN/m^2
S.B.C. of the Soil	250	kN/m^2
Depth of the foundation	2	m

Permissible Stresses

As per IS:456-2000 (Table:21) & IS:3370(Part-II) (Cl:3.3)

Permissible stress in compression (Direct) σcc	=	5	N/mm^2
Permissible stress in compression in			
Bending σcbc	=	7	N/mm^2
Permissible stress in steel σst_1	=	150	N/mm^2
Permissible stress in steel beyond 225 mm thkσst_2	=	190	N/mm^2
Modular ratio (m -1)	=	13	
Coeff. of moment of resistance Q	=	0.9	
Permissible stress in concrete (Direct tension) σct	=	2.8	N/mm^2
Permissible stress in concrete (Bond strength)ζbd	=	0.8	N/mm^2
for deformed bars ζbd	=	1.28	N/mm^2
for reinforcement bars in compression ζbd	=	1	N/mm^2

4.2.1 ASSUMED DIMENTIONS OF WATER CONTAINER

Inside diameter of tank	21.00	m
Thickness of top dome	0.10	m
Thickness of stair shaft	0.16	m
Thickness of bottom dome	0.20	m
Thickness of cylindrical wall at top t_2	0.20	m
Width of walking gallery at middle level	0.75	m
Thicknessf walking gallery at middle level	0.12	m
Rise of top Dome	2.10	m
Height of Cylindrical wall	2.05	m
Rise of Bottom Dome	1.30	m
Thickness of Bottom Dome shell	0.20	m
Thickness of conical shell	0.50	m
Depth of Freeboard	0.30	m
Inclination of Conical wall with horizontal θ	52.00	°
Height of conical wall	5.20	m
Radius of Bottom Ring Beam	12.60	m
Height of staging above Plinth level	30.00	m
Height of staging above G.L.	31.35	m
Depth of Footing from G.L.	2.00	m
Diameter of spiral stair column	0.30	m

4.2.2 DESIGN OF TOP DOME

load calculation

Thickness of dome	(t)	0.1	m
Self weight of dome	(w_{self})	2.5	kN/m^2
Live load on dome	(wl)	0.75	kN/m^2
Load due to Finishes	(w_f)	0.4	kN/m^2
Total load	(w) = (w_{self}) + (wl) + (w_f)	3.60	kN/m^2

Rise of Top Dome	(h_1)		2.1	m
Inside Radius of Tank	(r_1)		10.5	m
Surface Radius :	$R = 0.5 \times (r_1^2/h_1+h_1)$		27.3	m
	$\cos\phi$		0.82	
	ϕ		35.23	
Total live load on dome	$W_1=2\,\Pi R h_1 wl$		270.16	kN
Self weight of Dome	$W_2=2\Pi R h_1 w_d$		1027.51	kN
Meridional thrust	$N\phi = wR/(1+\cos\phi)$		54.13	kN/m
Compressive stress	$N\phi/t$		0.54	MPa
$Nq = wR(\cos\phi - 1/(1+\cos\phi)$			26.205	kN/m
Minimum reinforcement in either direction			150	mm^2/m
Provide 8 mm dia @ 300 mm c/c so steel provided			167.55	mm^2/m
Percentage of Steel			0.17	%
Refering IS: 3370 (Part III) Percentage of steel			0.30	%
For Tor steel			0.24	%

4.2.3 DESIGN OF TOP RING BEAM

Hoop tension in ring beam $T = (N\,\phi\cos\phi)(r_1)$			464.28	kN
Area of tension steel required = $As = T/\,\sigma st_1$			3095	mm^2
Bar diameter			20	mm
No. of bars needed	9.85	≅	10	No.s
Area of tension steel provided Asp =			3142	mm^2
				Check Safe
Let width of beam b_1 =			300	mm
Depth of beam required D_1 =	424	≅	400	mm
Tensile stress in concrete =			2.92	kN/m^2
				Check Safe

So Provide top ring beam of 300 x 400 mm with 10 No.s of 20 mm dia.

Main steel with 6 mm dia.@ 250 mm c/c.

Weight of Top Ring Beam	W_3		197	kN

4.2.4 DESIGN OF CYLINDRICAL WALL

Wall height h			2.05	m
Maximum hoop Tension is at h			2.05	m
Maximum hoop Tension $T_{1=}$ g r_2h			215	kN
Ast required = T_1 /σst			1435	mm^2
Bar Diameter =			16	mm
No.s of bars needed	7.14	≅	8	No.s
Area of tension steel provided	Asp		1608	mm^2

Check Safe

Hoop tension	At interval			Steel reqd	No.s bars	Min steel mm^2	**0.27**
	1.55	1.05	163	910	5	27.14	**Safe**
	1.05	0.55	110	560	3		**Safe**
	0.55	0	58	178	1		**Safe**

Bar Diameter				10		mm	
No.s of bars needed	for	1.55	1.05	2	162.8	mm^2	**Safe**
Area of tension steel provided	for	1.05	0.55	1	110.3	mm^3	**Safe**
	for	0.55	0	1	57.75	mm^4	**Safe**

Weight of Vertical Cylindrical Wall W_4 669.70 kN

4.2.5 DESIGN OF MIDDLE GALLARY

Width of walking gallery	0.75	m
Thickness of middle gallery slab	0.12	m
Width of Bottom Ring beam	0.5	m
Cantilever portion of slab	0.25	
Load calculation		
Self weight of gallery slab	736	N/m
Railing Loads	750	N/m
B.M.due to self weight	91.97	N/m/m

B.M.due to Railing		97.50	N/m/m	
B.M.due to UDL (Live load)		46.88	N/m/m	
OR B.M.due to end live load		130	N/m/m	
Total B.M. =		319.47	N/m/m	
Depth of slab required		18.88	mm	
Depth of slab Provided		120	mm	**Safe**
Effective depth =		90	mm	
Area of steel required		17.03	mm^2	
Provide bar diameter		8	mm	
No.s of Bars required	0.3 ≅	1		
Provide 8mm dia radial bar at 300 mm c/c & anchored into ring beam				
As provided		50.27	mm^2	**Safe**
Total weight of slab	W_5	51.43	kN	
Total live load on middle gallery	W_6	78.99	kN	
Total Railing load	W_7	53.96	kN	

4.2.6 DESIGN OF MIDDLE RING BEAM

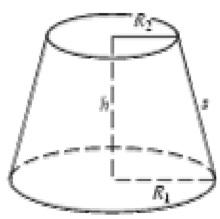
Let the width of Middle Ring Beam	b_2	500	mm
Let the Depth of Middle Ring Beam	d_2	500	mm
Self weight of the beam		6.25	kN/m
Total Self Weight W_8 =	W_8	430.01	kN
LL from the Top Dome	W_1	270.16	kN
Weight of Top Dome	W_2	1027.51	kN
Weight of Top Ring Beam	W_3	196.93	kN
Weight of Vertical wall	W_4	669.70	kN
Weight of Galllery slab	W_5	51.43	kN
LL on Gallery	W_6	78.99	kN
Weight of the Railing	W_7	53.96	kN
Total load transferred to the Conical Wall	W_9	2348.68	kN
Hoop Tension on Ring Beam T= W9*cot ϕ /(2P)		529.33	kN
Area of steel in Ring Beam		3528.84	mm^2

Providing Bar Diameter			20	mm	
No.s of Bars required	11.2	≅	12.00		
As Provided	Asp		3769.91	mm^2	**Check Safe**
Tensile stress in concrete			0.944994	kN/m^2	**Check Safe**

Provide 500x500 mm with 20 mm dia. 12 no.s

4.2.7 DESIGN OF CONICAL SHELL

Let thickness of the conical shell		0.50	m
Slope of the wall	θ	52	°
Height of Conical Wall	h_3	5.2	m
Self weight of slab		12.5	kN/m^2
Radius at bottom of conical slab	r_3	6.44	
Length of slab		6.60	m



Surface Area

$$A = \pi(R_1 + R_2)s = \pi(R_1 + R_2)\sqrt{(R_1 - R_2)^2 + h^2}.$$

FIGURE: 4.3 CONICAL SHELL VOLUME CALCULATIONS

The surface area, not including the top and bottom circles,		355.85076	m^2	
Weight of conical wall	W_{11}	6560.83	kN	
Weight of Water over conical wall	W_{12}	10052.19	kN	
Total load on conical slab	W_{13}	18961.70	kN	
Meridional thrust in slab of cone		594.92	kN/m	
Horizontal component of Meridiaonal Thrust	H_1	366.27	kN/m	
Thickness of Conical Slab at base		0.500	m	
The Meridional Thrust		1.19	MPa	**Check Safe**

Hoop tension from base of the cone

r_x = 6.4 + y

Height of water at this level h_x = h + (h_3 - y)

Normal load on the slanting slab px = water pressure + component of weight of the slab

P_x = (2.05+5.2-y)*10+12.5cosθ = 80.2 - 10Y

Provide Bar Diameter =	20	mm

Hoop Tension in the slab T = (p_x cosecθ) (r_x)

				Hoop tension k	Bars Reqd	Bars prov		Stress in conc
At	y =	0	m	655.16	13.9	14	**Safe**	0.01
At	y =	1	m	662.55	14.1	15	**Safe**	0.01
At	y =	2	m	644.57	13.7	14	**Safe**	0.01
At	y =	3	m	601.20	12.8	13	**Safe**	0.01
At	y =	4	m	532.45	11.3	12	**Safe**	0.01
At	y =	5	m	438.33	9.3	10	**Safe**	0.01
At	y =	5.2	m	416.46	8.8	9	**Safe**	0.01

Minimum reinforcement in radial direction 0.30 % 9898.34 mm^2

Provide 12 mm dia at 300 mm c/c

4.2.8 DESIGN OF BOTTOM DOME

Half chord length: r_3	6.44	m
Rise of the Dome: h_2	1.30	m
Thickness of the shell (Bottom dome)	0.20	m
Radius of dome: R	16.6	m
Self weight of the slab: W_{14}	664.60	kN
Weight of water over Bottom Dome: W_{15}	8580.68	kN
Total weight of the dome: W_{16}	9245.27	kN
Semicentral angle: Sin ϕ	0.39	
Meridional thrust: N ϕ	589.02	kN/m
Compressive Stress	2.95	N/mm^2
		Nominal
Minimum Reinforcement	460.00	mm^2/m

Provide 12 mm dia at 300 mm c/c

Horizontal thrust: H_2	362.63	kN/m

4.2.9 DESIGN OF BOTTOM RING BEAM

Radius of Bottom Ring Beam 6.30 m

The horizontal component of thrust from shell at ring beam level

H_1 366.27 kN/m

The horizontal component of Meridional thrust from bottom dome at beam level

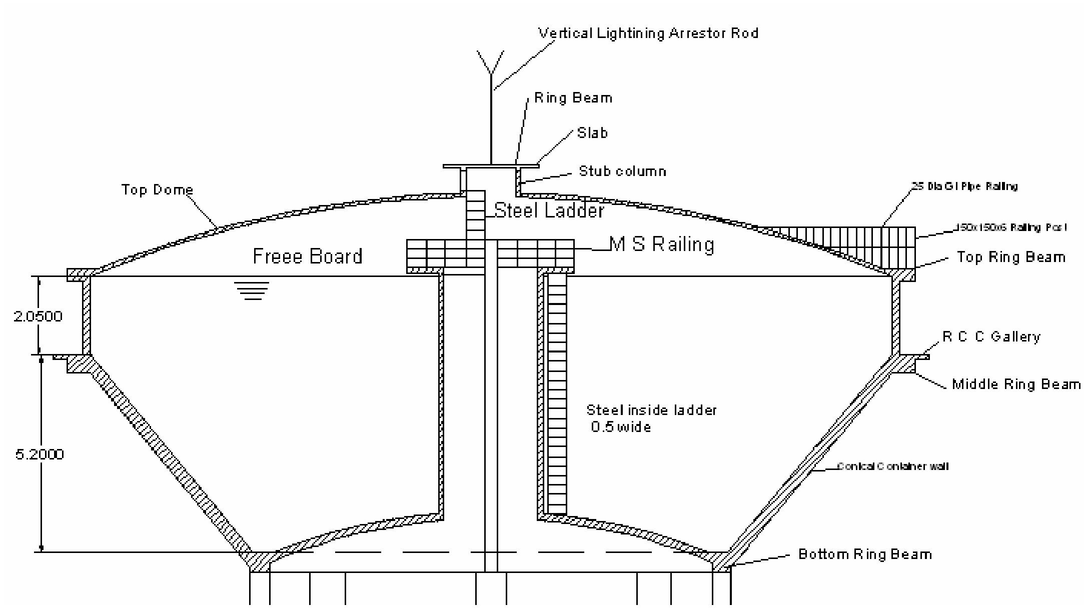
H_3 3.64 kN/m

The net force is inwards and causes compression on ring beam. The Hoop compression is

	T		23.41	kN
Providing a Ring Beam of width			500	mm
And depth of Ring Beam			500	mm
The Compressive stress			0.09	N/mm^2
			Nominal	
The Minimum reinforcement be 0.24 %			600	mm^2
Provide Bar Diameter			16	mm
No.s of Bars required	2.98	≅	4	No.s
Provide 16 mm dia. 4 No.s with 8 mm @ 300 mm c/c				
Weight of the beam	W_{17}		247.99	kN
Total weight of concrete upto ring beam is	W_{18}		28454.96	kN
Weight of water	W_{19}		18633	kN
Total weight	W		28454.96	kN
Mass of the container only			9822.10	kN

4.3 PERIPHERALLY BRACED TRESTLE

4.3.1 SEISMIC ANALYSIS AS PER PROPOSED DRAFT CODE



Mass of the container only:	= 9822.10	kN =	1001491	Kg
Mass of water in container, m	= 18633	kN =	1899888.26	Kg
Mass of columns:	= 2560	kN		
Mass of braces:	= 536.27	kN		
Mass of the staging:	= 3096.27	kN =	315708.07	Kg
Total mass of full tank:	= 31551.23	kN =	3217089.93	Kg
Total mass of empty tank:	= 12918.37	kN =	1317208.77	Kg

Mass of container and one third mass of staging is expressed as

ms = 9822.10 + (3096.27)/3

= 10854.19 kN = **1106693.31 Kg**

Lateral stiffness of staging which is only Peripherally braced

Ks = 4075.81kN/m = **4075810 N/m**

CG of empty container

= **3.528** m from top of bottom ring beam

Ec Modulus of Elasticity of concrete

= 25000 N/mm^2 = 25 kN/mm^2 = 25000000 kN/m^2

Top Dome Weight	=	1027.51 kN
Top Ring Beam Weight	=	196.93 kN

Cylindrical Wall	=	669.70 kN
Bottom Ring Beam	=	247.99 kN
Circular Ring Beam	=	430.01 kN
Bottom Dome	=	664.60 kN
Conical Dome	=	6560.83 kN
Water	=	18632.86 kN
Columns	=	2560.00 kN
Bracings	=	536.27 kN

Centre of Gravity of Empty Container from bottom slab

= 3.52 m

Mass **m** = mass of water = **1899888.26 kg**

Let h be the height equivalent circular cylinder

$$\pi\left(\frac{D}{2}\right)^2 h = 1899.88$$

So **h** = $1899.88 \text{x} \dfrac{4}{21^2 \text{x} \pi}$ = **5.48** m

Inner diameter of tank, **D** = **21** m, h/D = 5.48/21 = **0.26** so,

$$\frac{mi}{m} = \frac{\tanh 0.866 \frac{D}{h}}{0.866 \frac{D}{h}} = \frac{\tanh 0.866 \frac{21}{5.48}}{0.866 \frac{21}{5.48}} = 0.300746$$

mi = 1899888.26 x 0.300746

= **571385.59 kg**

hi/ h = 0.375 < 0.75 so **hi** = 0.375 x 5.48 = **2.06** m

$$\frac{hi^*}{h} = \frac{0.866 \frac{D}{h}}{2 \tanh 0.866 \frac{D}{h}} = \frac{0.866 \frac{21}{5.48}}{2 \tanh 0.866 \frac{21}{5.48}} = \mathbf{1.65} \text{ for h/D <1.33}$$

so **hi*** = 1.65 x 5.48 = **9.07 m**

$$\frac{mc}{m} = 0.23 \frac{\tanh 3.68 \frac{h}{D}}{\frac{h}{D}} = 0.23 \frac{\tanh 3.68 \frac{5.48}{21}}{\frac{5.48}{21}} = \mathbf{0.66}$$

so **mc** = 0.66 x 1899888.26 = **1246178.6 kg**

$$\frac{hc}{h} = 1 - \frac{\cosh\left(3.68\frac{h}{D}\right) - 1.0}{3.68\frac{h}{D}\sinh\left(3.68\frac{h}{D}\right)} = 1 - \frac{\cosh\left(3.68\frac{5.48}{21}\right) - 1.0}{3.68\frac{5.48}{21}\sinh\left(3.68\frac{5.48}{21}\right)} = 0.54$$

so **hc** = 0.54 x 5.48 = **2.94** m

$$\frac{hc^*}{h} = 1 - \frac{\cosh\left(3.68\frac{h}{D}\right) - 2.01}{3.68\frac{h}{D}\sinh\left(3.68\frac{h}{D}\right)} = 1 - \frac{\cosh\left(3.68\frac{5.48}{21}\right) - 2.01}{3.68\frac{5.48}{21}\sinh\left(3.68\frac{5.48}{21}\right)} = 1.48$$

so **hc*** = 1.48 x 5.48 = **8.10** m

$$kc = 0.836\frac{mg}{h}\tanh^2\left(3.68\frac{h}{D}\right) = kc = 0.836\frac{1899888.26 \text{x} 9.81}{5.48}\tanh^2\left(3.68\frac{5.48}{21}\right)$$

K_c = **1575780.14** N/mm

h_s = **31.60** m ....Structural height of staging, measured from top of foundation to the bottom of container wall

h_{cg} = **35.13** m ....Height of center of gravity of the empty container of elevated tank, measured from base of staging

Time period:

Time period of Impulsive mode,

$$T_i = 2\pi\sqrt{\left(\frac{(m_i + m_s)}{K_s}\right)} = 2\pi\sqrt{\left(\frac{(571385.59 + 1106693.31)}{4075810}\right)} = \mathbf{4.03} \text{ sec}$$

Time period of Convective mode,

For h/D = 0.26, Cc = 3.8 from Graph

$$T_c = C_C\sqrt{\frac{D}{g}} = 3.8\sqrt{\frac{21}{9.81}} = \mathbf{Tc} = \mathbf{5.56} \text{ sec}$$

Design horizontal Seismic Coefficients

Design horizontal seismic coefficient for impulsive mode,

$$(Ah)_i = \frac{Z}{2}\frac{I}{R}\left(\frac{Sa}{g}\right)_i$$

Where, Z = **0.16** ……. (IS 1893, PART (I), Table2, Zone III)

I = **1.5**

Since Staging has Special Moment Resisting Frames (SMRF), R is taken as **2.5**

Here **Ti** = **4.03** sec

Site has **Medium Soil**

Damping = 5 %

Here the time period is greater than 4.0 sec and as per Cl.4.5.3. of Proposed draft We can use the same expression, which is applicable for 4.0 seconds. So

Sa/g = 1.36/ T = 1.36 / 4.03 = **0.34**

Hence, (Sa/g)i = 0.34 ……………………………….....(IS 1893) (Part I) Figure 2

$(Ah)_i = \frac{Z}{2}\frac{I}{R}\left(\frac{Sa}{g}\right)_i =$ $(Ah)_i = \left(\frac{0.16}{2}\right)\left(\frac{1.5}{2.5}\right)(0.34)$ = **0.01632**

…………..... (Sections 4.5 and 4.5.1)

For Convective mode, value of Response reduction factor is taken as **2.5**

For Convective mode as per Cl 4.5.1

T_c = **5.56** sec

Site has Medium Soil

Damping = 0.5%

Here the time period is greater than 4 seconds and as per Cl.4.5.3. of Proposed draft

Code, we can use the same expression, which is applicable for 4.0 seconds. So

(Sa/ g) = 1.36/ T = 1.36/ 5.56 = **0.2446**, this value is for Damping factor 5.0 %

Multiplying factor of 1.75 is to be used to obtain Sa/g values for 0.5 % Damping ……..... (Section 4.6.2)

(Ah) $_c$= (0.16/2) x (1.5/2.5) x (0.2446) x1.75

= **0.02054**

Base shear

Base shear art the bottom of staging in Impulsive mode,

$V_i = (A_h)_i(m_i + m_s)g$ = $V_i = (0.01632)_i(571385.59 + 1106693.31)9.81$= **268.66** kN

…………..... (Section 4.7.2)

Similarly, base shear in Convective mode,

$V_C = (A_h)_c\, m_c g$ = $(0.02054)_C$ x1246178.6x9.81/1000 = **251.1** kN

Total base shear at the bottom of staging,

$V = \sqrt{V_i^2 + V_c^2}$ $= \sqrt{(268.66)^2 + (251.1)^2}$ = **367.73** kN

Base Moment

Overturning moment at the base of staging in Impulsive mode

$M_i^* = (A_h)_i [m_i(h_i^* + h_s) + m_s h_{cg}] g$ $= (0.01632)_i x[571385.59(9.07 + 31.60) + 1106693.31x35.13]x9.81$

= **M_i^*** = **9944.784** kNm ……….. (Section 4.7.2)

Similarly overturning moment in Convective mode,

= $M_c^* = (A_h)_c m_c (h_c^* + h_s) g$ $= (0.02054)_c x1246178.6x(8.10 + 31.60)x9.81$ = **9968.74** kNm

Total overturning moment

$M = \sqrt{M_i^{*2} + M_c^{*2})}$ = $\sqrt{(9944.78)^2 + (9968.74)^2}$ = **14080.99 kNm**

Sloshing wave height

Maximum sloshing wave height,

$d_{max} = (A_h)_c R \frac{D}{2}$ = $(0.02054)_c 2.05 \frac{21}{2}$ = 0.54 m ………… (Section 4.11)

Height of sloshing wave is more than free board of 0.3 m

Analysis for Tank Empty Condition

For empty condition tank will be considered as

Single degree of freedom system as described in Section 4.7.4

Time period of Impulsive mode,

$T = T_e = 2\pi \sqrt{\frac{m_s}{K_s}}$ = $2\pi \sqrt{\frac{1106693.31}{4075810}}$ = **3.27** sec.

Empty tank will not have convective mode of vibration.

Design Horizontal Seismic Coefficient

Design horizontal Seismic Coefficient corresponding to

Impulsive time period **Ti** = **3.27** sec.

$(A_h)_i = \frac{Z}{2} \frac{I}{R} \left(\frac{Sa}{g}\right)_i$…………… (Section 4.5 and 4.5.1)

Where, Z = **0.16** …... (IS 1893: part I):Table2; Zone IV)

I = **1.5**

Since Staging has Special Moment Resisting Frames (SMRF), R is taken as 2.5

Here Ti= **3.27** sec

Site has **Medium soil**

Damping = 5 %

Hence, (Sa/g)i = 1.36 / 3.24 = **0.416** ……… (IS 1893) (Part I) Figure 2

(Ah)i = $\left(\frac{0.16}{2}\right)\left(\frac{1.5}{2.5}\right)(0.416)_i$ = **0.019968**

Base shear

Base shear art the bottom of staging in Impulsive mode,

$V = V_i = (A_h)_i m_s g$ = $(0.019968)_i x 1106693.31 x 9.81$ = **216.78** kN

………………... (Section 4.6.2)

Base Moment

Total base moment,

$M^* = (A_h)_i m_s h_{cg} g$ = $(0.019968)_i x 1106693.31 x 35.13 x 9.81$ …………… (Section 4.7.3)

= **7615.68** kNm

Since total base shear **367.73** kN and base moment **14080.99** kN-m in tank full condition are more than that total base Shear **216.78** kN and base moment **7615.68** kNm in tank empty condition,

So Design will be governed by Tank full condition.

Summary:

Tank Full	**Base Shear in kN**	**Base Moment in kNm**	**Base Shear Net**	**Base Moment Net**	**Time Period**
Impulsive	268.66	9944.784	**367.73**	**14080.99**	**4.03**
Convective	251.10	9968.74			**5.56**
Tank Empty	**Base Shear in kN**	**Base Moment in kNm**	**Base Shear Net**	**Base Moment Net**	**Time Period**
Impulsive	216.78	7615.68	**216.78**	**7615.68**	**3.27**
Convective	—	—			—

4.3.2 SEISMIC ANALYSIS AS PER IS 1893-1984

FOR PERIPHERALLY BRACED WATER TANK TRESTLE

TANK FULL CONDITION

Calculation of C G of water tank from top of the footing

h = Staging height + C.G. of water container

= **34.878** m

Design horzontal Seismic Coefficients

As per seismic coefficient method

alpha h = ßxIxαo

ß = **1** .....For raft foundation Table 3

where Fo = **0.2** .....IS 1893-1984 (Part I) Table2;Zone III

I = **1.5** ....For Water tanks Table 4

Here T = 2 x 3.14 x (3005807.2/ (4075810))^0.5 = **5.39** sec

Site has Medium Soil so Sa/g= **0.04**

αh= β x I x Fo x Sa/g = 1*0.2*1.5*0.04

= **0.012**

W = weight of container+0.33x weight of staging + weight of water

9822.1+1/3 x 3096.3+18633

=29487.2kN

= 3005807.2 kg

Base shear =αhW = 0.012 x 3005807.2 kg

= 36070 kg = **353.8** kN

Moment at base of staging = 353.8 x 34.878 = **12338** kNm

TANK EMPTY CONDITION

ß = **1** .....For raft foundation Table 3

where Fo = **0.2** .....IS 1893-1984 (Part I) Table2;Zone III

I = **1.5** ....For Water tanks Table 4

here,

T = sec =2 x 3.14x (1106693.31)^0.5 /(4075810)^0.5

= **3.27** sec

Site has Medium Soil

Damping = 5 %

Hence , (Sa/g) = **0.04**

(IS 1893-1984) (Part I) Figure 2

(αh) = **0.012**

W = **10843.879**

1106693 kg

Base shear αhW = **130.3** kN

Moment at base of staging = **4544** kN-m

So Design will be governed by tank full condition.

4.3.3 COMPARISON BETWEEN EXISTING & DRAFT CODE

	IS 1893-2002 PROPOSED DRAFT		IS1893-1984	
	Base Moment (kNm)	Base Shear (kN)	Base Moment (kNm)	Base Shear (kN)
Design values	**14080.99**	**367.73**	**12344.44**	**353.93**
Impulsive	**9944.78**	**268.66**	**12344.44**	**353.93**
Convective	**9968.74**	**251.10**	—	
% COMPARISION WITH IS 1893-1984	**114.07**	**103.90**	**100**	**100**
Design Governing Case	**Tank Full Condition**		**Tank Full Condition**	
Response Reduction Factor	**R = 2.5**		**β = 1.0, Fo = 0.2**	
Tank Full Ti sec	**4.03**		**5.40**	
Tank Full Tc sec	**5.56**		—	
Tank Empty Te sec	**3.27**		**3.27**	

4.3.4 WIND ANALYSIS AS PER IS 875-2005 PROPOSED DRAFT CODE

Wind data:

1 Wind zone **3** Vb = **44** m/s
(IS: 875-pt.3,Sec.5.2,Fig. 1)

2 Terrain catregory **B** Open terrain with wellscattered structures
Category 2, as defined in IS 875
(IS: 875-pt.3,Sec.5.3.2.1)

Design Factors

Risk Coefficient factor k_1 = **0.91**
(IS: 875-pt.3,Sec.5.3.1,Table-1)

Terrain and Height factor k_2, varies with height and is given in Table 22.1
(IS: 875-pt.3,Sec.5.3.2.2,Table-2)

Topography Factor k3 = **1** Soil Slope < 3°
(IS: 875-pt.3,Sec.5.3.3.1)

Importance Factor for cyclonioc region k4 = **1.3**
(Important structure after cyclone) (IS: 875-pt.3,Sec. 5.3.4)

Wind directinality Factor kd = **1** for staging and also cyclone affected area
(IS: 875-pt.3,Sec. 6.1.1, cl. 5.4.1)

Area averaging Factor ka = **1** for staging
= **0.8** for Tank Portion

Tributory area = **134.6** m^2 (IS: 875-pt.3,Sec. 6.1.2, Table 4)

Design Wind Pressure

Design Wind speed = Vz = $VbxK_1xK_2xK_{3x}k4$
= **52.05** x k_2 m/s (IS: 875-pt.3,Sec. 5.3)

Pz =0.6* (Vz)^2 = **1626** x k_2

Pd = Pz * ka * kd (IS: 875-pt.3,Sec. 5.4, sec 6.1)

Levels	Height upto	K2	Design Velocity	Design Pressure	Level 31.35	Column Nos	Bottom Ring Beam
1st Level	**0.85**	1.000	52.05	1.15		7	
2nd Level	**5.19**	1.000	52.05	1.15	26.89		7th Bracing
3rd Level	**9.53**	1.000	52.05	1.15		6	
	10	1.000	52.05	1.15	22.55		6th Bracing
4th Level	**13.87**	1.039	54.07	1.19		5	
	15	1.050	54.65	1.21	18.21		5th Bracing
5th Level	**18.21**	1.063	55.32	1.22		4	
	20	1.070	55.70	1.23	13.87		4th Bracing
6th Level	**22.55**	1.083	56.36	1.24		3	
7th Level	**26.89**	1.104	57.49	1.27	9.53		3rd Bracing
	30	1.120	58.30	1.29		2	
Top of Trestle	**31.35**	**1.123**	58.47	1.29	5.19		2nd Bracing
Bottom of cyli	37.8	**1.140**	**59.31**	1.31		1	
Top of cylinde	39.85	**1.145**	**59.58**	1.31	0.85		1st Brace at Plinth
Top of roof	41.95	**1.155**	**60.14**	1.33	0		Ground level
	50	1.170	60.90	1.34	-2		Foundation

Wind Load Calculations:

External Pressure coefficient for roof and bottom of tank :

(z/H) - 1 = (41.95/9.6) - 1 = **3.369**

(IS: 875-pt.3,Sec.6.2.2.8,Table 14)

H/D = **0.4571**

Therefore, Cpe = -0.75 for roof and -0.81 for bottom

Eccentricity of force at rc = 0.1*(21) = **2.1** m

Total force acting on the roof of the structure

P = $0.785 \times D^2 \times$ (pi - Cpe x Pd)

= **344.31** kN acting upwarc **2.10** m from center of dome

(IS: 875-pt.3,Sec.6.2.2.9)

Note: If no opening exist; like in RCC tanks, pi = 0

Roof pressure will be used with Gravity loads for design of dome. (IS: 875-pt.3,Sec.6.2.2.8)

Overall Horizontal Force on the tank :

$F = C_f \times Ae \times P_d$ (or P_z)

(IS: 875-pt.3,Sec.6.3,6.3.3.1 (c))

No horizontal force will act on top dome. The effect of wind pressure on dome has been included with an eccentricity.

Cylindrical Portion:

V_z(avg) = (58.47+ 59.58)/2

= **59.03** m/s

Vz x b = **1240** > 6

h/b **0.098** < 0.5

Therefore , Cf = **0.7** from Table 20 (rough) and

Pd =KaxKdxPz so for top Pd = 1*0.8*Pz and at staging = 1* 1* Pz

Pz = **1.49** kN/m^2 at top an **1.48** kN/m^2 at bottom

This being a very small difference, heigher value may be taken.

Fcylinder = 0.7 x 21 x 1.49 = **21.90** kN/m height

Conical Bottom:

Vd (avg) = (59.31+58.47) / 2 = **58.89** m/s

Vd x b = **989.41 and** **989.41 > 6**

h/b = **0.260 < 2**

Therefore , Cf = **0.7** from Table 20 (rough)

and Pz = **1.48** kN/m^2 at top an **1.46** kN/m^2 at bottom.

This is being a very small difference , heigher value may be taken

F conical dome = 0.7 x ((21-12.6)/2+12.6) x **1.48**

= **17.40** kN/m height

Staging :

Pd =	**1.15** kN/m^2	upto 10 m for 1st ,2nd and 3rd Level Bracings			
	1.19 kN/m^2	at	13.87	m	4th Level Bracings
	1.22 kN/m^2	at	18.21	m	5th Level Bracings
	1.24 kN/m^2	at	22.55	m	6th Level Bracings
	1.27 kN/m^2	at	26.89	m	7th Level Bracings

In order to calculate wind force on each column,each column is considered as an individual member (IS875-pt.3,Sec. 6.3.3.1 c,table 20) and no sheilding effect is considered on leeward columns, as the columns are placed far apart on periphery only)

Perimeter length : **12.6** m

No. of columns in periphery Nc = **8**

Therefore for one column:

Width of column **0.5** m

Depth of column **0.8** m

Vd x b = 52.33 x 0.. **29.45 > 6**

h/b = 61.7 and **61.7 > 20**

Therefore , Cf = **1.2** from table 20 for rough surface finish

Fcolumn = 1.2 x 0.5 x 1.15

= **0.6886** kN/m height upto 10 m height for 1 and 2 nd column

Fcolumn = 1.2 x 0.5 x 1.19

= **0.7153** .... kN/m for **13.87** m height 3rd level column

Fcolumn = 1.2 x 0.5 x 1.22

= **0.7153** .... kN/m for **18.21** m height 4th level column

Fcolumn = 1.2 x 0.5 x 1.24

= **0.7456** .... kN/m for **22.55** m height 5th level column

Fcolumn = 1.2 x 0.5 x 1.27

= **0.7606** .... kN/m for **26.89** m height 6th level column

Fcolumn = 1.2 x 0.5 x 1.29

= **0.7736** .... kN/m for **31.35** m height 7th level column

Fbracings = 1.0 x { 2 x (12.6- 0.5*5)} x **1.15**

= **23.1845** kN/m……….. height acting upto 3rd brace level

Fbracings = 1.0 x { 2 x (12.6- 0.5*5)} x **1.19**

= **24.0817** kN/m………….height acting at 4th brace level

Fbracings = 1.0 x { 2 x (12.6- 0.5*5)} x **1.22**

= **24.6414** kN/m………… height acting at 5th brace level

Fbracings = 1.0 x { 2 x (12.6- 0.5*5)} x **1.24**

= **25.1030** kN/m…………. height acting at 6th brace level

Fbracings = 1.0 x { 2 x (12.6- 0.5*5)} x **1.27**

= **25.6061** kN/m ………….height acting at 7th brace level

This is calculated considering it as an individual member and using table 23 with h/b ratio <2 ; F ring beam = 1x 12.6 x 1.29

= **16.2458** kN/m height as above

4.3.5 WIND ANALYSIS AS PER IS 875-1987

Wind data:

1 Wind zone : **3** Vb = **44** m/s

(IS: 875-pt.3,Sec.5.2,Fig. 1)

2 Terrain catregory : **B** Open terrain with wellscattered structures

Category 2, as defined in IS 875

(IS: 875-pt.3,Sec.5.3.2.1)

Design Factors

Risk Coefficient factor k_1 = **1.07**

(IS: 875-pt.3,Sec.5.3.1,Table-1)

Terrain and Height factor k_2, varies with height and is given in Table 22.1

(IS: 875-pt.3,Sec.5.3.2.2,Table-2)

Topography Factor k3 = **1** Soil Slope < 3°

(IS: 875-pt.3,Sec.5.3.3.1)

Design Wind Pressure

Design Wind speed = Vz = $V_b \times K_1 \times K_2 \times K_3$

= **47.08** x k_2 (IS: 875-pt.3,Sec. 5.3)

Levels	Height upto	K_2	Design Velocity	Design Pressure
1st Level	**0.85**	0.980	46.14	1.02
2nd Level	**5.19**	0.980	46.14	1.02
3rd Level	**9.53**	0.980	46.14	1.02
	10	0.980	46.14	1.02
4th Level	**13.87**	1.011	47.60	1.05
	15	1.020	48.02	1.06
5th Level	**18.21**	1.039	48.93	1.08
	20	1.050	49.43	1.09
6th Level	**22.55**	1.063	50.03	1.10
7th Level	**26.89**	1.084	51.06	1.13
	30	1.100	51.79	1.14
Top of Trestle	**31.35**	**1.103**	51.95	1.15
Bottom of cylinder	37.8	**1.120**	**52.71**	1.16
Top of cylinder	39.85	**1.125**	**52.95**	1.17
Top of roof	41.95	**1.135**	**53.46**	1.18
	50	1.150	54.14	1.19

Level	**Column Nos**	Bottom
31.35		Ring Beam
	7	
26.89		7th Bracing
	6	
22.55		6th Bracing
	5	
18.21		5th Bracing
	4	
13.87		4th Bracing
	3	
9.53		3rd Bracing
	2	
5.19		2nd Bracing
	1	
0.85		1st Brace at Plinth
0		Ground level
-2		Foundation

Wind Load Calculations:

External Pressure coefficient for roof and bottom of tank :

(z/H) - 1 = (41.95/9.6) - 1 = **3.369**

(IS: 875-pt.3,Sec.6.2.2.9,Table 19)

H/D = **0.4571**

Therefore, Cpe = -0.75 for roof and -0.81 for bottom

Eccentricity of force at roof

= 0.1*(21) = **2.1** m

Total force acting on the roof of the structure

P = 0.785 x D^2 x (pi - Cpe x Pd)

= **306.03** kN acting upwards at **2.10** m from center of dome

(IS: 875-pt.3,Sec.6.2.2.9)

Note: If no opening exist; like in RCC tanks, pi = 0

Roof pressure will be used with Gravity loads for design of dome.

Overall Horizontal Force on the tank :

F = C_f x Ae x P_d (or P_z) (IS: 875-pt.3,Sec.6.3,6.3.3.1 (c))

No horizontal force will act on top dome. The effect of wind pressure on dome has been included with an eccentricity.

Cylindrical Portion:

V_d(avg) = (52.71+52.95)/2

= **52.83** m/s and Vd x b = **1109.4** > 6

h/b = **0.098** < 0.5

Therefore , Cf = **0.7** from Table 23 (rough) and

Pz = **1.17** at top and **1.16** at bottom

This being a very small difference, heigher value may be taken.

Fcylinder = 0.7 x 21 x 1.17 = **17.20** kN/m height

Conical Bottom:

Vd (avg) = (52.71+51.95) / 2 = **52.33** m/s

Vd x b = **879.08 and** **879.08 > 6**

h/d = **0.260 < 0.5**

Therefore , Cf = **0.7** from Table 23 (rough)

and Pz = **1.16** at top and **1.15** at bottom.

This is being a very small difference , heigher value may be taken

Fconical dome = 0.7 x ((21-12.6)/2+12.6) x 1.17

= **13.76** kN/m height

Staging :

Pd =	**1.02** kN/m^2	upto 10 m for 1st ,2nd and 3rd Level Bracings				
	1.05 kN/m^2	at	**13.87**	m	4th Level	Bracings
	1.08 kN/m^2	at	**18.21**	m	5th Level	Bracings
	1.10 kN/m^2	at	**22.55**	m	6th Level	Bracings
	1.13 kN/m^2	at	**26.89**	m	7th Level	Bracings

In order to calculate wind force on each column,each column is considered as an individual member (IS875-pt.3,Sec. 6.3.3.1 c,table 23)and no sheilding effect is considered on leeward columns, as the columns are placed far apart on periphery only)

Perimeter length : **12.6** m

No. of columns in periphery Nc = **8**

Therefore for one column:

Width of column = **0.5** m

Depth of column = **0.8** m

Vd x b = 52.33 x C **26.16 > 6**

h/b = 61.7 and **61.7 > 20**

Therefore , Cf = **1.2** from table 23 for rough surface finish

Fcolumn = 1.2 x 0.5 x 1.02

= **0.6104** kN/m height upto 10 m height for 1 and 2 nd column

Fcolumn = 1.2 x 0.5 x 1.05

= **0.6297** .... kN/m for **13.87** m height 3rd level column

Fcolumn = 1.2 x 0.5 x 1.08

= **0.6297** .... kN/m for **18.21** m height 4th level column

Fcolumn = 1.2 x 0.5 x 1.10

= **0.6620** .... kN/m for **22.55** m height 5th level column

Fcolumn = 1.2 x 0.5 x 1.13

= **0.6755** ……… kN/m for **26.89** m height 6th level columı

Fcolumn = 1.2 x 0.5 x 1.15

= **0.6873** ……… kN/m for **31.35** m height 7th level columı

Fbracings = 1.0 x { 2 x (12.6- 0.5*5)} x 1.02

= **20.5505** kN/m……….. height acting upto 3rd brace level

Fbracings = 1.0 x { 2 x (12.6- 0.5*5)} x 1.05

= **21.1997** kN/m………….height acting at 4th brace level

Fbracings = 1.0 x { 2 x (12.6- 0.5*5)} x 1.08

= **21.7932** kN/m………… height acting at 5th brace level

Fbracings = 1.0 x { 2 x (12.6- 0.5*5)} x 1.10

= **22.2858** kN/m…………. height acting at 6th brace level

Fbracings = 1.0 x { 2 x (12.6- 0.5*5)} x 1.13

= **22.7408** kN/m ………….height acting at 7th brace level

This is calculated considering it as an individual member and using table 20 with h/b ratio <2

F ring beam = 1x 12.6 x 1.15 = **14.4324** kN/m height as above

Summary of forces and total loads on tank

Element	Force per unit Ht	Height of element in m	Top horizontal force	CG of force from Ground	Moment in kNm
Cylindrical wall	17.20	2.05	35.26	38.83	1368.89
Conical dome	13.76	2.10	28.89	36.75	1061.87
Top ring beam	14.43	0.50	7.22	39.60	285.76
7th column	0.69	4.34	2.98	29.18	87.04
6th column	0.68	4.34	2.93	24.84	72.82
5th column	0.66	4.34	2.87	20.50	58.89
4th column	0.63	4.34	2.73	16.16	44.16
3rd column	0.61	4.34	2.65	11.82	31.31
2nd column	0.61	4.34	2.65	7.48	19.82
1st column	0.61	4.34	2.65	3.14	8.32
7th Bracing	22.74	0.50	11.37	26.89	305.75
6th Bracing	22.29	0.50	11.14	22.55	251.27
5th Bracing	21.79	0.50	10.90	18.21	198.43
4th Bracing	21.20	0.50	10.60	13.87	147.02
3rd Bracing	20.55	0.50	10.28	9.53	97.92
2nd Bracing	20.55	0.50	10.28	5.19	53.33
1st Bracing	20.55	0.50	10.28	0.85	8.73
					4101.3

SUMMARY OF WIND FORCES AND TOTAL LOADS ON TANKS BY EXISTING AND DRAFT CODE

			IS 875-1987			IS 875-2005 PROPOSED DRAFT			
Element	Height of element in m	CG of force from Ground	Force per unit Ht old	Top horizontal force	Moment in kNm	Force per unit Ht draft	Top horizontal force draft	Moment in kNm	Increase in forces
Cylindrical portion	2.05	38.83	17.20	35.26	1368.89	21.90	44.90	1743.29	**27.35**
Conical dome	2.10	36.75	13.76	28.89	1061.87	17.40	36.55	1343.22	**26.50**
Top ring beam	0.50	39.60	14.43	7.22	285.76	16.25	8.12	321.67	**12.56**
7th column	4.34	29.18	0.69	2.98	87.04	0.77	3.36	97.97	**12.56**
6th column	4.34	24.84	0.68	2.93	72.82	0.76	3.30	81.99	**12.60**
5th column	4.34	20.50	0.66	2.87	58.89	0.75	3.24	66.34	**12.64**
4th column	4.34	16.16	0.63	2.73	44.16	0.72	3.10	50.17	**13.59**
3rd column	4.34	11.82	0.61	2.65	31.31	0.69	2.99	35.33	**12.82**
2nd column	4.34	7.48	0.61	2.65	19.82	0.69	2.99	22.36	**12.82**
1st column	4.34	3.14	0.61	2.65	8.32	0.69	2.99	9.38	**12.82**
7th Bracing	0.50	26.89	22.74	11.37	305.75	25.61	12.80	344.27	**12.60**
6th Bracing	0.50	22.55	22.29	11.14	251.27	25.10	12.55	283.04	**12.64**
5th Bracing	0.50	18.21	21.79	10.90	198.43	24.64	12.32	224.36	**13.07**
4th Bracing	0.50	13.87	21.20	10.60	147.02	24.08	12.04	167.01	**13.59**
3rd Bracing	0.50	9.53	20.55	10.28	97.92	23.18	11.59	110.47	**12.82**
2nd Bracing	0.50	5.19	20.55	10.28	53.33	23.18	11.59	60.16	**12.82**
1st Bracing	0.50	0.85	20.55	10.28	8.73	23.18	11.59	9.85	**12.82**
					4101.33			4970.88	

4.3.6 DESIGN OF PERIPHERALLY BRACED TRESTLE

DESIGN OF COLUMN SECTION

for 8 no. of column

Grade of concrete = 25 N/mm^2

Ultimate load Pu = 7353.45 kN

Ulitmate Moment Mu= 187.84 kNm

assumed bar dia. 25 mm

Main steel

Assumed b = 550 mm & D = 800 mm with d' in mm = 40

Ratio (d' /D) = **0.05**

Pu/(fck*b*D) = **0.668**

&

$Mu/(fck*b*d^2)$ = **0.0237**

Referring interaction chart corresponding to fy = 415 N/mm^2

and (d'/D) = **0.07**

thus

(p/fck) = 0.09 so p = 2.25 %

A = (pbD/100) = **9900** mm^2

Provide **25** mm dia distributing on all sides equally

Provide **20.1783** ≈ 16 on two faces and as shown on other two.

≈ 22 no.s

Transverse steel

using 8 mm dia ties, spacing of least of following

1. Least Lateral dimention = **550** mm
2. 16* dia of main steel = **400** mm
3. 48* dia of ties = **384** mm

Grade of steel = 415 N/mm^2

Provide **8** mm dia ties at **384** ≈ 300 mm c/c.

DESIGN OF BRACES

for 191peripheral bracing

Mu = 117.29 kNm

Vu = 50.54 kN

Section of brace is b = 250 mm

D = 400 mm

d = 360 mm

So Mu. Lim = $(0.148*fck*b*d^2)$ = **119.88** kNm

Thus Ast = [0.36*fck*b*(0.53d)/(0.87*fy)]

= **1189** mm^2

Use **3.78673** ≈ 4 bars of 20 mm dia each side

ζv = (Vu/bd) = 0.56156 N/mm^2

shear stress

1.25	0.7
1.5	0.74
1.3956	**0.723**

(100*Ast/(b*d)) = **1.396**

from Is 456-2000, Permissible shear stress

ζc = 0.723289 N/mm^2 > **0.561556** **Safe**

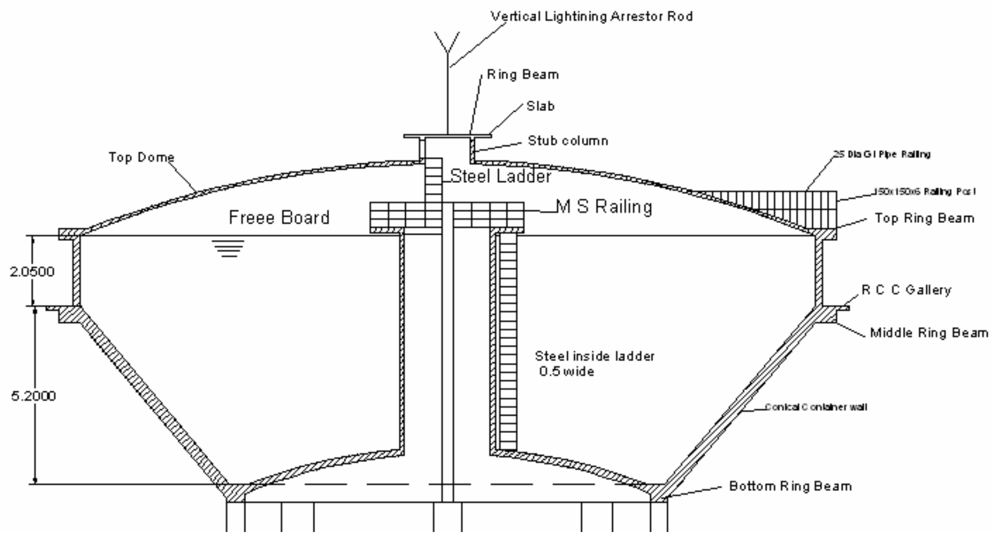
so using 8 mm dia 4 legged No.s stirrups as nominal shear reinforcements, the spacing is given by

Sv = (Asv*fy/(0.4*b)) = **208.5** ≈ 300 mm, centers

SUMMARY:	**Column**			
Main steel :	**25**	mm dia bars total	**22**	no.s
		equally distributed on all faces		
Transverse steel:	**8**	mm dia bars at	**300**	mm centers
	Peripheral Bracings			
Main steel :	**8**	No.s of mm dia and bottom equally distributed	**20**	bars on top
Transverse steel:	**8**	mm dia bars at	**250**	mm centers

4.4 DIAGONALLY AND PERIPHERALLY BRACED TRESTLE

4.4.1 SEISMIC ANALYSIS AS PER PROPOSED DRAFT CODE



Mass of the container only: = 9822.10 kN = 1001491 Kg

Mass of water in container, m = 18633 kN = 1899881.16 Kg

Mass of columns: = 2560 kN

Mass of braces: = 536.27 kN

Mass of the staging: = 3096.27 kN = 315708.0743 Kg

Total mass of full tank: = 31551.23 kN = 3217089.935 Kg

Total mass of empty tank: = 12918.37 kN = 1317208.77 Kg

Mass of container and one third mass of staging is expressed as

ms = 9822.10 + (3096.27)/3

= 10854.19 kN = **1106693.31 Kg**

Lateral stiffness of staging which id only Peripherally braced

Ks = 8181.29 kN/m = **8181290 N/m**

CG of empty container

= 3.528 m from top of bottom ring beam

Ec Modulus of Elasticity of concrete

= 25000 N/mm^2 = 25 kN/mm^2 = 25000000 kN/m^2

Top Dome Weight = 1027.51 kN

Top Ring Beam Weight = 196.93 kN

Cylindrical Wall = 669.70 kN

Bottom Ring Beam = 247.99 kN

Circular Ring Beam = 430.01 kN

Bottom Dome	=	664.60 kN
Conical Dome	=	6560.83 kN
Water	=	18632.86 kN
Columns	=	2560.00 kN
Bracings	=	536.27 kN

Centre of Gravity of Empty Container from bottom slab

= 3.52 m

Mass **m** = mass of water = **1899888.26 kg**

Let h be the height equivalent circular cylinder

$$\pi\left(\frac{D}{2}\right)^2 h = 1899.88$$

So **h** $= 1899.88 \text{x} \dfrac{4}{21^2 \text{x} \pi}$ = **5.48** m

Inner diameter of tank, **D** = **21** m, h/D = 5.48/21 = **0.26** so,

$$\frac{mi}{m} = \frac{\tanh 0.866 \frac{D}{h}}{0.866 \frac{D}{h}} = \frac{\tanh 0.866 \frac{21}{5.48}}{0.866 \frac{21}{5.48}} = 0.300746$$

$\mathbf{m_i}$ = 1899888.26 x 0.300746

= **571385.59 kg**

hi/ h = 0.375 < 0.75 so **hi** = 0.375 x 5.48 = **2.06** m

$$\frac{hi^*}{h} = \frac{0.866 \frac{D}{h}}{2 \tanh 0.866 \frac{D}{h}} = \frac{0.866 \frac{21}{5.48}}{2 \tanh 0.866 \frac{21}{5.48}} = 1.65 \text{ for } h/D < 1.33$$

So $\mathbf{hi^*}$ = 1.65 x 5.48 = **9.07 m**

$$\frac{mc}{m} = 0.23 \frac{\tanh 3.68 \frac{h}{D}}{\frac{h}{D}} = 0.23 \frac{\tanh 3.68 \frac{5.48}{21}}{\frac{5.48}{21}} = 0.66$$

so $\mathbf{m_c}$ = 0.66 x 1899888.26 = **1246178.6 kg**

$$\frac{hc}{h} = 1 - \frac{\cosh\left(3.68 \frac{h}{D}\right) - 1.0}{3.68 \frac{h}{D} \sinh\left(3.68 \frac{h}{D}\right)} = 1 - \frac{\cosh\left(3.68 \frac{5.48}{21}\right) - 1.0}{3.68 \frac{5.48}{21} \sinh\left(3.68 \frac{5.48}{21}\right)} = 0.54$$

so $\mathbf{h_c}$ = 0.54 x 5.48 = **2.94** m

$$\frac{hc^*}{h} = 1 - \frac{\cosh\left(3.68\frac{h}{D}\right) - 2.01}{3.68\frac{h}{D}\sinh\left(3.68\frac{h}{D}\right)} = 1 - \frac{\cosh\left(3.68\frac{5.48}{21}\right) - 2.01}{3.68\frac{5.48}{21}\sinh\left(3.68\frac{5.48}{21}\right)} = 1.48$$

so $\mathbf{hc^*}$ = 1.48 x 5.48 = **8.10** m

$$\mathrm{k}c = 0.836\frac{mg}{h}\tanh^2\left(3.68\frac{h}{D}\right) = \mathrm{k}c = 0.836\frac{1899888.26\mathrm{x}9.81}{5.48}\tanh^2\left(3.68\frac{5.48}{21}\right)$$

$\mathbf{K_c}$ = **1575780.14** N/mm

$\mathbf{h_s}$ = **31.60** m ……………….....Structural height of staging, measured from top of foundation to the bottom of container wall

$\mathbf{h_{cg}}$ = **35.13** m ……………....Height of center of gravity of the empty container of elevated tank, measured from base of staging

Time period:

Time period of Impulsive mode,

$$T_i = 2\pi\sqrt{\left(\frac{(m_i + m_s)}{K_s}\right)} = 2\pi\sqrt{\left(\frac{(571385.59 + 1106693.31)}{8181290}\right)} = \mathbf{2.844}\ \text{sec}$$

Time period of Convective mode,

For h/D = 0.26, Cc = 3.8 from Graph

$$T_c = C_C\sqrt{\frac{D}{g}} = 3.8\sqrt{\frac{21}{9.81}} = \mathbf{Tc} = \mathbf{5.56}\ \text{sec}$$

Design horizontal Seismic Coefficients

Design horizontal seismic coefficient for impulsive mode,

$$(Ah)_i = \frac{Z}{2}\frac{I}{R}\left(\frac{Sa}{g}\right)_i$$

Where, Z = **0.16** …….. (IS 1893, PART (I), Table2, Zone III)

I = **1.5**

Since Staging has Special Moment Resisting Frames (SMRF), R is taken as **2.5**

Here **Ti** = **2.844** sec

Site has **Medium Soil**

Damping = 5 %

Here the time period is greater than 4.0 sec and as per Cl.4.5.3. of Proposed draft we can use the same expression, which is applicable for 4.0 seconds. So

Sa/g = 1.36/ T = 1.36 / 2.844 = **0.4782**

Hence, $(Sa/g)_i$ = 0.4782

$$(Ah)_i = \frac{Z}{2}\frac{I}{R}\left(\frac{Sa}{g}\right)_i = (Ah)_i = \left(\frac{0.16}{2}\right)\left(\frac{1.5}{2.5}\right)(0.4782)$$ = **0.0229536**

…………... (Sections 4.5 and 4.5.1)

For Convective mode, value of Response reduction factor is taken as **2.5**

For Convective mode as per Cl 4.5.1

Tc = **5.56** sec

Site has Medium Soil

Damping = 0.5%

Here the time period is greater than 4 seconds and as per Cl.4.5.3. of Proposed draft Code, we can use the same expression, which is applicable for 4.0 seconds. So

(Sa/ g) = 1.36/ T = 1.36/ 5.56 = **0.2446** This value is for damping 5 %

Multiplying factor of 1.75 is to be used to obtain Sa/g values for 0.5 %

Damping ………………….. (Section 4.6.2)

$(Ah)_c$= (0.16/2)x(1.5/2.5)x(0.2446)x1.75

= **0.02054**

Base shear

Base shear art the bottom of staging in Impulsive mode,

$V_i = (A_h)_i(m_i + m_s)g$ $= V_i = (0.0229536)_i(571385.59 + 1106693.31)9.81/1000$ = **377.86** kN …………………(Section 4.7.2)

Similarly, base shear in Convective mode,

$V_C = (A_h)_C m_s g$ = $(0.02054)_C x1246178.6x9.81$ = **251.1** kN

Total base shear at the bottom of staging,

$V = \sqrt{V_i^{\,2} + V_c^{\,2}}$ = $\sqrt{(377.861)^2 + (251.1)^2}$ = **453.68** kN

Base Moment

Overturning moment at the base of staging in Impulsive mode

$M_i^{\,*} = (A_h)_i[m_i(h_i^* + h_s) + m_s h_{cg}]g$ =

= $(0.0229536)_i x[571385.59(9.07 + 31.60) + 1106693.31x35.13]x9.81$

M_i^* = **13987.046** kNm ………. (Section 4.7.2)

Similarly overturning moment in Convective mode,

$M_c^* = (A_h)_c m_c (h_c^* + h_s) g$ = $(0.02054)_c x1246178.6x(8.10 + 31.60)x9.81$ = **9968.74** kNm

Total overturning moment

$M = \sqrt{M^*{}_i{}^2 + M^*{}_c{}^2)}$ = $\sqrt{(13987.046)^2 + (9968.74)^2}$ = **17175.95 kNm**

Sloshing wave height

Maximum sloshing wave height,

$d_{\max} = (A_h)_c R \frac{D}{2}$ = $(0.02054)_c 2.05 \frac{21}{2}$ = **0.54** m

.... (Section 4.11)

Height of sloshing wave is more than free board of 0.3 m

Analysis for Tank Empty Condition

For empty condition tank will be considered as

Single degree of freedom system as described in Section 4.7.4

Time period of Impulsive mode,

$T = T_e = 2\pi \sqrt{\frac{m_s}{K_s}}$ = $2\pi \sqrt{\frac{1106693.31}{8181290}}$ = **2.3097** sec.

Empty tank will not have convective mode of vibration.

Design Horizontal Seismic Coefficient

Design horizontal Seismic Coefficient corresponding to

Impulsive time period **Ti** = **2.3097** sec.

$(A_h)_i = \frac{Z}{2} \frac{I}{R} \left(\frac{Sa}{g} \right)_i$ .... (Section 4.5 and 4.5.1)

Where, Z = **0.16** .... (IS 1893: part I):Table2; Zone IV)

I = **1.5**

Since Staging has Special Moment Resisting Frames (SMRF), R is taken as 2.5

Here Ti= **2.3097** sec

Site has **Medium soil**

Damping = **5** %

Hence, (Sa/g)i = 1.36 / 2.3097 = **0.589** ...... (IS 1893) (Part I) Figure 2

$$\left(\frac{0.16}{2} \right) \left(\frac{1.5}{2.5} \right) (0.589)_i$$

$(Ah)_i$ = = **0.028272**

Base shear

Base shear art the bottom of staging in Impulsive mode,

$V = V_i = (A_h)_i m_s g$ $= (0.028272)_i x 1106693.31 x 9.81$ = **306.94** kN

………………... (Section 4.6.2)

Base Moment

Total base moment,

$M^* = (A_h)_i m_s h_{cg} g$ = $(0.028272)_i x 1106693.31 x 35.13 x 9.81$ …………… (Section 4.7.3)

= **10782.78** kNm

Since total base shear **453.68** kN and base moment **17175.95** kN-m in tank full Condition is more than that total base Shear **306.94** kN and base moment **10782.78** kNm in tank empty condition,

So Design will be governed by Tank full condition.

Tank Full	**Base Shear in kN**	**Base Moment in kNm**	**Base Shear Net**	**Base Moment Net**	**Time Period**
Impulsive	377.86	13987.04	**453.63**	**17175.95**	**2.844**
Convective	251.10	9968.74			**5.56**
Tank Empty	**Base Shear in kN**	**Base Moment in kNm**	**Base Shear Net**	**Base Moment Net**	**Time Period**
Impulsive	306.94	10782.78	**306.94**	**10782.78**	**2.3097**
Convective	—	—			—

ANALYSIS FOR HYDROSTATIC PRESSURE

It will be same for peripherally braced water tank as container remaining same for both cases.

4.4.2 SEISMIC ANALYSIS AS PER IS 1893-1984

TANK FULL CONDITION

Calculation of C G of water tank from top of the footing

h = Staging height + C.G. of water container

= **34.878** m

Design horzontal Seismic Coefficients

As per seismic coefficient method

α h = ßxIxFoxSa/g

ß = **1** …..For raft foundation Table 3

where Fo = **0.2** …..IS 1893-1984 (Part I) Table2;Zone III

I = **1.5** …....For Water tanks Table 4

Here T =2x3.14 x (3005807.2/ (8181290))^0.5 = **3.81 sec**

Site has Medium Soil so = Sa/g= **0.04**

αh= ß x I x Fo x Sa/g = 1*0.2*1.5*0.04 = **0.012**

W = weight of container+0.33x weight of staging + weight of water

= 9822.10+0.33x 3096.3+18633

=29487.2kN

= 3005807.2 kg

Base shear =αhW = 0.012 x 3005807.2 kg

= 36069.7 kg = **353.8** kN

Moment at base of staging = **12338** kNm

TANK EMPTY CONDITION

ß = **1** …..For raft foundation Table 3

where Fo = **0.2** …..IS 1893-1984 (Part I) Table2;Zone III

I = **1.5** …....For Water tanks Table 4

here,

T = sec = 2 x 3.14x (1106693.31)^0.5 /(8181290)^0.5

= **2.31** sec

Site has Medium Soil

Damping = 5 %

Hence , (Sa/g) = **0.05**

(IS 1893-1984) (Part I) Figure 2

(αh) = **0.015**

W = **10843.879**

= **1106693** kg

Base shear αhW = **162.8** kN

Moment at base of staging = **5680** kN-m

So Design will be governed by tank full condition.

4.4.3 COMPARISON BETWEEN EXISTING & PROPOSED DRAFT CODE

	IS 1893-2002 PROPOSED DRAFT		IS1893-1984	
	Base Moment (kNm)	Base Shear (kN)	Base Moment (kNm)	Base Shear (kN)
Design values	**17175.95**	**453.68**	**12338.00**	**353.80**
Impulsive	**13987.05**	**377.86**	**12338.00**	**353.80**
Convective	**9968.74**	**251.10**	—	
% INCREASE COMPARIS WITH IS 1893-1984	**139.21**	**128.23**	**100**	**100**
Design Governing Case	**Tank Full Condition**		**Tank Full Condition**	
Response Reduction Factor	**R = 2.5**		$\beta = 1.0$, $F_o = 0.2$	
Tank Full Ti sec	**2.85**		**3.81**	
Tank Full Tc sec	**5.56**		—	
Tank Empty Te sec	**2.31**		**2.31**	

4.4.4 WIND ANALYSIS AS PER PROPOSED DRAFT CODE

As the surface area obstructed by diagonals will be negligible so wind force calculation is assumed to be same as for peripherally braced trestle for simplicity.

4.4.5 WIND ANALYSIS AS PER IS 875-1987

As the surface area obstructed by diagonals will be negligible so wind force calculation is assumed to be same as for peripherally braced trestle for simplicity.

4.4.6 DESIGN OF PERIPHERALLY WITH DIAGONALLY BRACED TRESTLE

DESIGN OF COLUMN SECTION

for 8 no.s column

Grade of concrete =	25	N/mm^2	
Ultimate load Pu =	7670.72	kN	Factored values
Ulitmate Moment Mu=	144.06	kNm	
assumed bar dia.	25	mm	

Main steel

Assumed b = 550 mm & D = 800 mm with d' in mm = 40

Ratio (d' /D) = **0.05**

Pu/(fck*b*D) = **0.697** &

$Mu/(fck*b*d^2)$ = **0.0181**

Referring interaction chart corresponding to fy = 415 N/mm^2

and (d'/D) = **0.07**

thus

(p/fck) = 0.09 so p = 2.25 %

A = (pbD/100) = **9900** mm^2

Provide **25** mm dia distributing on all sides equally

Provide **20.1783** ≈ 16 on two faces and as shown on other two.

≈ 22 no.s

Transverse steel

using 8 mm dia ties, spacing of least of following

1. Least Lateral dimention = **550** mm
2. 16* dia of main steel = **400** mm
3. 48* dia of ties = **384** mm

Grade of steel = 415 N/mm^2

Provide **8** mm dia ties at **384** ≈ 300 mm c/c.

DESIGN OF BRACES

for 191 no. bracing

Mu =	79.65	kNm	Factored values			
Vu =	34.47	kN				
Section of brace is	b =	230	mm			
	D =	350	mm			
	d =	**310**	mm			
So Mu. Lim =	$(0.148*fck*b*d^2)$ =			**81.7811**	kNm	
Thus Ast =	[0.36*fck*b*(0.53d)/(0.87*fy)]					
=	**941.98**	mm^2				
Use	**1.91995**	≈	2	bars of	25	mm dia each side
ζv =	(Vu/bd) =	0.48345	N/mm^2			

(100*Ast/(b*d)) = **1.376**

1.25	0.7
1.5	0.74
ans	**0.72**

from Is 456-2000, Permissible shear stress

ζc = 0.720196 N/mm^2 > **0.48345** **Safe**

so using 8 mm dia 4 legged No.s stirrups as nominal shear reinforcements, the spacing is given by

Sv = (Asv*fy/(0.4*b)) = **226.63** ≈ 300 mm, centers

DESIGN OF TENSION MEMBER

DENSITY OF STEEL PROVIDEE	76.8123	kN/m^3				
length =	**6.34606**	m				
tensuion	75.02	kN	Factored values			
Permissible stress =	305.9	N/mm^2				
area required =	75*1000/159.6	=		**245.244**	mm^2	
using	20	mm dia bars having area	=	**314**	mm^2	
As Ast provided	> Ast require	so safe				
Stress in section =	**238.92**	N/mm^2	**<**	**305.9**	N/mm^2	**Safe**

4.4.7 DETAILING OF CONNECTION OF DIAGONAL BRACING WITH EXISTING CONCRETE MEMBERS

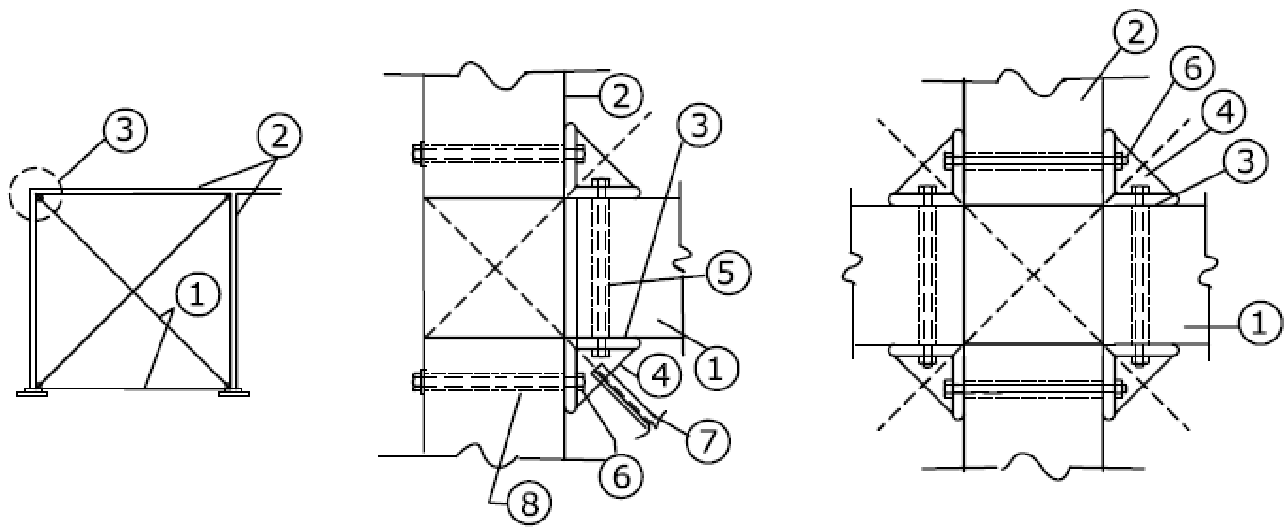


FIGURE 4.4 DETAILING FOR STEEL DIAGONAL X BRACINGS

(A) ARRANGEMENTS OF BRACINGS

1 New bracing members

2 Existing concrete structural elements

3 Joints of bracing members and existing RC elements

(B) TYPICAL CONNCETION DETAILS

1 Existing beam

2 Existing column

3 Structural steel angle cleat

4 Structural steel gusset plate

5 Bolt

6 Grout to be injected in the gap between concrete surface and steel surface

7 Bracing members

8 Gap in the hole for bolt to be filled by grout

4.5 DESIGN FORCE SUMMARY

(A) FOR COLUMNS

PERIPHERALLY BRACED TRESTLE

for columns

For axial force	Axial force	Shear @ Y in kNm	Shear @ Z in kNm	TORSION	Moment @ Y in kNm	Moment @ Z in kNm
8	958.39	0.00	20.64	0.00	125.23	0.00
18	930.54	0.00	17.60	0.00	86.35	0.00
28	883.37	0.00	14.89	0.00	53.47	0.00
38	828.07	0.00	14.14	0.00	39.48	0.00
48	769.97	0.00	13.94	0.00	31.10	0.00
58	712.41	0.00	13.71	0.00	22.87	0.00
68	658.78	0.00	14.53	0.00	11.61	0.00
78	613.61	0.00	6.13	0.00	11.73	0.00

For Mz torsion	Axial force	Shear @ Y in kNm	Shear @ Z in kNm	TORSION	Moment @ Y in kNm	Moment @ Z in kNm
2	0.00	67.14	0.00	1.93	0.00	333.40
12	0.00	70.20	0.00	2.20	0.00	268.99
22	0.00	72.95	0.00	2.76	0.00	211.08
32	0.00	73.77	0.00	3.66	0.00	183.37
42	0.00	74.03	0.00	4.94	0.00	168.15
52	0.00	74.23	0.00	6.47	0.00	153.96
62	0.00	72.81	0.00	7.69	0.00	128.48
72	0.00	79.66	0.00	7.66	0.00	78.28

PERIPHERALLY & DIAGONALLY BRACED TRESTLE

for columns

For axial force	Axial force	Shear @ Y in kNm	Shear @ Z in kNm	TORSION	Moment @ Y in kNm	Moment @ Z in kNm
8	1169.91	0.00	19.83	0.00	96.04	0.00
18	1142.12	0.00	12.31	0.00	52.02	0.00
28	1083.12	0.00	8.01	0.00	25.28	0.00
38	1013.78	0.00	7.24	0.00	18.93	0.00
48	941.92	0.00	7.11	0.00	16.01	0.00
58	870.60	0.00	7.00	0.00	12.71	0.00
68	802.83	0.00	7.37	0.00	7.50	0.00
78	743.24	0.00	3.38	0.00	3.89	0.00

For Mz torsion	Axial force	Shear @ Y in kNm	Shear @ Z in kNm	TORSION	Moment @ Y in kNm	Moment @ Z in kNm
2	24.06	63.53	2.58	2.58	6.62	258.92
12	51.30	45.56	0.09	2.92	0.87	159.37
22	58.08	39.47	0.18	3.60	0.13	107.60
32	58.78	37.77	0.11	4.67	0.04	93.56
42	58.07	37.45	0.13	6.10	0.03	89.75
52	56.96	38.00	0.09	7.73	0.02	87.98
62	54.69	39.27	0.42	8.98	0.12	82.52
72	46.58	50.74	0.23	8.81	1.29	63.21

(B) FOR BRACINGS

DESIGN FORCES FOR PERIPHERAL BRACINGS BY DRAFT CODE PROVISIONS

PERIPHERALLY BRACED TRESTLE

For Peripheral Bracings

For axial force	Axial force	Shear @ Y in kNm	Shear @ Z in kNm	TORSION	Moment @ Y in kNm	Moment @ Z in kNm
195	1.65	33.69	0.02	0.09	0.04	78.20
295	1.48	56.98	0.02	0.19	0.15	132.26
395	0.43	66.81	0.02	0.21	0.32	155.05
495	0.14	70.55	0.01	0.25	0.57	163.74
595	0.13	71.08	0.06	0.31	1.04	164.95
695	0.65	68.66	0.29	0.43	1.98	159.38
795	4.02	61.39	0.76	0.48	3.53	142.35

For Mz torsion	Axial force	Shear @ Y in kNm	Shear @ Z in kNm	TORSION	Moment @ Y in kNm	Moment @ Z in kNm
191	1.65	33.69	0.02	0.09	0.04	78.20
291	1.48	56.98	0.02	0.19	0.15	132.26
391	0.43	66.81	0.02	0.21	0.32	155.05
491	0.14	70.55	0.01	0.25	0.57	163.74
591	0.13	71.08	0.06	0.31	1.04	164.95
691	0.65	68.66	0.29	0.43	1.98	159.38
791	4.02	61.39	0.76	0.48	3.53	142.35

PERIPHERALLY & DIAGONALLY BRACED TRESTLE

For Peripheral Bracings

For axial force	Axial force	Shear @ Y in kNm	Shear @ Z in kNm	TORSION	Moment @ Y in kNm	Moment @ Z in kNm
195	29.89	22.70	0.02	0.04	0.08	52.93
295	52.55	33.38	0.05	0.09	0.23	77.53
395	58.71	36.09	0.07	0.11	0.45	83.77
495	60.20	36.78	0.04	0.15	0.82	85.37
595	60.09	36.96	0.06	0.23	1.45	85.78
695	59.93	36.69	0.34	0.36	2.57	85.17
795	51.35	34.69	0.88	0.49	4.37	80.35

For Mz torsion	Axial force	Shear @ Y in kNm	Shear @ Z in kNm	TORSION	Moment @ Y in kNm	Moment @ Z in kNm
191	33.02	22.98	0.02	0.12	0.04	53.11
291	52.69	33.92	0.00	0.17	0.23	78.68
391	57.00	36.65	0.04	0.17	0.51	85.03
491	58.53	37.33	0.10	0.20	0.89	86.63
591	59.03	37.50	0.23	0.26	1.54	87.02
691	58.63	37.22	0.52	0.38	2.67	86.38
791	55.22	35.24	1.07	0.48	4.45	81.83

DESIGN FORCES FOR DIAGONAL BRACINGS BY DRAFT CODE PROVISIONS

Maximum Tensile Force : 75 kN

4.6 DESIGN OF FOOTING

4.6.1 DESIGN OF FOOTING FOR PERIPHERALLY BRACED TRESTLE

DATA:

Outside diameter of raft	D1 =	14	m
Inside diameter of raft	D2 =	4	m
Mean Diameter	=	6.05	m
Thickness of shaft	=	0.5	m
Outside radius	a =	7	m
Mean radius	β a =	3.0250	m
	β =	0.43214	
	αa =	4.00000	
	α =	0.57143	
Area of raft	A =	153.938	m^2
Moment of Inertia	I =	1885.74	m^4
Section Modulus	Z =	269.392	m^3

Loads:

Direct load ,	P =	**3216.0**	T
Moment ,	M =	**1405.0**	T - m
	S.B.C. =	**7.5**	T / m^2

If P/A < M/Z , use equivalent moment for foundation design as follows:

q_p =P/A =	**20.89**	T / m^2
M/Z =	**5.2**	T / m^2
P/A + M/Z =	**26.11**	T / m^2
P/A - M/Z =	**15.68**	T / m^2

Design Pressure for Raft:

Due to direct load = P / A = p = **20.89** T / m^2

Due to moment = M_{eq} / Z = q = **5.22** T / m^2

RADIAL AND TANGENTIAL MOMENTS DUE TO DIRECT LOAD:

Constants:

$Y_1 = -\beta^4 + (8\alpha^2 \beta^2 \ln \beta) - (\beta^2 \gamma_2) - (\gamma_3 \ln \beta)$

$\mathbf{Y_1 = 4.3925}$

$Y_2 = (5.48\,\alpha^2 - 2.52 - 2.96\,\beta^2 - (8 \ln \beta) + (8\,\alpha^4 \ln \alpha)/(\alpha^2-1)$

$\mathbf{Y_2 = -7.3519}$

$Y_3 = (\alpha^2*((-6.82)-(8*\beta^2)-(21.65*\ln \beta)+(21.65*\alpha^2*\ln \alpha)/(\alpha^2-1))$

$\mathbf{Y_3 = 5.1346}$

$Y_4 = (-8)\alpha^2$

$\mathbf{Y_4 = -2.6122}$

$Y_5 = (8\,\beta^2 \ln \beta) - (\beta^2 \gamma_6) + (-\beta^4) - (\gamma_7 \ln\beta)$

$\mathbf{Y_5 = -1.6107}$

$Y_6 =$ $(5.48 - 2.52\alpha^2 - 2.96\ \beta^2 - (8 \ln \beta)+(8\ \alpha^4 \ln \alpha)/(\alpha^2-1)$

$Y_6 =$ -15.3519

$Y_7 =$ $(- 6.82\alpha^2) -(8\ \beta^2)+(21.65\alpha^4 \ln \alpha)/((\alpha^2-1)-(21.65\ \alpha^2 \ln \beta))$

$Y_7 =$ -3.8013

$Y_8 =$ -8.0000

Moments:

For $f < \beta$

$M_{ri} = (pa^2 / 64) * (-12.6\ f^2 -2.3\ Y_2 + (0.85/ f^2)\ Y_3 - Y_4\ (3.15 + 2.3 \ln f))$

$M_{ti} = (pa^2 / 64) * (-5.8\ f^2 -2.3\ Y_2 - (0.85/ f^2)\ Y_3 - Y_4\ (1.45 + 2.3 \ln f))$

$Q_{ri} = (pa / 2f)\ (\alpha^2 - f^2)$; $Q_t = 0$

For $f > \beta$

$M_{re} = (pa^2 / 64) * (-12.6\ f^2 -2.3\ Y_6 +(0.85/ f^2)\ Y_7 - Y_8\ (3.15 + 2.3 \ln f))$

$M_{te} = (pa^2 / 64) * (-5.8\ f^2 -2.3\ Y_6 - (0.85/ f^2)\ Y_7 - Y_8\ (1.45 + 2.3 \ln f))$

$Q_{re} = pa\ (1 - f^2) / 2f$

RADIAL AND CIRCUMFERNTIAL MOMENTS AT CENTER OF SHAFT:

Diameter at the section =	**12.2**	m
f = d'/D =	**0.87**	m

Radial Moments:

$M_{ri} =$ **706.25** T-m

Circumferential Moments:

$M_{ti} =$ **707.42** T-m

RADIAL AND TANGENTIAL MOMENTS DUE TO MOMENT:

Constants:

$Y_1 = -\beta^4 - (\gamma_2\ \beta^2) - (\gamma_3 /\beta^2)-(\gamma_4 \ln \beta)$

$Y_1 =$ 27.0996

$Y_2 =$ $(-5.46)*((1+\alpha^4)/(\alpha^2+1))-(3 / \beta^2) - 0.81\beta^2$

$Y_2 =$ 11.3584

$Y_3 = (3\beta^2\alpha^4)-(11.12\alpha^4/\beta^2)+(20.24\alpha^4/(\alpha^2 +1)$

$Y_3 =$ -4.4707

$Y_4 = 12\ \alpha^4$

$Y_4 =$ 1.2795

$Y_5 =$ $\gamma_6\ \beta^2 - \beta^4 - \gamma_7/ \beta^2 - \gamma_8 \ln\beta$

$Y_5 =$ 31.4401

$Y_6 = (-5.46)*((1+\alpha^4)/(\alpha^2+1))+(3\alpha^4 / \beta^2) - 0.81\beta^2$

$Y_6 =$ -4.7061

$Y_7 = (3\beta^2)-(11.12\alpha^4/\beta^2)+(20.24\alpha^4/(\alpha^2 +1)$

$Y_7 =$ -4.1618

$Y_8 =$ 12.0000

Moments:

For $f < \beta$

$M_{ri} = (qa^2 / 192) * (-20.6 f^3 -6.3 f Y_2 - (1.7/ f^3) Y_3 - Y_4 (1.15 / f)) * \text{Cos}\theta$

$M_{ti} = (qa^2 / 192) * (-7 f^3 - 2.9 f Y_2 + (1.7/ f^3) Y_3 - Y_4 (1.15 / f)) * \text{Cos}\theta$

$Q_{ri} = (qa / 192) (-72f^2 - 8 Y_2 + (2/ f^2) Y_4) \text{Cos}\ \theta$

$Q_{ti} = (qa / 192) (24 f^2 + 8 Y_2 + (2/ f^2) Y_4) \text{Sin}\ \theta$

For $f > \beta$

$M_{re} = (qa^2 / 192) * (-20.6 f^3 -6.3 f Y_6 - (1.7/ f^3) Y_7 - Y_8 (1.15 / f)) \text{Cos}\theta$

$M_{te} = (qa^2 / 192) * (-7 f^3 - 2.9 f Y_6 + (1.7/ f^3) Y_7 - Y_8 (1.15 / f)) * \text{Cos}\theta$

$Q_{re} = (qa / 192) (-72f^2 - 8 Y_6 + (2/ f^2) Y_8) \cos\theta$

Diameter at the section =	**19.65**	m
f = d'/D =	**1.40**	m

Radial Moments:

M_{ri} = **-30.11** T-m

Circumferential Moments:

M_{ti} = **-16.76** T-m

4.7.2 DESIGN OF FOOTING FOR PERIPHERALLY AND DIAGONALLY BRACED TRESTLE

DATA:

Outside diameter of raft	D1 =	14	m
Inside diameter of raft	D2 =	4	m
Mean Diameter	=	6.05	m
Thickness of	=	0.5	m
Outside radius	a =	7	m
Mean radius	β a =	3.0250	m
	β =	0.43214	
	αa =	4.00000	
	α =	0.57143	
Area of raft	A =	153.938	m^2
Moment of Inertia	I =	1885.74	m^4
Section Modulus	Z =	269.392	m^3

Loads:

Direct load ,	P =	**3216.0**	T
Moment ,	M =	**1674.3**	T - m
	S.B.C. =	**7.5**	T / m^2

If P/A < M/Z , use equivalent moment for foundation design as follows:

q_p =P/A =	**20.89**	T / m^2
M/Z =	**6.2**	T / m^2
P/A + M/Z =	**27.11**	T / m^2
P/A - M/Z =	**14.68**	T / m^2

Design Pressure for Raft:

Due to direct load	= P / A = p =	**20.89**	T / m^2
Due to moment	= M_{eq} / Z = q =	**6.22**	T / m^2

RADIAL AND TANGENTIAL MOMENTS DUE TO DIRECT LOAD:

Constants:

$Y_1 = -\beta^4 + (8\alpha^2 \beta^2 \ln \beta) - (\beta^2 \gamma_2) - (\gamma_3 \ln \beta)$

$\mathbf{Y_1 = 4.3925}$

$Y_2 = (5.48\ \alpha^2 - 2.52 - 2.96\ \beta^2 - (8 \ln \beta) + (8\ \alpha^4 \ln \alpha)/(\alpha^2-1)$

$\mathbf{Y_2 = -7.3519}$

$Y_3 = (\alpha^{2} * ((-6.82)-(8*\beta^2)-(21.65* \ln \beta)+(21.65*\alpha^2 * \ln \alpha)/(\alpha^2-1))$

$\mathbf{Y_3 = 5.1346}$

$Y_4 = (-8)\alpha^2$

$\mathbf{Y_4 = -2.6122}$

$Y_5 = (8\ \beta^2 \ln \beta) - (\beta^2 \gamma_6) + (-\beta^4) - (\gamma_7 \ln\beta)$

$\mathbf{Y_5 = -1.6107}$

$Y_6 =$ (5.48 - 2.52α^2 - 2.96 β^2 - (8 ln β)+(8 α^4 ln α)/(α^2-1)

Y_6 = -15.3519

$Y_7 =$ (- 6.82α^2) -(8 β^2)+(21.65α^4 ln α)/((α^2-1)-(21.65 α^2 ln β))

Y_7 = -3.8013

Y_8 = -8.0000

Moments:

For f < β

M_{ri} = (pa^2 / 64) * (-12.6 f^2 -2.3 Y_2 + (0.85/ f^2) Y_3 - Y_4 (3.15 + 2.3 ln f))

M_{ti} = (pa^2 / 64) * (-5.8 f^2 -2.3 Y_2 - (0.85/ f^2) Y_3 - Y_4 (1.45 + 2.3 ln f))

Q_{ri} = (pa / 2f) (α^2 - f^2) ; Q_t = 0

For f > β

M_{re} = (pa^2 / 64) * (-12.6 f^2 -2.3 Y_6 +(0.85/ f^2) Y_7 - Y_8 (3.15 + 2.3 ln f))

M_{te} = (pa^2 / 64) * (-5.8 f^2 -2.3 Y_6 - (0.85/ f^2) Y_7 - Y_8 (1.45 + 2.3 ln f))

Q_{re} = pa (1 - f^2) / 2f

RADIAL AND CIRCUMFERNTIAL MOMENTS AT CENTER OF SHAFT:

Diameter at the section = **12.2** m

f = d'/D = **0.87** m

Radial Moments:

M_{ri} = **706.25** T-m

Circumferential Moments:

M_{ti} = **707.42** T-m

RADIAL AND TANGENTIAL MOMENTS DUE TO MOMENT:

Constants:

Y_1 = -β^4 - (γ_2 β^2) - (γ_3 /β^2)-(γ_4 ln β)

Y_1 = 27.0996

Y_2 = (-5.46)*((1+α^4)/(α^2+1))-(3 / β^2) - 0.81β^2

Y_2 = 11.3584

Y_3 = (3$\beta^2\alpha^4$)-(11.12α^4/β^2)+(20.24α^4/(α^2 +1)

Y_3 = -4.4707

Y_4 = 12 α^4

Y_4 = 1.2795

Y_5 = γ_6 β^2 - β^4 - γ_7/ β^2 - γ_8 lnβ

Y_5 = 31.4401

Y_6 = (-5.46)*((1+α^4)/(α^2+1))+(3α^4 / β^2) - 0.81β^2

Y_6 = -4.7061

Y_7 = (3β^2)-(11.12α^4/β^2)+(20.24α^4/(α^2 +1)

Y_7 = -4.1618

Y_8 = 12.0000

Moments:

For $f < \beta$

$M_{ri} = (qa^2 / 192) * (-20.6 f^3 - 6.3 f Y_2 - (1.7/ f^3) Y_3 - Y_4 (1.15 / f)) * \cos\theta$

$M_{ti} = (qa^2 / 192) * (-7 f^3 - 2.9 f Y_2 + (1.7/ f^3) Y_3 - Y_4 (1.15 / f)) * \cos\theta$

$Q_{ri} = (qa / 192) (-72f^2 - 8 Y_2 + (2/ f^2) Y_4) \cos\theta$

$Q_{ti} = (qa / 192) (24 f^2 + 8 Y_2 + (2/ f^2) Y_4) \sin\theta$

For $f > \beta$

$M_{re} = (qa^2 / 192) * (-20.6 f^3 - 6.3 f Y_6 - (1.7/ f^3) Y_7 - Y_8 (1.15 / f)) \cos\theta$

$M_{te} = (qa^2 / 192) * (-7 f^3 - 2.9 f Y_6 + (1.7/ f^3) Y_7 - Y_8 (1.15 / f)) * \cos\theta$

$Q_{re} = (qa / 192) (-72f^2 - 8 Y_6 + (2/ f^2) Y_8) \cos\theta$

Diameter at the section = **19.65** m

f = d'/D = **1.40** m

Radial Moments:

M_{ri} = **-35.88** T-m

Circumferential Moments:

M_{ti} = **-19.97** T-m

4.7 DESIGN SUMMARY

Type of braced frame : **Peripherally braced frame**
Code referred : IS1893-2005 (Part II)Draft code

For columns

Level	Width	Length	Main dia	No.s	Stirrups dia	c/c spacing
1st	550	800	25	22	8	300
2nd	550	800	25	18	8	300
3rd	550	800	25	18	8	300
4th	550	800	25	18	8	300
5th	550	800	25	16	8	300
6th	550	800	25	14	8	300
7th	550	800	25	14	8	300
8th	550	800	25	14	8	300

Total main steel & stirrups steel in m^3 for given no. of columns

For Bracings

Length of bracing = 4630 mm

Level	b	D	main dia	no.s	Stirrups dia	c/c spacing
1st	250	400	20	4	8	250
2nd	300	480	20	6	8	300
3rd	300	500	25	4	8	300
4th	300	550	25	5	8	300
5th	300	550	25	5	8	300
6th	300	550	25	5	8	300
7th	300	480	20	6	8	300

Total Total values in m^3

Type of braced frame : **Peripherally & Diagonally braced frame**
Code referred : IS1893-2005 (Part II)Draft code

For columns

Level	Width	Length	Main dia	No.s	Stirrups dia	c/c spacing
1st	550	800	25	22	8	300
2nd	550	800	25	20	8	300
3rd	550	800	25	18	8	300
4th	550	800	25	18	8	300
5th	550	800	25	18	8	300
6th	550	800	25	16	8	300
7th	550	800	25	14	8	300
8th	550	800	25	14	8	300

Total main steel & stirrups steel in m^3 for given no. of columns

For Bracings

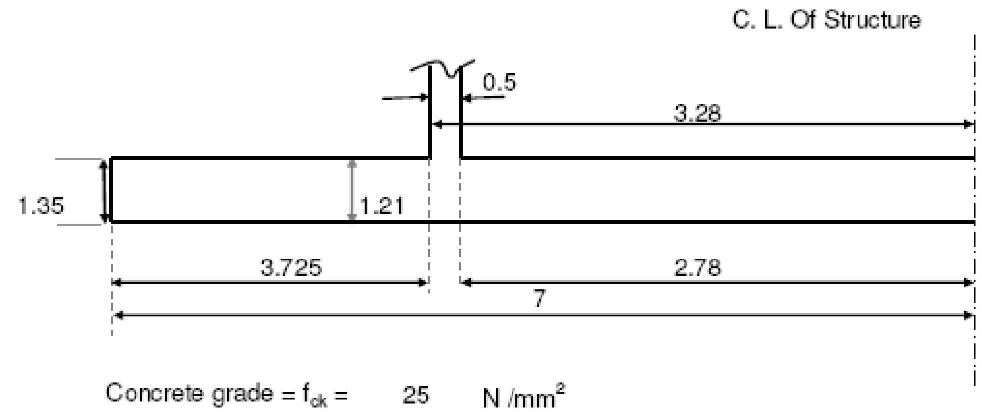
Length of bracing = 4630 mm

Level	b	D	main dia	no.s	Stirrups dia	c/c spacing
1st	230	350	25	2	8	230
2nd	300	380	25	3	8	300
3rd	300	380	25	3	8	300
4th	300	400	25	3	8	300
5th	300	400	25	3	8	300
6th	300	400	25	3	8	300
7th	300	380	25	3	8	300

SUMMARY OF RADIAL AND TANGENTIAL MOMENTS IN RAFT:

Radial Distance (m)	Dia. d' (m)	f d' / D1	Due to Direct Load M_r (T-m)	Due to Direct Load M_t (T-m)	Due to Moment M_r (T-m)	Due to Moment M_t (T-m)	Combined M_r (T-m)	Combined M_t (T-m)
7.000	14.000	1.000	714.63	709.23	3.09	-18.94	717.72	728.17
6.628	13.255	0.947	713.44	708.72	5.79	-21.21	719.23	729.92
6.255	12.510	0.894	709.09	707.85	8.34	-24.17	717.43	732.02
5.883	11.765	0.840	701.15	706.80	10.90	-27.99	712.06	734.79
5.510	11.020	0.787	689.13	705.81	13.66	-32.89	702.79	738.70
5.138	10.275	0.734	672.30	705.25	16.92	-39.20	689.22	744.45
4.765	9.530	0.681	649.74	705.67	21.09	-47.41	670.82	753.08
4.393	8.785	0.628	620.09	707.89	26.83	-58.29	646.92	766.18
4.020	8.040	0.574	581.45	713.19	35.21	-73.04	616.65	786.23
3.648	7.295	0.521	530.93	723.62	48.00	-93.67	578.93	817.29
3.275	6.550	0.468	464.07	742.56	68.35	-123.67	532.43	866.23
3.025	6.050	0.432	232.83	253.14	-43.37	-19.70	276.20	272.84
2.775	5.550	0.396	238.79	255.89	-39.47	-17.96	278.26	273.85
2.498	4.995	0.357	244.81	258.66	-35.23	-16.07	280.04	274.72
2.220	4.440	0.317	250.20	261.14	-31.08	-14.20	281.28	275.34
1.943	3.885	0.278	254.95	263.32	-27.02	-12.37	281.96	275.69
1.665	3.330	0.238	259.06	265.22	-23.02	-10.55	282.09	275.77
1.388	2.775	0.198	262.55	266.82	-19.09	-8.76	281.64	275.58
1.110	2.220	0.159	265.40	268.13	-15.21	-6.99	280.61	275.12
0.833	1.665	0.119	267.62	269.15	-11.37	-5.23	278.99	274.38
0.555	1.110	0.079	269.20	269.88	-7.57	-3.48	276.76	273.36
0.278	0.555	0.040	270.15	270.32	-3.78	-1.74	273.93	272.06
0.000	0.000	0.000	270.47	270.47	0.00	0.00	270.47	270.47

DESIGN OF RAFT USING LIMIT STATE METHOD AS PER IS 456: 2000



Concrete grade = f_{ck} = 25 N /mm^2

Steel grade = f_y = 415 N /mm^2

Effective depth = 1.21

Thickness at Outside Edge = 1.35 m

Thickness at Inside Edge = 1.35 m

Effective cover = 70 mm

Diameter of bar = 25 mm

$A_{st,\ min.}$ = 0.15 * bd / 100 at any section

Load Factor (RF) = 1

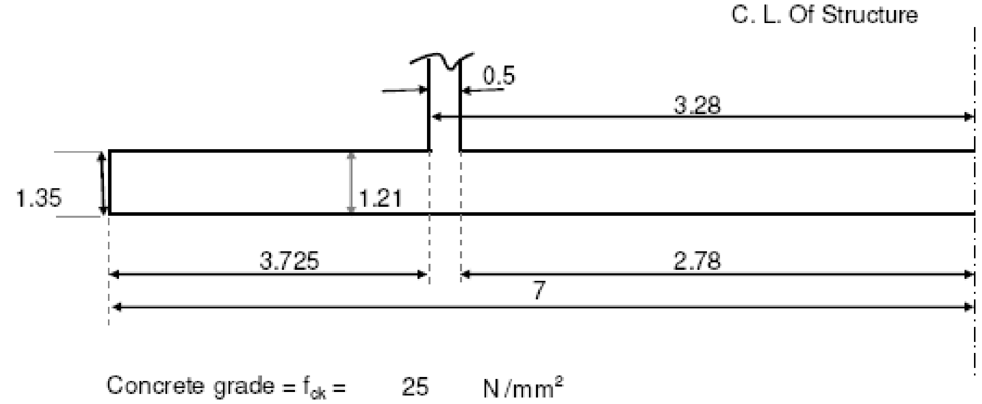
DESIGN FOR FLEXURE:

Diameter (m)	Radial Distance (m)	Effective Thickness (m)	RF * Moment M_r (T-m)	RF * Moment M_t (T-m)	RADIAL STEEL M_u/bd^2 (N/mm^2)	$p_{t,\ req}$	$A_{st,\ req}$ (cm^2/ m)	$A_{st,\ req}$ (cm^2)	Provide Dia	Provide Nos.	$A_{st,\ pro}$ (cm^2)	TANGENTIAL STEEL M_u/bd^2 (N/mm^2)	$p_{t,\ req}$	$A_{st,\ req}$ (cm^2/ m)	Provide Dia	Provide spacing	$A_{st,\ pro}$ (cm^2/ m)
14.00	**7.000**	1.280	717.72	728.17	4.381	1.686	215.75	9489	25	225	1104	4.444	1.726	220.96	25	240	20.44
13.26	6.628	1.280	719.23	729.92	4.390	1.691	216.49	9015	25	225	1104	4.455	1.733	221.85	25	240	20.44
12.51	6.255	1.280	717.43	732.02	4.379	1.684	215.60	8473	25	225	1104	4.468	1.742	222.92	25	240	20.44
11.77	5.883	1.280	712.06	734.79	4.346	1.664	212.98	7872	25	225	1104	4.485	1.753	224.35	25	200	24.53
11.02	5.510	1.280	702.79	738.70	4.289	1.629	208.56	7220	25	225	1104	4.509	1.769	226.39	25	200	24.53
10.28	5.138	1.280	689.22	744.45	4.207	1.580	202.27	6529	25	225	1104	4.544	1.792	229.43	25	175	28.04
9.53	4.765	1.280	670.82	753.08	4.094	1.516	194.07	5810	25	450	2209	4.596	1.829	234.12	25	175	28.04
8.79	4.393	1.280	646.92	766.18	3.948	1.437	183.92	5076	25	450	2209	4.676	1.887	241.52	25	150	32.71
8.04	4.020	1.280	616.65	786.23	3.764	1.342	171.76	4338	25	80	393	4.799	1.982	253.65	25	150	32.71
7.30	3.648	1.280	578.93	817.29	3.534	1.231	157.51	3610	25	80	393	4.988	2.149	275.04	25	130	37.74
6.55	**3.275**	1.280	532.43	866.23	3.250	1.102	**141.08**	2903	25	80	393	5.287	2.515	321.98	25	130	37.74
6.05	**3.025**																
5.55	**2.775**	1.280	278.26	273.85	1.698	0.515	**65.87**	1148	25	450	2209	1.671	0.506	64.72	25	140	35.04
5.00	2.498	1.280	280.04	274.72	1.709	0.518	66.33	1041	25	450	2209	1.677	0.507	64.94	25	140	35.04
4.44	2.220	1.280	281.28	275.34	1.717	0.521	66.66	930	25	450	2209	1.681	0.509	65.10	25	150	32.71
3.89	1.943	1.280	281.96	275.69	1.721	0.522	66.84	816	25	450	2209	1.683	0.509	65.20	25	200	24.53
3.33	1.665	1.280	282.09	275.77	1.722	0.522	66.87	700	20	450	1414	1.683	0.510	65.22	25	200	24.53
2.78	1.388	1.280	281.64	275.58	1.719	0.521	66.75	582	20	450	1414	1.682	0.509	65.17	25	200	24.53
2.22	1.110	1.280	280.61	275.12	1.713	0.519	66.48	464	20	450	1414	1.679	0.508	65.05	25	220	22.30
1.67	0.833	1.280	278.99	274.38	1.703	0.516	66.06	346	20	450	1414	1.675	0.507	64.86	25	220	22.30
1.11	0.555	1.280	276.76	273.36	1.689	0.512	65.48	228	20	450	1414	1.668	0.505	64.59	25	220	22.30
0.56	0.278	1.280	273.93	272.06	1.672	0.506	64.74	113	20	450	1414	1.661	0.502	64.25	25	250	19.63
0.00	**0.000**	1.280	270.47	270.47	1.651	0.499	63.84	0	20	450	1414	1.651	0.499	63.84	25	250	19.63

SUMMARY OF RADIAL AND TANGENTIAL MOMENTS IN RAFT:

Radial Distance (m)	Dia. d' (m)	f d' / D1	Due to Direct Load M_r (T-m)	Due to Direct Load M_t (T-m)	Due to Moment M_r (T-m)	Due to Moment M_t (T-m)	Combined M_r (T-m)	Combined M_t (T-m)
7.000	14.000	1.000	714.63	709.23	3.69	-22.57	718.32	731.79
6.628	13.255	0.947	713.44	708.72	6.90	-25.27	720.34	733.99
6.255	12.510	0.894	709.09	707.85	9.94	-28.80	719.03	736.65
5.883	11.765	0.840	701.15	706.80	12.99	-33.35	714.14	740.15
5.510	11.020	0.787	689.13	705.81	16.28	-39.19	705.41	745.00
5.138	10.275	0.734	672.30	705.25	20.16	-46.71	692.46	751.96
4.765	9.530	0.681	649.74	705.67	25.13	-56.50	674.86	762.17
4.393	8.785	0.628	620.09	707.89	31.97	-69.46	652.06	777.35
4.020	8.040	0.574	581.45	713.19	41.95	-87.04	623.40	800.23
3.648	7.295	0.521	530.93	723.62	57.19	-111.62	588.13	835.24
3.275	6.550	0.468	464.07	742.56	81.45	-147.38	545.53	889.94
3.025	6.050	0.432	232.83	253.14	-51.69	-23.47	284.51	276.62
2.775	5.550	0.396	238.79	255.89	-47.03	-21.40	285.82	277.29
2.498	4.995	0.357	244.81	258.66	-41.98	-19.15	286.79	277.80
2.220	4.440	0.317	250.20	261.14	-37.04	-16.92	287.23	278.06
1.943	3.885	0.278	254.95	263.32	-32.19	-14.74	287.14	278.06
1.665	3.330	0.238	259.06	265.22	-27.44	-12.58	286.50	277.79
1.388	2.775	0.198	262.55	266.82	-22.75	-10.44	285.30	277.26
1.110	2.220	0.159	265.40	268.13	-18.13	-8.33	283.53	276.46
0.833	1.665	0.119	267.62	269.15	-13.55	-6.23	281.17	275.39
0.555	1.110	0.079	269.20	269.88	-9.02	-4.15	278.21	274.03
0.278	0.555	0.040	270.15	270.32	-4.50	-2.07	274.65	272.39
0.000	0.000	0.000	270.47	270.47	0.00	0.00	270.47	270.47

DESIGN OF RAFT USING LIMIT STATE METHOD AS PER IS 456: 2000



Concrete grade = f_{ck} = 25 N/mm²

Steel grade = f_y = 415 N/mm²

Effective depth = 1.21

Thickness at Outside Edge = 1.35 m

Thickness at Inside Edge = 1.35 m

Effective cover = 70 mm

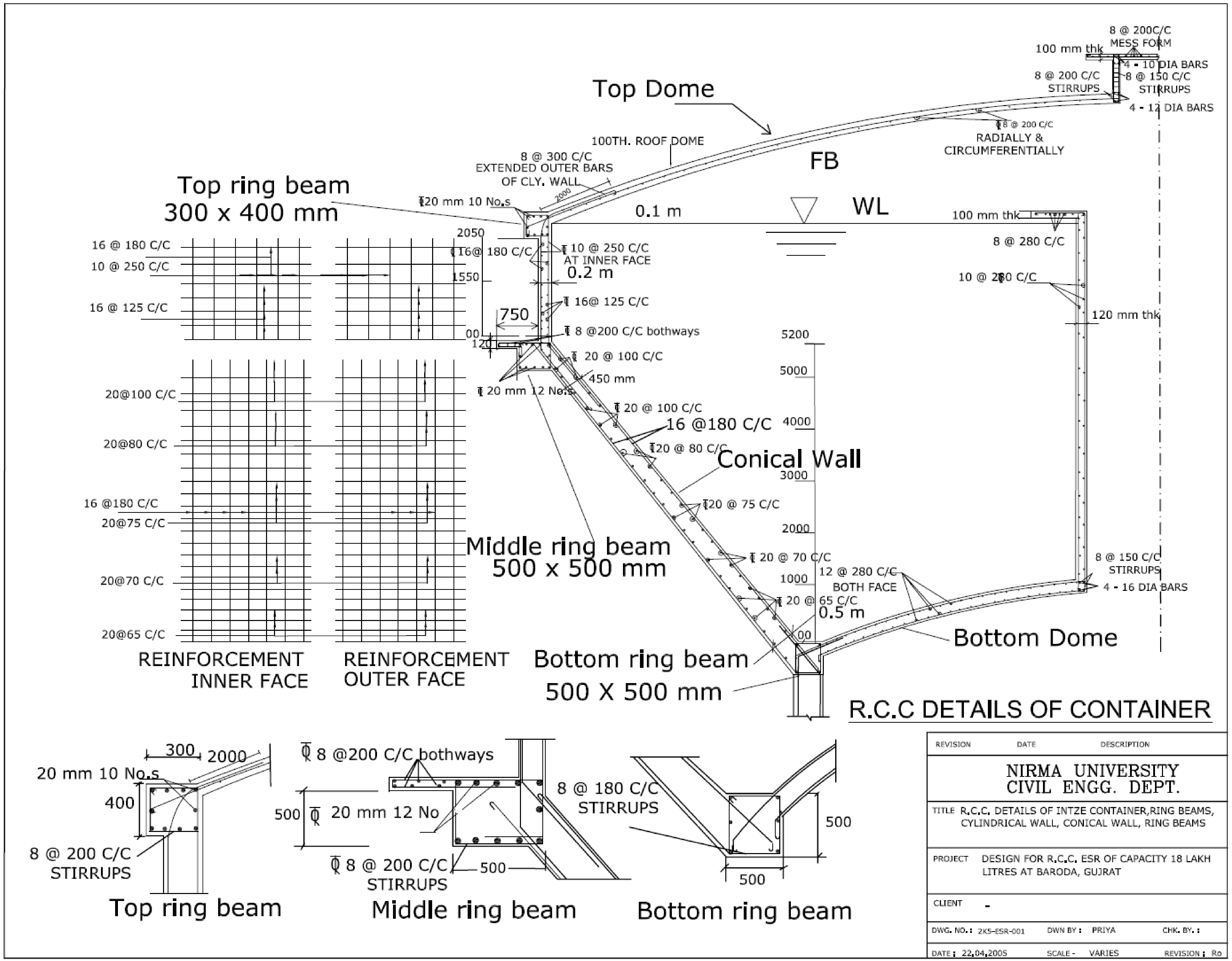
Diameter of bar = 25 mm

$A_{st,\ min.}$ = 0.15 * bd / 100 at any section

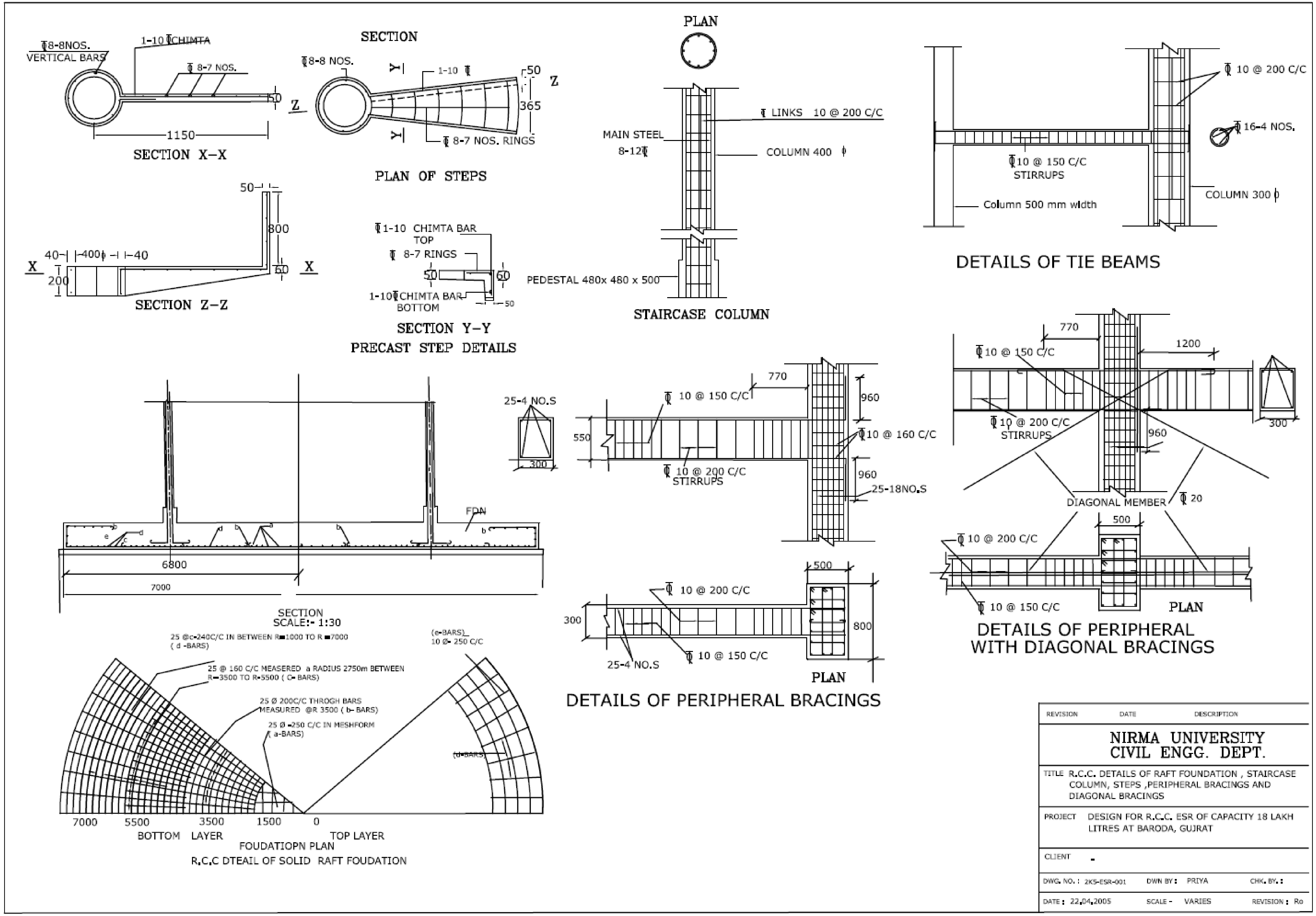
Load Factor (RF) = 1

DESIGN FOR FLEXURE:

Diameter (m)	Radial Distance (m)	Effective Thickness (m)	RF * Moment M_r (T-m)	RF * Moment M_t (T-m)	RADIAL STEEL M_u/bd^2 (N/mm²)	$p_{t,\ req}$	$A_{st,\ req}$ (cm²/ m)	$A_{st,\ req}$ (cm²)	Provide Dia	Provide Nos.	$A_{st,\ pro}$ (cm²)	TANGENTIAL STEEL M_u/bd^2 (N/mm²)	$p_{t,\ req}$	$A_{st,\ req}$ (cm²/ m)	Provide Dia	Provide spacing	$A_{st,\ pro}$ (cm²/ m)
14.00	**7.000**	1.280	718.32	731.79	4.384	1.688	216.04	9502	25	225	1104	4.467	1.741	222.81	25	240	20.44
13.26	6.628	1.280	720.34	733.99	4.397	1.696	217.04	9038	25	225	1104	4.480	1.750	223.94	25	240	20.44
12.51	6.255	1.280	719.03	736.65	4.389	1.691	216.39	8504	25	225	1104	4.496	1.760	225.32	25	240	20.44
11.77	5.883	1.280	714.14	740.15	4.359	1.672	213.99	7909	25	225	1104	4.518	1.775	227.15	25	200	24.53
11.02	5.510	1.280	705.41	745.00	4.305	1.639	209.80	7263	25	225	1104	4.547	1.795	229.73	25	200	24.53
10.28	5.138	1.280	692.46	751.96	4.226	1.592	203.75	6577	25	225	1104	4.590	1.824	233.50	25	175	28.04
9.53	4.765	1.280	674.86	762.17	4.119	1.530	195.84	5863	25	450	2209	4.652	1.869	239.21	25	175	28.04
8.79	4.393	1.280	652.06	777.35	3.980	1.454	186.06	5135	25	450	2209	4.745	1.939	248.15	25	150	32.71
8.04	4.020	1.280	623.40	800.23	3.805	1.363	174.41	4405	25	80	393	4.884	2.053	262.83	25	150	32.71
7.30	3.648	1.280	588.13	835.24	3.590	1.257	160.90	3687	25	80	393	5.098	2.262	289.56	25	130	37.74
6.55	**3.275**	1.280	545.53	889.94	3.330	1.137	**145.59**	2996	25	80	393	5.432	2.941	376.45	25	130	37.74
6.05	**3.025**																
5.55	**2.775**	1.280	285.82	277.29	1.745	0.530	**67.85**	1183	25	450	2209	1.692	0.513	65.61	25	140	35.04
5.00	2.498	1.280	286.79	277.80	1.750	0.532	68.10	1069	25	450	2209	1.696	0.514	65.75	25	140	35.04
4.44	2.220	1.280	287.23	278.06	1.753	0.533	68.22	952	25	450	2209	1.697	0.514	65.81	25	150	32.71
3.89	1.943	1.280	287.14	278.06	1.753	0.533	68.19	832	25	450	2209	1.697	0.514	65.81	25	200	24.53
3.33	1.665	1.280	286.50	277.79	1.749	0.531	68.03	712	20	450	1414	1.696	0.514	65.75	25	200	24.53
2.78	1.388	1.280	285.30	277.26	1.741	0.529	67.71	590	20	450	1414	1.692	0.513	65.61	25	200	24.53
2.22	1.110	1.280	283.53	276.46	1.731	0.525	67.25	469	20	450	1414	1.687	0.511	65.40	25	220	22.30
1.67	0.833	1.280	281.17	275.39	1.716	0.521	66.63	349	20	450	1414	1.681	0.509	65.12	25	220	22.30
1.11	0.555	1.280	278.21	274.03	1.698	0.514	65.86	230	20	450	1414	1.673	0.506	64.76	25	220	22.30
0.56	0.278	1.280	274.65	272.39	1.676	0.507	64.93	113	20	450	1414	1.663	0.503	64.34	25	250	19.63
0.00	**0.000**	1.280	270.47	270.47	1.651	0.499	63.84	0	20	450	1414	1.651	0.499	63.84	25	250	19.63



Top Dome
100TH. ROOF DOME
8 @ 300 C/C
EXTENDED OUTER BARS
OF CLY. WALL
Top ring beam
300 x 400 mm
20 mm 10 No.s
FB
WL
0.1 m
0.2 m
10 @ 250 C/C
AT INNER FACE
16@ 180 C/C
16@ 125 C/C
8 @200 C/C bothways
750
20 @ 100 C/C
450 mm
20 mm 12 No.s
20 @ 100 C/C
16 @180 C/C
20 @ 80 C/C
Conical Wall
20 @ 75 C/C
20 @ 70 C/C
20 @ 65 C/C
0.5 m
12 @ 280 C/C
BOTH FACE
Middle ring beam
500 x 500 mm
Bottom ring beam
500 X 500 mm
Bottom Dome
8 @ 200C/C
MESS FORM
100 mm thk
4 - 10 DIA BARS
8 @ 200 C/C
STIRRUPS
8 @ 150 C/C
STIRRUPS
4 - 12 DIA BARS
8 @ 200 C/C
RADIALLY &
CIRCUMFERENTIALLY
100 mm thk
8 @ 280 C/C
10 @ 280 C/C
120 mm thk
8 @ 150 C/C
STIRRUPS
4 - 16 DIA BARS
5200
5000
4000
3000
2000
1000
16 @ 180 C/C
10 @ 250 C/C
16 @ 125 C/C
20@100 C/C
20@80 C/C
16 @180 C/C
20@75 C/C
20@70 C/C
20@65 C/C
REINFORCEMENT
INNER FACE
REINFORCEMENT
OUTER FACE
R.C.C DETAILS OF CONTAINER
300
2000
20 mm 10 No.s
400
8 @ 200 C/C
STIRRUPS
Top ring beam
8 @200 C/C bothways
500
20 mm 12 No
8 @ 200 C/C
STIRRUPS
500
8 @ 180 C/C
STIRRUPS
Middle ring beam
500
500
Bottom ring beam
REVISION
DATE
DESCRIPTION
NIRMA UNIVERSITY
CIVIL ENGG. DEPT.
TITLE R.C.C. DETAILS OF INTZE CONTAINER,RING BEAMS,
CYLINDRICAL WALL, CONICAL WALL, RING BEAMS
PROJECT DESIGN FOR R.C.C. ESR OF CAPACITY 18 LAKH
LITRES AT BARODA, GUJRAT
CLIENT -
DWG. NO.: 2K5-ESR-001
DWN BY : PRIYA
CHK. BY. :
DATE : 22.04.2005
SCALE - VARIES
REVISION : Ro


SECTION X-X
SECTION
PLAN OF STEPS
SECTION Z-Z
SECTION Y-Y
PRECAST STEP DETAILS
PLAN
MAIN STEEL 8-12
LINKS 10 @ 200 C/C
COLUMN 400
PEDESTAL 480x 480 x 500
STAIRCASE COLUMN
10 @ 200 C/C
16-4 NOS.
10 @ 150 C/C STIRRUPS
COLUMN 300
Column 500 mm width
DETAILS OF TIE BEAMS
25-4 NO.S
10 @ 150 C/C
10 @ 200 C/C STIRRUPS
10 @ 160 C/C
25-18NO.S
10 @ 200 C/C
10 @ 150 C/C
PLAN
DETAILS OF PERIPHERAL BRACINGS
DIAGONAL MEMBER
10 @ 200 C/C STIRRUPS
10 @ 150 C/C
PLAN
DETAILS OF PERIPHERAL WITH DIAGONAL BRACINGS
SECTION SCALE:- 1:30
25 @c=240C/C IN BETWEEN R=1000 TO R =7000 (d -BARS)
25 @ 160 C/C MEASERED a RADIUS 2750m BETWEEN R=3500 TO R=5500 (C- BARS)
25 Ø 200C/C THROGH BARS MEASURED @R 3500 (b- BARS)
25 Ø =250 C/C IN MESHFORM (a-BARS)
(e-BARS) 10 Ø= 250 C/C
BOTTOM LAYER
TOP LAYER
FOUDATIOPN PLAN
R.C.C DTEAIL OF SOLID RAFT FOUDATION
REVISION DATE DESCRIPTION
NIRMA UNIVERSITY
CIVIL ENGG. DEPT.
TITLE R.C.C. DETAILS OF RAFT FOUNDATION , STAIRCASE COLUMN, STEPS ,PERIPHERAL BRACINGS AND DIAGONAL BRACINGS
PROJECT DESIGN FOR R.C.C. ESR OF CAPACITY 18 LAKH LITRES AT BARODA, GUJRAT
CLIENT -
DWG. NO. : 2KS-ESR-001 DWN BY : PRIYA CHK. BY. :
DATE : 22.04.2005 SCALE - VARIES REVISION : Ro

5. DYNAMIC ANALYSIS OF WATERTANK

5.1 MODEL SIMULATION

The approximate analysis is can be obtained following *IS: 1893-2001 (Part I)* procedure. For the exact analysis we have to take help from software. In present project, for the dynamic analysis of Case study problem STAAD Pro software is used. The results are compared with the approximate analysis and found to be comparable showing accuracy of work.

The wire frame model constructed in STAAD is as shown below.

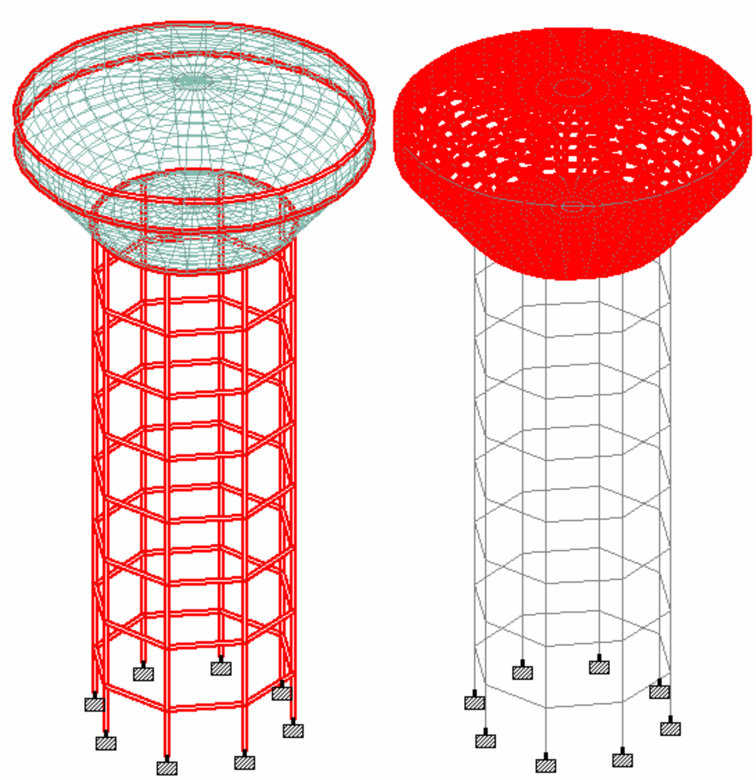


FIGURE 5.1 BEAM ELEMENTS AND PLATE ELEMENTS

MODEL DATA

No. of Nodes: 1154

No. of plates: 1120

No. of beams: 216

Base condition: Fixed Joints

GEOMETRICAL DIMENTIONS OF WATERTANK

Capacity	=	1800 m^3
Staging height above ground	=	30 m
Radius of container	=	21 m
Height of cylindrical wall	=	2.05 m
No. of columns	=	8 No.s along periphery
No. of Bracing panels	=	7 No.s
Panel depth	=	4.34 m at above plinth
At plinth level depth of panel	=	3.1 m
Rise of top Dome	=	2.1 m
Rise of Bottom Dome	=	1.3 m
Length of Bracing	=	4.63 m
Depth of footing below ground	=	2 m

STRUCTURAL DIMENTIONS

Thickness of Top dome	=	100 mm
Thickness of Bottom dome	=	200 mm
Thickness of Cylindrical wall	=	200 mm
Thickness of Conical Wall	=	500 mm
Columns	=	500 x 800 mm
Peripheral Bracings	=	200 x 500 mm
Bottom Ring Beam	=	500 x 500 mm
Middle Ring Beam	=	500 x 500 mm
Top Ring Beam	=	300 x 400 mm

SEISMIC DATA

Zone: III

Soil Condition: Medium Soil

I importance factor: 1.5

R: 2.5 as per IS 1893-2005 (Part II) Proposed Draft Code

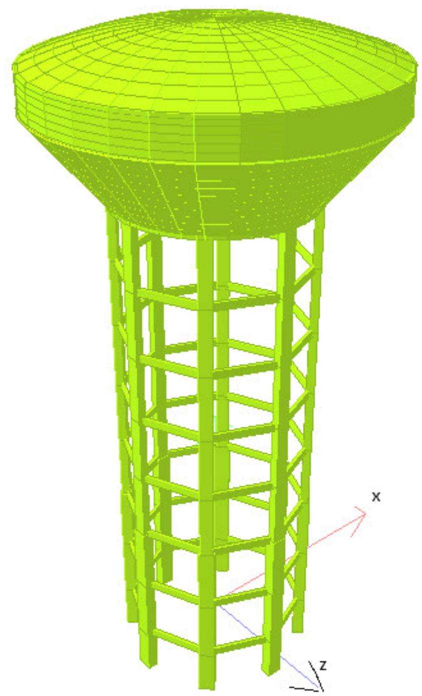


FIGURE 5.2 THREE DIMENTIONAL VIEW OF WATERTANK

5.2 PROPERTIES OF MODEL

MATERIAL PROPERTIES ASSIGNED

ISOTROPIC CONCRETE

E 2.17185e+007

POISSON 0.17

DENSITY 23.5616

ALPHA 1e-005

DAMP 0.05

5.3 LOADS CONSIDERED

LOADS CONSIDERED

LOAD 1 SELFWEIGHT

LOAD 2 LIVE LOAD

LOAD 3 WATERLOAD

LOAD 4 HYDROSTATIC LOADS

LOAD 5 SEISMIC LOADS

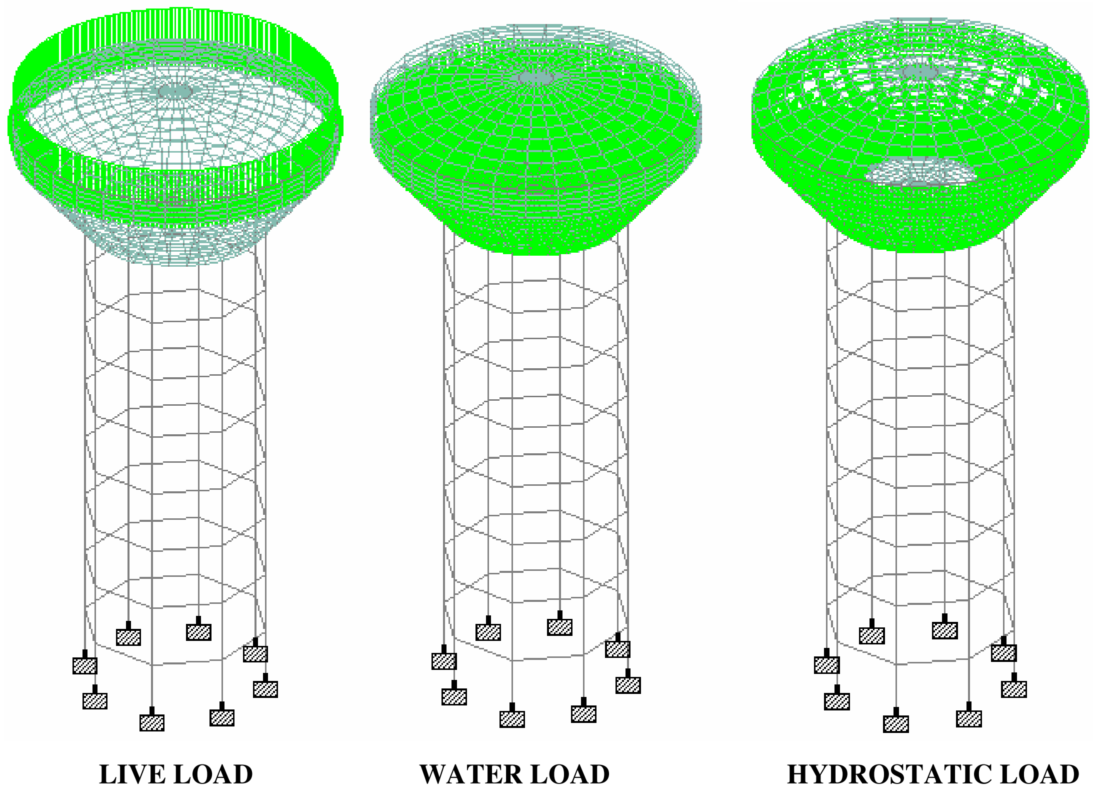


FIGURE 5.3 LOADS CONSIDERED FOR DYNAMIC ANALYSIS

LOAD COMBINATIONS USED FOR CALCULATING DESIGN FORCES

LOAD COMB 11 1.5DL + 1.5LL

1 1.5 2 1.5 3 1.5

LOAD COMB 12 1.5DL + 1.5EQLX

1 1.5 5 1.5 3 1.5

LOAD COMB 13 1.5DL - 1.5EQLX

1 1.5 5 -1.5 3 1.5

LOAD COMB 14 1.5DL + 1.5EQLZ

1 1.5 6 1.5 3 1.5

LOAD COMB 15 1.5DL - 1.5EQLZ

1 1.5 6 -1.5 3 1.5

LOAD COMB 16 0.9DL + 1.5EQLX

1 0.9 5 1.5 3 0.9

LOAD COMB 17 0.9DL - 1.5EQLX

1 0.9 5 -1.5 3 0.9

LOAD COMB 18 0.9DL + 1.5EQLZ

1 0.9 6 1.5 3 0.9

LOAD COMB 19 0.9DL - 1.5EQLZ

1 0.9 6 -1.5 3 0.9

LOAD COMB 20 1.2DL + 1.2LL + 1.2EQLX

1 1.2 2 1.2 5 1.2 3 1.2

LOAD COMB 21 1.2DL + 1.2LL - 1.2EQLX

1 1.2 2 1.2 5 -1.2 3 1.2

LOAD COMB 22 1.2DL + 1.2LL + 1.2EQLZ

1 1.2 2 1.2 6 1.2 3 1.2

LOAD COMB 23 1.2DL + 1.2LL - 1.2EQLZ

1 1.2 2 1.2 6 -1.2 3 1.2

5.4 RESULTS IN GRAPHICAL FORM

TABLE 5.1 EIGEN SOLUTIONS

MODE	FREQUENCY(CYCLES/SEC)	PERIOD(SEC)	ACCURACY
1	0.18	5.62	7.12E-16
2	0.18	5.62	1.78E-16
3	0.21	4.83	3.94E-16
4	2.25	0.44	2.70E-15
5	2.25	0.44	2.99E-15
6	2.35	0.43	1.30E-11
7	2.35	0.43	6.50E-07
8	3.03	0.33	5.27E-08
9	3.06	0.33	4.16E-13
10	3.06	0.33	9.80E-13
11	3.46	0.29	2.22E-11
12	3.46	0.29	1.94E-11
13	3.94	0.25	3.45E-12
14	4.13	0.24	8.62E-15
15	4.74	0.21	3.21E-13

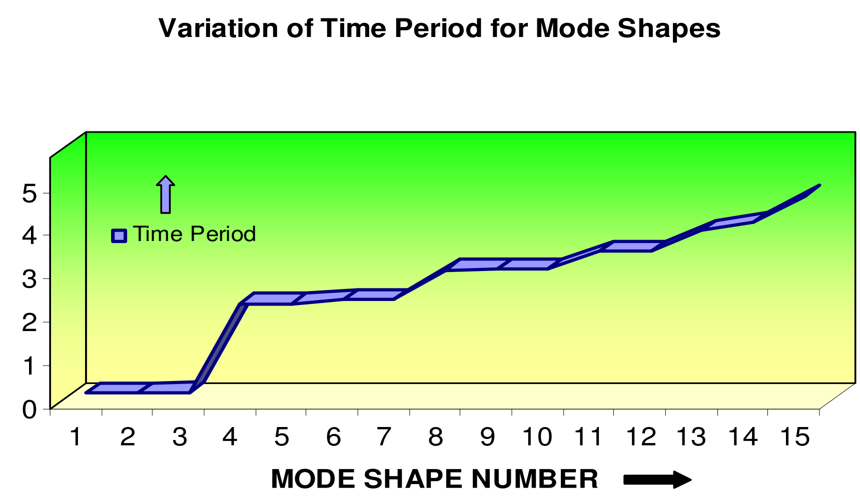


FIGURE 5.4 VARIATIONS OF FREQUENCIES FOR DIFFERENT MODES

TABLE 5.2 BASE SHEAR RESULTS

Dynamic Analysis Output		
	Method	**Base Shear in kN**
1	**SRSS**	**382.72**
2	ABS	446.45
3	CQC	382.78
Manual (Proposed code)		
	Case	**Base Shear in kN**
1	Impulsive	268.66
2	Convective	251.10
3	**SRSS**	**349.15**

5.5 CONCLUSION

- Dynamic analysis results of Base shear are comparable with manual calculation as per Proposed Draft validates base shear calculation.

Note: The consideration of hydrostatic pressure is avoided at this stage of modeling due to some of complexities like when Water get sloshed then it will loose its contact with container wall. The reference input file is attached in soft copy in CD submitted.

6. ESTIMATION AND COST ANALYSIS

6.1 CALCULATION OF QUANTITIES IN m^3

Type of braced frame : **Peripherally braced frame**
Code referred : IS1893-2005 (Part II)Draft code

For columns

Level	Width	Length	Main dia	No.s	% Steel	Main steel	Stirrups	Concrete
1st	550	800	25	22	2.25	0.246	0.180	10.912
2nd	550	800	25	18	2	0.306	0.138	15.277
3rd	550	800	25	18	1.9	0.290	0.138	15.277
4th	550	800	25	18	1.9	0.290	0.138	15.277
5th	550	800	25	16	1.7	0.260	0.138	15.277
6th	550	800	25	14	1.5	0.229	0.115	15.277
7th	550	800	25	14	1.5	0.229	0.115	15.277
8th	550	800	25	14	1.45	0.222	0.115	15.277
Total main steel & stirrups steel in m3 for given no. of columns						**2.07**	**1.08**	**117.85**

For Bracings

Length of bracing = 4630 mm

Level	b	D	main dia	no.s	% steel	reqd %	Main steel	stirrups	conc m^3
1st	250	400	20	4	2.72	1.18	0.044	0.0097	3.704
2nd	300	480	20	6	2.726	1.21	0.065	0.0094	5.334
3rd	300	500	25	4	1.778	1.21	0.067	0.0096	5.556
4th	300	550	25	5	1.778	1.22	0.075	0.0103	6.112
5th	300	550	25	5	1.778	1.22	0.075	0.0103	6.112
6th	300	550	25	5	1.778	1.22	0.075	0.0103	6.112
7th	300	480	20	6	1.778	1.21	0.065	0.0094	5.334
					Total values in m3		**0.464**	**0.0689**	**38.262**

Type of braced frame : **Peripherally & Diagonally braced frame**
Code referred : IS1893-2005 (Part II)Draft code

For columns

Level	Width	Length	Main dia	No.s	% Steel	Main steel	Stirrups	Concrete
1st	550	800	25	22	2.25	0.246	0.180	10.912
2nd	550	800	25	20	2.2	0.336	0.158	15.277
3rd	550	800	25	18	2	0.306	0.138	15.277
4th	550	800	25	18	2	0.306	0.138	15.277
5th	550	800	25	18	2	0.306	0.138	15.277
6th	550	800	25	16	1.75	0.267	0.138	15.277
7th	550	800	25	14	1.5	0.229	0.115	15.277
8th	550	800	25	14	1.5	0.229	0.115	15.277
Total main steel & stirrups steel in m3 for given no. of columns						**2.22**	**1.12**	**117.85**

For Bracings

Length of bracing = 4630 mm

Level	b	D	main dia	no.s	% steel	reqd %	Main steel	stirrups	conc m^3
1st	230	350	25	2	2.336	1.16	0.035	0.0098	2.982
2nd	300	380	25	3	1.363	1.179	0.050	0.0087	4.223
3rd	300	380	25	3	1.778	1.179	0.050	0.0087	4.223
4th	300	400	25	3	1.778	1.185	0.053	0.0006	4.445
5th	300	400	25	3	1.778	1.185	0.053	0.0006	4.445
6th	300	400	25	3	1.778	1.185	0.053	0.0006	4.445
7th	300	380	25	3	1.778	1.179	0.050	0.0087	4.223
					Total values in m3		**0.342**	**0.0377**	**28.984**

For Diagonal Bracings

Length of diagonal = 6.34 m + lap length = 6.34 + 47x20x2 =8.22 m , no.s = 112, dia 20 mm
so volume in m^3 = 0.289 m^3 and weight = 0.289*7.83 ton= 2.263 ton so cost = 2.263x 35000= **Rs. 79205/-**

6.2 CALCULATION OF STEEL FOR LATERAL TIES FOR COLUMNS

No.of bars along length	**6**	No.s
Clear cover	**40**	mm
Clear spacing c/c of stirrups	**142.4**	mm
Stirrup dia	**8**	mm
No.s of columns	**8**	mm
Length of column	**4340**	mm
Spacing of stirrups	**300**	mm
No.s of stirrups	**15**	No.s
Area of stirrup	**50.24**	mm^2

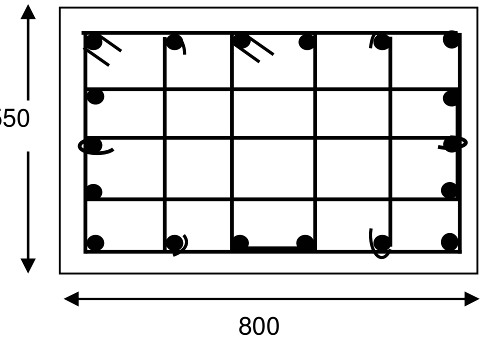
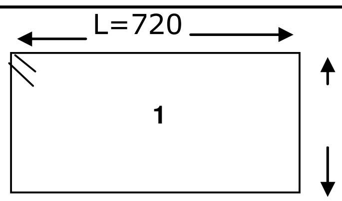
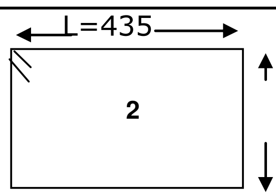
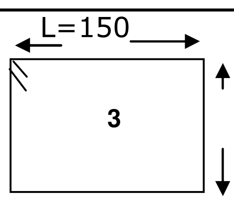
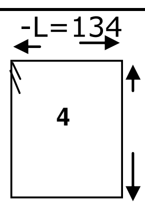
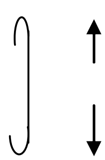
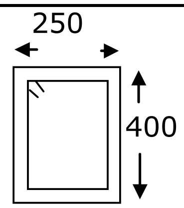
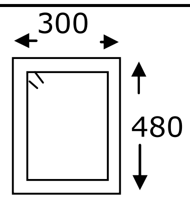
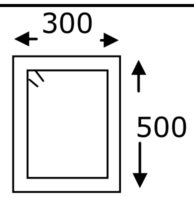
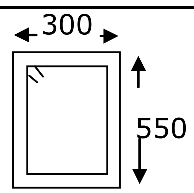
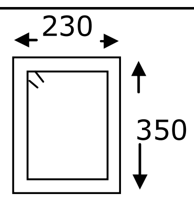
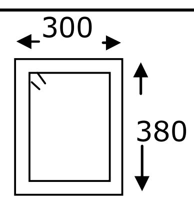
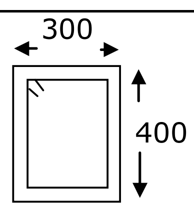


Diagram	Length of stirrup in mm	Total length	Presence is yes 1 if no then put 0
 h=462	2364.01	18912.08	1
 h=462	1794.41	14355.28	1
 h=462	1224.81	9798.48	1
 h=462	655.21	5241.68	0
5  h=462	462.01	3696.08	0

Total volume of stirrup steel for given no.s of columns in m^3 **0.017309**

6.3 CALCULATION OF STEEL FOR STIRRUPS OF BRACINGS

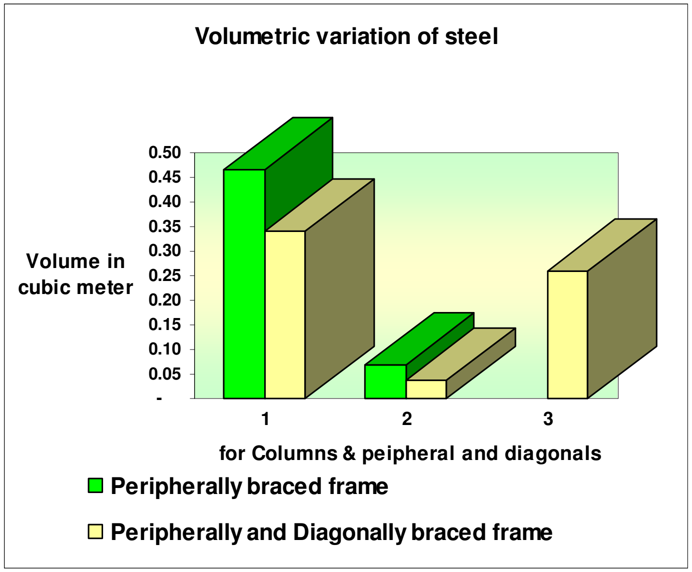
Clear cover =	25	mm
No.s of bracings	8	No.s
length of bracings	4630	mm
Dia of stirrups	8	mm

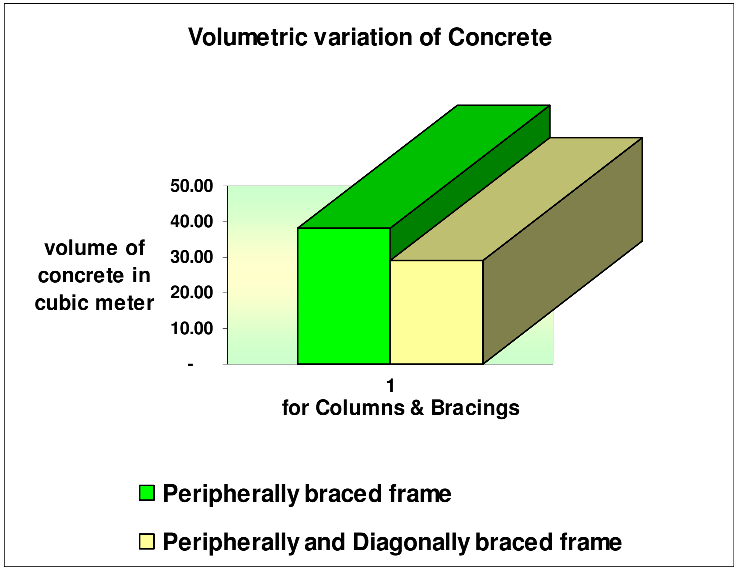
Diagram	Length of stirrups in mm	Total length for columns	No.s oF Stirrups	Quantity in m^3
	1200.01	9600.08	20	0.00965
	1460.01	11680.08	16	0.00939
	1500.01	12000.08	16	0.00965
	1600.01	12800.08	16	0.01029
	1160.01	9280.08	21	0.00979
	1360.01	10880.08	16	0.00875
	1400.01	11200.08	16	0.00900

6.4 COST COMPARISON FOR TRESTLES WITH DIFFERENT BRACING SYSTEMS

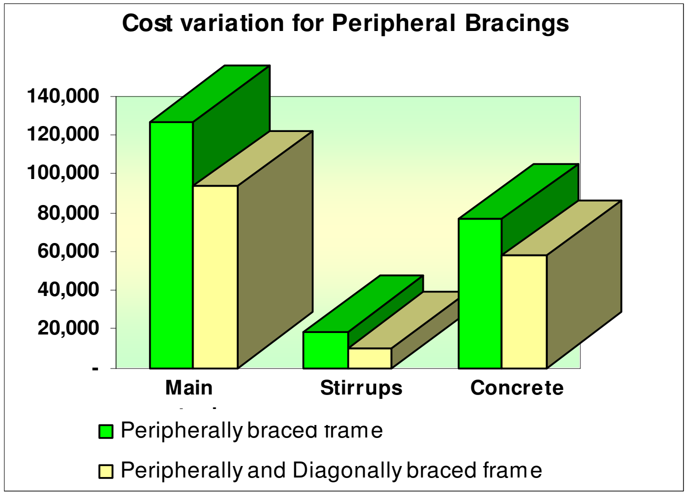
			Peripherally braced frame		Diagonally braced frame	
		Rate	in m^3	Cost	in m^3	Cost
For columns	Main Steel	35000 / tonne	2.071	567,584.30	2.22	609,450.37
	Stirrups	35000 / tonne	1.080	295,974.00	1.12	307,615.64
	Concrete	2000 / m3	117.850	235,699.20	117.85	235,699.20
				1,099,257.50		**1,152,765.21**
		Rate	in m^3	Cost	in m^3	Cost
For Bracings	Main Steel	35000 / tonne	0.464	127,159.20	0.342	93725.1
	Stirrups	35000 / tonne	0.069	18,882.05	0.0377	10331.685
	Concrete	2000 / m3	38.262	76,524.00	28.984	57,968.00
				222,565.25		**162024.785**
		Total cost of Staging :-		**1,321,822.74**		**1,314,790.00**
For Diagonals	Bracings	35000 / tonne	—	—	0.26	79100.00
		Final cost of staging:-		**1,321,822.74**		**1,393,890.00**

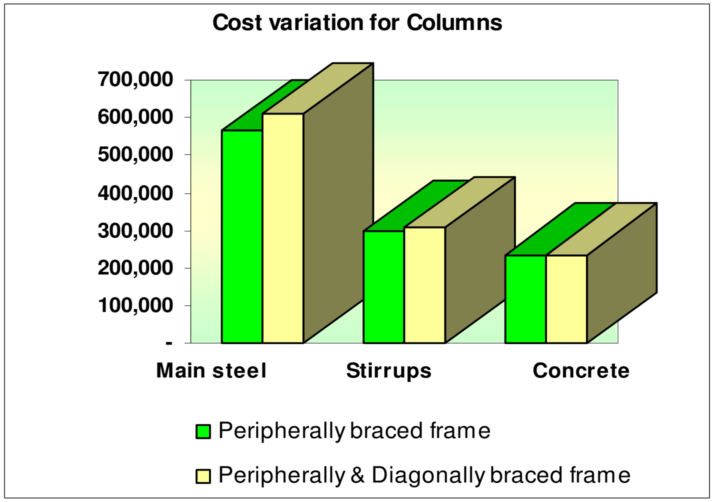
6.5 VOLUMETRIC COMPARISON FOR TRESTLES IN GRAPHICAL FORM

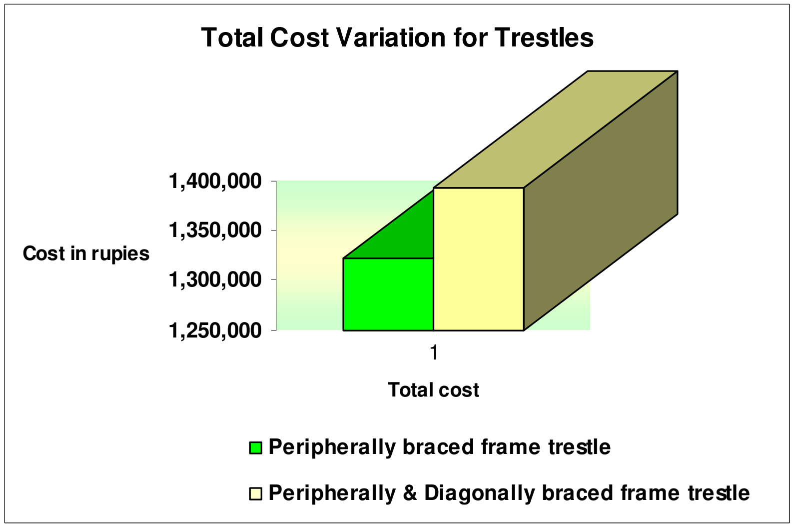




6.6 COST COMPARISON FOR TRESTLES IN GRAPHICAL FORM (cost is in Rupees)







6.7 CONCLUSION

The graph shows clearly that when steel diagonals are provided then the moment variation in peripheral bracings becomes nearly of equal value in case of intermediate panels but the axial forces in columns are increasing considerably, resulting heavy column sections for the trestle with steel diagonals also heavy foundation than conventional peripherally braced trestle.

Also the volume of steel required for diagonals is approximately 70 to 80 % that of overall steel requirement thus it makes trestle uneconomical in cost criteria.

But if we consider efficient working of Diagonally and Peripherally braced trestle during Earthquake then this initial investment is safer than failure of structure.

7. CONCLUSIONS

7.1 PROPOSED CODE COMPARISION

The equation for Base shear by any code results into following form

$$\mathbf{V} = \mathbf{C.W}$$

Where,

C = Coefficient (Depending upon Importance, Location and some Structural Properties (distribution of mass and stiffness)) and

W = Seismic Weight (W = mg; m = mass which participates in vibration)

The main differences between *IS: 1983-1984* and *IS: 1893-2005* (Part 2) Proposed Draft code are as follows

IS: 1893-1984

In this code, **C** the coefficient was not dependent upon Ductility of the supporting system. Factor (Sa/g) was being determined from Natural Period.

In IS: 1893-1984 code there was no formula for trestle lateral stiffness is provided so staging was assumed to be stiffer as per *SP: 22* explanatory examples for simplicity of calculations but in Proposed draft code flexibility of frame staging is considered.

Also in *IS: 1893-1984* code **W** i.e. mass participating in vibration was assumed to be the total water mass in the container which is not rational as some part of volume of water in container retain itself with container during vibrations and some portion does not take part during vibration of water container.

Proposed Draft for IS: 1893 (Part II)

In proposed draft code **W** i.e. mass participating in vibration is assumed to be consisting of the Impulsive as well as Convective water mass in the container. **W** i.e. mass participating in vibration is being considered **lower** than assumptions of *IS: 1893-1984* code.

In Proposed Draft, the major reason for **High** Design Seismic Forces is "to account for redundancy, ductility and overstrength of the supporting system of tanks by the value of R = 2.5"... This means tanks are expected to be capable of dissipating the energy by half a margin as compared to building frames detailed for ductility. (For SMRF, R=5.0 in building)

CONCLUSIONS REGARDING EARTHQUAKE AND WIND DRAFT CODE

After referring Proposed seismic draft code the base shear values for considered structure is found to be approximately 10 to 40 % more than the base shear values for the same structure obtained by referring *IS: 1893-1984* code in case of impulsive base shear and on applying SRSS rule for design base shear.
Above conclusions are validated by the results of case study problem as discussed in chapter 4.

The increase in base shear for Peripherally braced trestle is about 15 % whereas it is nearly 40% for peripherally and Diagonally braced trestle.

For Wind draft code the values for wind design forces are increasing by 10-15 % for staging while there is increase of 20-25 % in design values for container this is on result of inclusion of shape factors for intze container in draft code.

Special factor of safety for storms is considered in Wind draft code

CONCLUSIONS REGARDING STEEL DIAGONAL BRACINGS

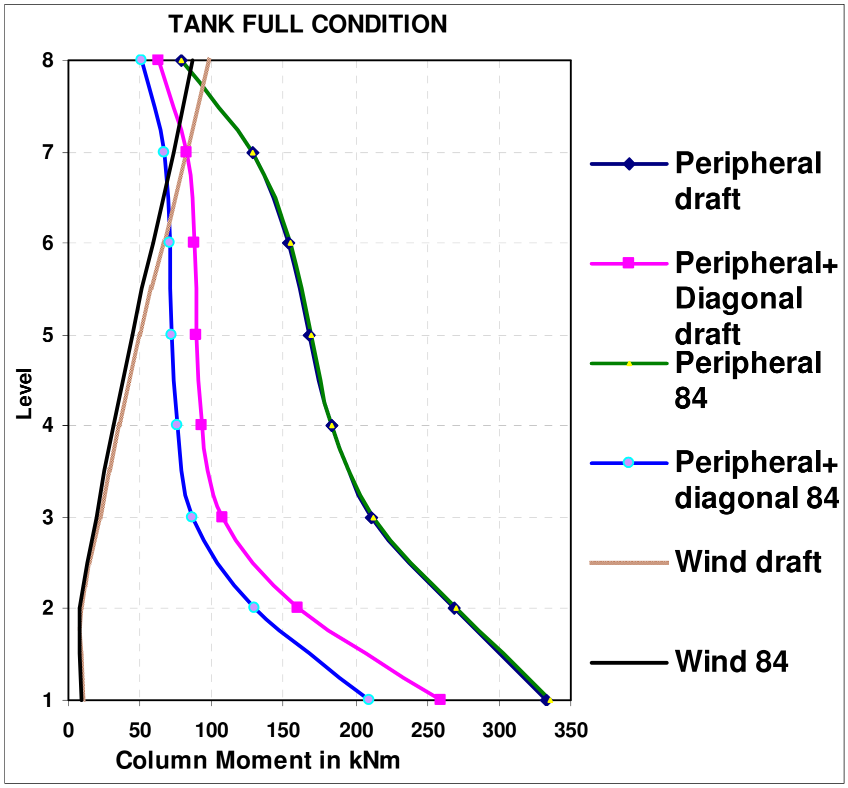
When we use steel diagonals with Reinforced concrete frames then the 'Truss action' is incorporated in Frame action making stiffer composite staging.
The following are the some of the advantages of Diagonal bracings
1. Diagonal bracings are characterised by extensive yielding in tension and inelastic buckling of bracings.

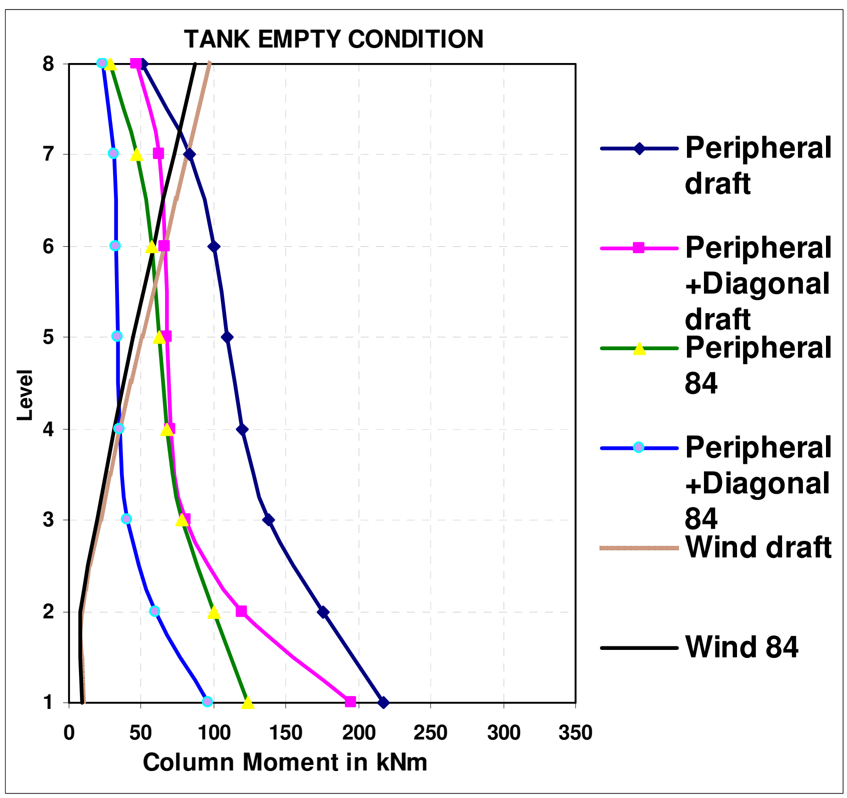
2. The story drift is also controlled in case of diagonal bracings. Due to diagonal bracings axial forces in columns increases considerably so we have to do extra strengthening of columns and footing to avoid premature failure.

3. As the cost of retrofitting with the use Diagonal bracings is cheaper than the cost required for dismantling old frame structure and new construction together we can conclude that Steel Diagonals can be effectively used for retrofitting.

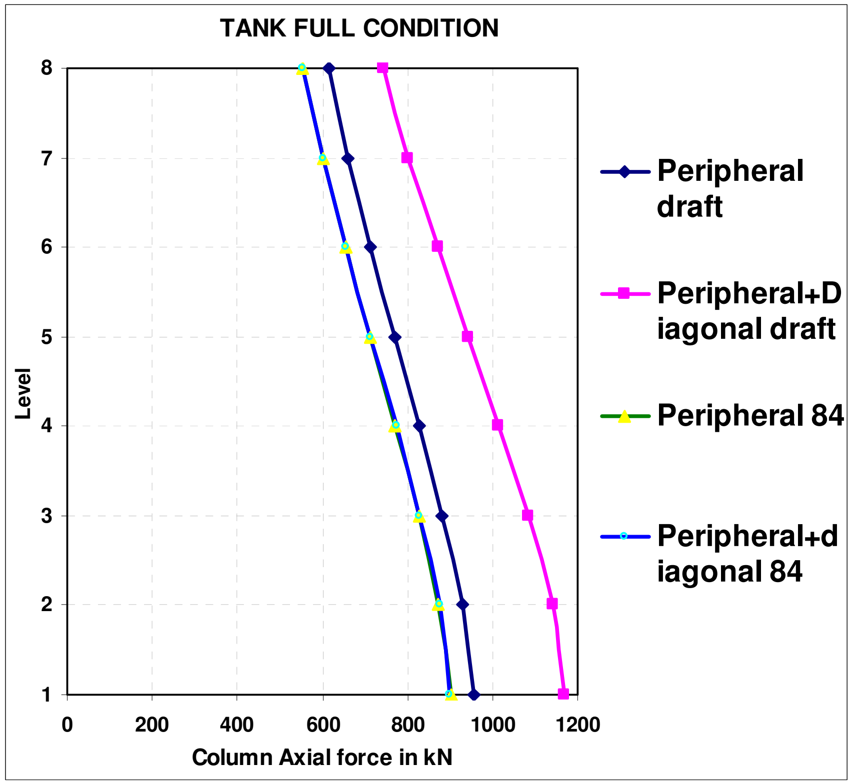
7.2 GRAPHICAL REPRESENTATIONS

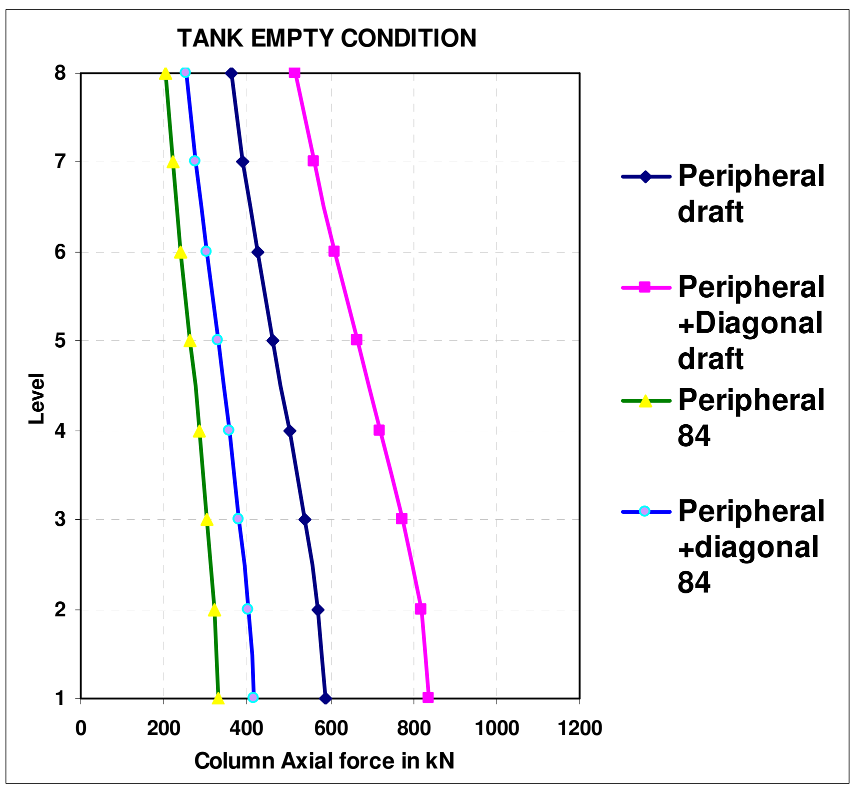
7.2.1 COLUMN MOMENT VARIATION



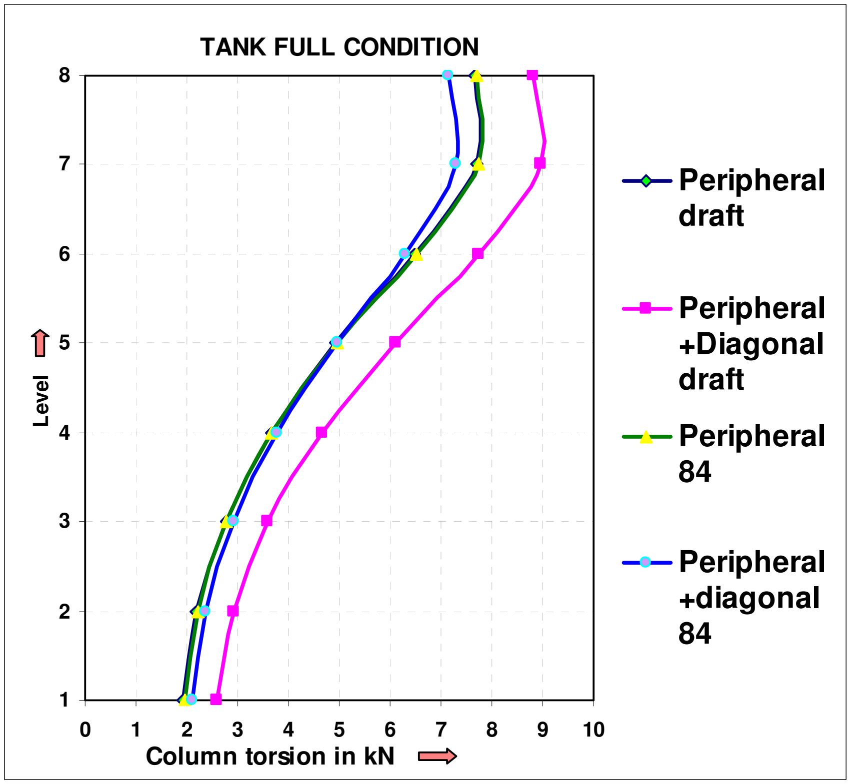


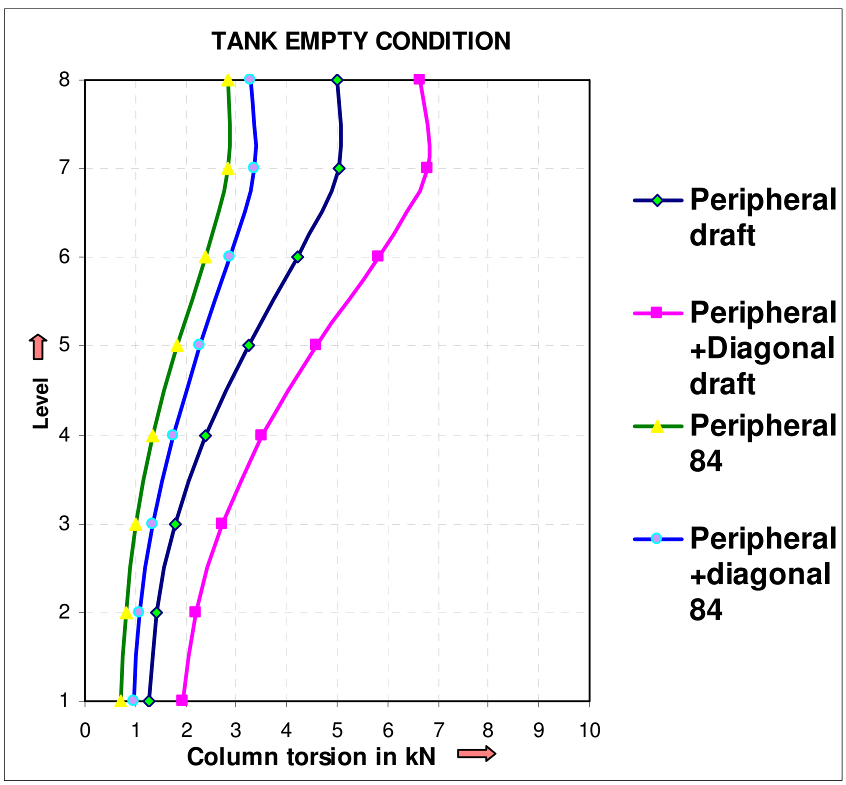
7.2.2 COLUMN AXIAL FORCE VARIATION



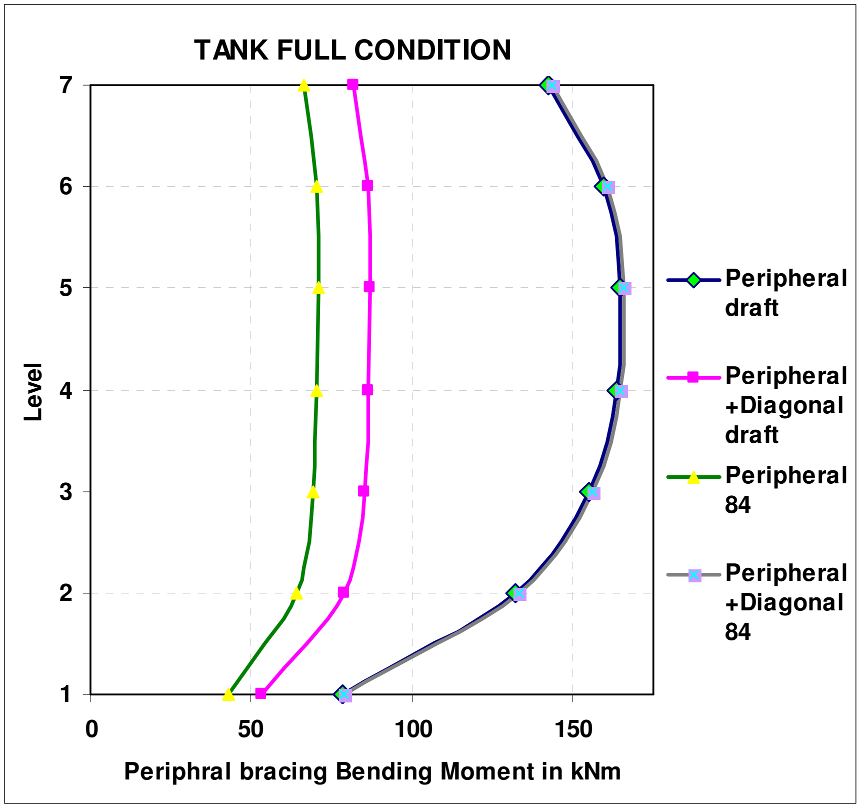


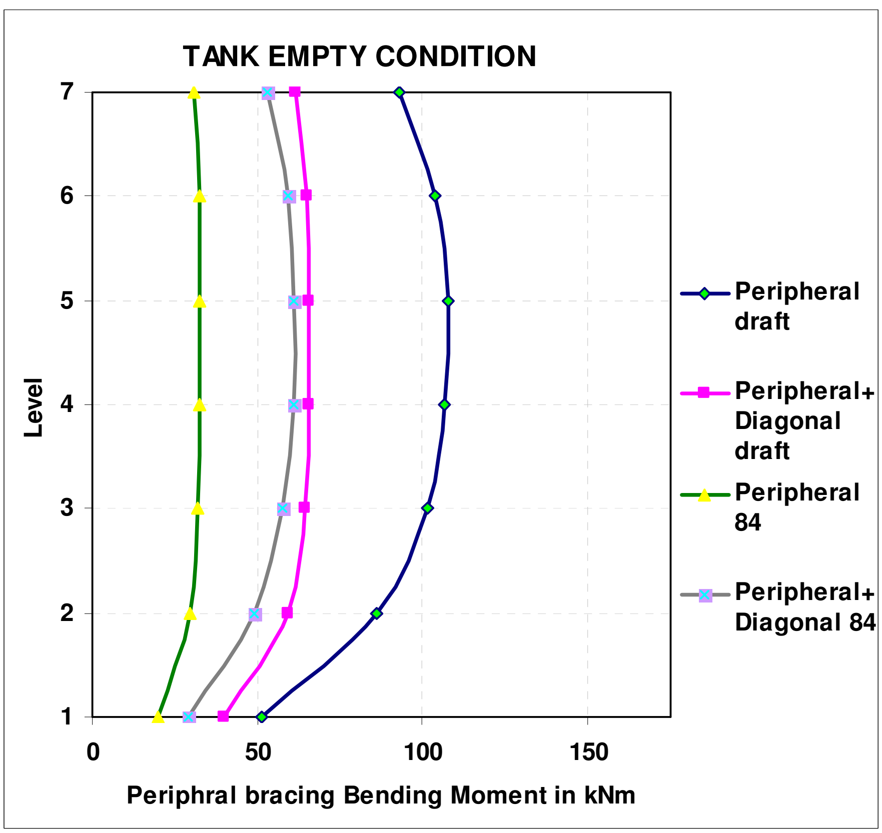
7.2.3 COLUMN TORSION VARIATION



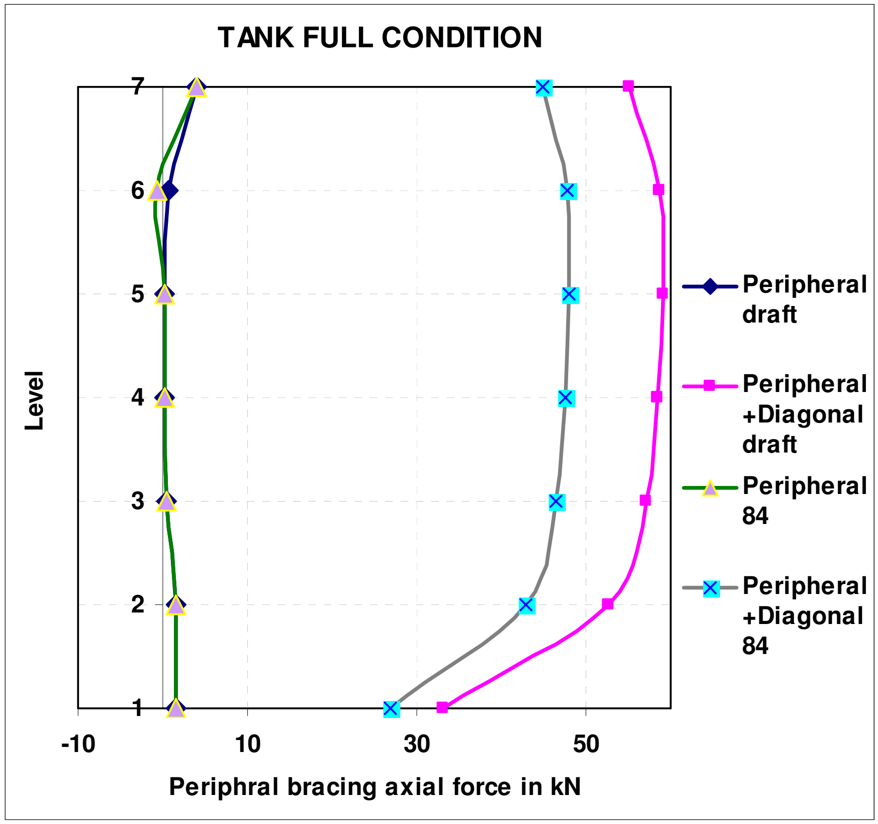


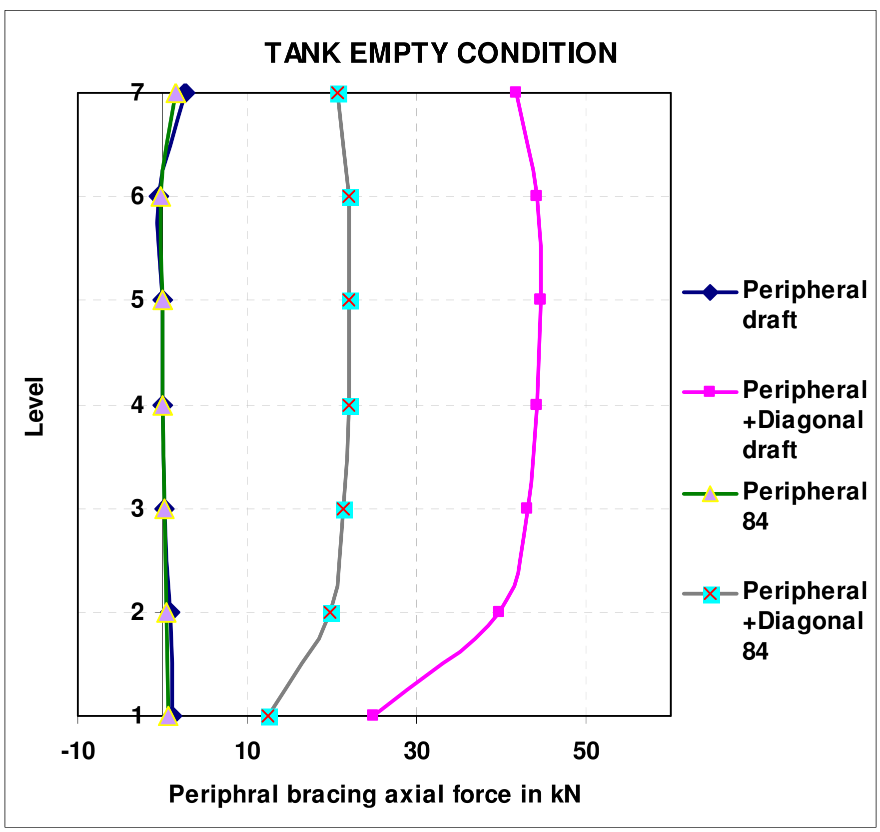
7.2.4 PERIPHERAL BRACINGS MOMENT VARIATION





7.2.5 PERIPHERAL BRACINGS' AXIAL FORCE VARIATION





7.3 CONCLUDING REMARKS

Comparison between column moments

The values of column moments are reduced to nearly 80 % of value in case of tank full condition for diagonally braced trestle than the conventionally braced trestle.

We can find overall equal moment distribution for the intermediate peripheral bracings which are combined with Diagonal bracings.

Whereas the sole peripheral bracings' moment varies from all levels

Here note that the lateral stiffness of staging for both calculations may be for IS: 1893-1984 or for Proposed draft are carried out considering Flexibility of structure.

In case of Diagonally and Peripherally braced trestle the column moments are comparatively high for the bottommost and first story column which shows necessity to have control on drift in these areas.

Column axial forces

If we see the graphs for column axial forces the value in case of Diagonally braced trestle the values are approx increasing by 30% than the Peripherally braced trestle, this is the result of Truss Action which is incorporated with Frame action which reduces moments in columns but increases Axial force in columns due to diagonal bracings which are acting like tension members of a Truss.

Compared with *IS: 1893-1984* the values for column axial forces are increasing by 20% approximately.

Column torsion

The column torsion is increases from bottom towards upwards portion of staging.

Thus torsion is more critical for upper stories of staging; compared to earlier code value of torsion is increasing by 20% approximately by the use of Proposed Draft code whereas we can find increase in torsion when we use the diagonal bracings additional to Peripheral bracings.

Peripheral Bracings moment variation

The moments are varying largely from position to position in case of solely use of Peripheral bracings, but when we use diagonal bracings additionally with peripheral bracings the moment for intermediately located peripheral bracings is approx same which shows reduction in concrete as this moment is lesser than peripheral trestle and also shows approx same section of concrete can be used for intermediate peripheral bracings.

Peripheral bracings axial force

Due to truss action resulting by the use of diagonal bracings; the peripheral bracings are subjected with axial forces which are nearly 30 to 50 % greater than the trestle without Diagonal bracings. Thus proper check regarding axial force of peripheral braces should be considered because concrete is weak in tension.

7.4 FUTURE SCOPE OF WORK

For the Draft code comparison we can do lot of work which includes

1. Study of foreign codes and their comparison with draft code for reviews and commentary
2. Dynamic analysis for sloshing effect and draft code results
3. Most of the tanks are having obstructions like inside columns in tank container, which will reduce sloshing mass this study should be carried out to judge exact amount of impulsive and convective mass
4. The safety of tank is generally checked for tank full and tank empty conditions, studies regarding similar exercises can be carried out for partial full conditions. To get probable critical design values for tank design.

The studies pertaining to different bracing systems are carried out in this piece of work. For the same topic following much more work can be carried out;

1. Study regarding variations like increase in diagonal steel or reduction which will effect on base shear should be studied to arrive at optimal solution of design of trestle with diagonal bracings.
2. Working of diagonal bracings, their safety and function should be checked with some criteria to assure the proper working. This study should be included as energy dissipation capacity is more in steel diagonals during vibrations.
3. Using Diagonal bracings retrofitting is better as diagonals are fitted in existing RC members it won't create much problem for retrofitting than other methods provided original beam column junctions are properly designed, Detailed and constructed. such studies for cross bracings can be carried out and the tables for ratio like stiffness of trestle and concrete can be done to have optimal solution.

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LATERAL LOAD DISTRIBUTION ON STAGING

For Proposed Draft code

Lateral Load Distribution For Peripheral Braced Trestle

Column No.	Node No.	Case1 Critical for Column							Node No.	Case 2 Critical for Bracing					
		Axial Force			Shear					Axial Force			Shear		
C1	10016	FY	0.2500	M/r	FX	0.000	S	C1	10020	FY	0.2310	M/r	FX	0.0366	S
C2	10012	FY	0.1768	M/r	FX	0.125	S	C2	10016	FY	0.2310	M/r	FX	0.0366	S
C3	10008	FY	0.0000	M/r	FX	0.250	S	C3	10012	FY	0.0957	M/r	FX	0.2134	S
C4	10004	FY	-0.1768	M/r	FX	0.125	S	C4	10008	FY	-0.0957	M/r	FX	0.2134	S
C5	10032	FY	-0.2500	M/r	FX	0.000	S	C5	10004	FY	-0.2310	M/r	FX	0.0366	S
C6	10028	FY	-0.1768	M/r	FX	0.125	S	C6	10032	FY	-0.2310	M/r	FX	0.0366	S
C7	10024	FY	0.0000	M/r	FX	0.250	S	C7	10028	FY	-0.0957	M/r	FX	0.2134	S
C8	10020	FY	0.1768	M/r	FX	0.125	S	C8	10024	FY	0.0957	M/r	FX	0.2134	S
Total			0.0000	M/r		1.000	S				0.0000	M/r		1.0000	S

Impulsive condition			
	S =	**268.66**	kN
	M =	**9944.8**	kNm
	r =	**6.05**	m

h = (mi(hi*+hs)+mshcg)/(mi+ms)
= 36.989 -31.6
= **5.389** m

Per Tank Full Impulsive Condition

Column No.	Node No.	Case1 Critical for Column					Node No.	Case 2 Critical for Bracing			
		Axial Force		Shear				Axial Force		Shear	
C1	10016	FY	**410.94**	FX	**0.00**	C1	10020	FY	**379.66**	FX	**9.84**
C2	10012	FY	**290.58**	FX	**33.58**	C2	10016	FY	**379.66**	FX	**9.84**
C3	10008	FY	**0.00**	FX	**67.17**	C3	10012	FY	**157.26**	FX	**57.33**
C4	10004	FY	**-290.58**	FX	**33.58**	C4	10008	FY	**-157.26**	FX	**57.33**
C5	10032	FY	**-410.94**	FX	**0.00**	C5	10004	FY	**-379.66**	FX	**9.84**
C6	10028	FY	**-290.58**	FX	**33.58**	C6	10032	FY	**-379.66**	FX	**9.84**
C7	10024	FY	**0.00**	FX	**67.17**	C7	10028	FY	**-157.26**	FX	**57.33**
C8	10020	FY	**290.58**	FX	**33.58**	C8	10024	FY	**157.26**	FX	**57.33**
Total			**0.00**		**268.66**				**0.00**		**268.66**

Convective condition			
	S =	**251.1**	kN
	M =	**9968.7**	kNm
	r =	**6.05**	m

hc* = 8.1 m
= **8.1** m

Per Tank Full Convective Condition

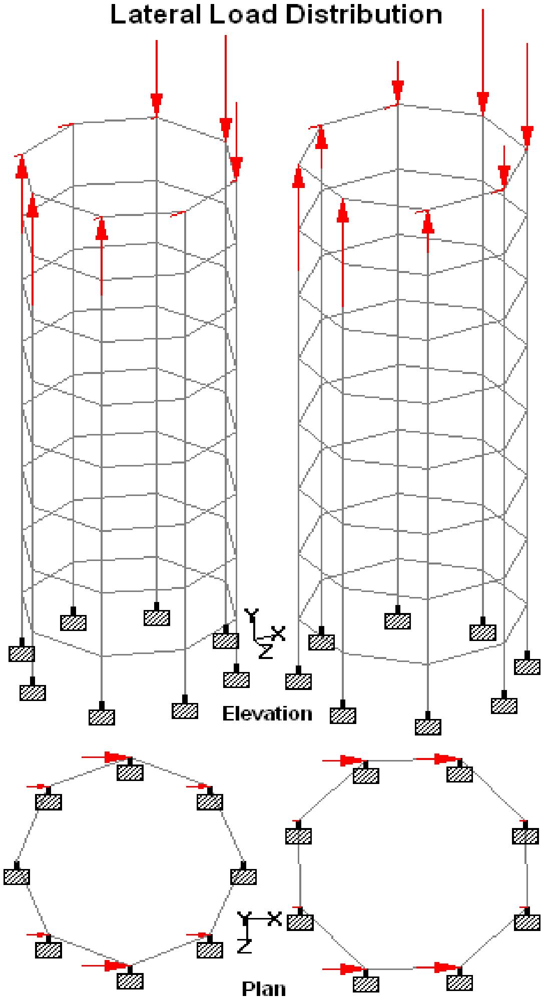
Column No.	Node No.	Case1 Critical for Column					Node No.	Case 2 Critical for Bracing			
		Axial Force		Shear				Axial Force		Shear	
C1	10016	FY	**411.93**	FX	**0.00**	C1	10020	FY	**380.57**	FX	**9.19**
C2	10012	FY	**291.28**	FX	**31.39**	C2	10016	FY	**380.57**	FX	**9.19**
C3	10008	FY	**0.00**	FX	**62.78**	C3	10012	FY	**157.64**	FX	**53.58**
C4	10004	FY	**-291.28**	FX	**31.39**	C4	10008	FY	**-157.64**	FX	**53.58**
C5	10032	FY	**-411.93**	FX	**0.00**	C5	10004	FY	**-380.57**	FX	**9.19**
C6	10028	FY	**-291.28**	FX	**31.39**	C6	10032	FY	**-380.57**	FX	**9.19**
C7	10024	FY	**0.00**	FX	**62.78**	C7	10028	FY	**-157.64**	FX	**53.58**
C8	10020	FY	**291.28**	FX	**31.39**	C8	10024	FY	**157.64**	FX	**53.58**
Total			**0.00**		**251.10**				**0.00**		**251.10**

Tank empty condition	S =	216.78 kN
	M =	7615.7 kNm
	r =	6.05 m

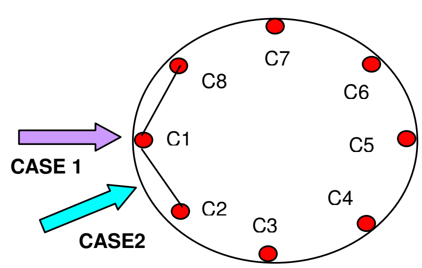
hcg= 35.071 -31.6
hcg= **3.471** m

Per Tank Empty Condition													
Column No.	Node No.	Case1 Critical for Column					Node No.	Case 2 Critical for Bracing					
		Axial Force		Shear				Axial Force		Shear			
C1	10016	FY	**314.70**	FX	**0.00**	C1	10020	FY	**290.74**	FX	**7.94**		
C2	10012	FY	**222.52**	FX	**27.10**	C2	10016	FY	**290.74**	FX	**7.94**		
C3	10008	FY	**0.00**	FX	**54.20**	C3	10012	FY	**120.43**	FX	**46.26**		
C4	10004	FY	**-222.52**	FX	**27.10**	C4	10008	FY	**-120.43**	FX	**46.26**		
C5	10032	FY	**-314.70**	FX	**0.00**	C5	10004	FY	**-290.74**	FX	**7.94**		
C6	10028	FY	**-222.52**	FX	**27.10**	C6	10032	FY	**-290.74**	FX	**7.94**		
C7	10024	FY	**0.00**	FX	**54.20**	C7	10028	FY	**-120.43**	FX	**46.26**		
C8	10020	FY	**222.52**	FX	**27.10**	C8	10024	FY	**120.43**	FX	**46.26**		
Total			**0.00**		**216.78**				**0.00**		**216.78**		

Lateral Load Distribution



Axial force on each Column				
1	**Tank full Impulsive condition**			
	Impulsive load	=	**-2036.27**	kN
	Live Load	=	**-43.65**	kN
2	**Tank full Convective condition**			
	Convective load	=	**-1528.13**	kN
	Live Load	=	**-43.65**	kN
3	**Tank Empty condition**			
	Axial load	=	**-1343.27**	kN
	Live Load	=	**-43.65**	kN



For IS: 1893-1984

Lateral Load Distribution For Peripheral Braced Trestle 1893-84

Column No.	Node No.	Case1 Critical for Column							Node No.	Case 2 Critical for Bracing					
		Axial Force			Shear					Axial Force			Shear		
C1	10016	FY	0.2500	M/r	FX	0.000	S	C1	10020	FY	0.2310	M/r	FX	0.0366	S
C2	10012	FY	0.1768	M/r	FX	0.125	S	C2	10016	FY	0.2310	M/r	FX	0.0366	S
C3	10008	FY	0.0000	M/r	FX	0.250	S	C3	10012	FY	0.0957	M/r	FX	0.2134	S
C4	10004	FY	-0.1768	M/r	FX	0.125	S	C4	10008	FY	-0.0957	M/r	FX	0.2134	S
C5	10032	FY	-0.2500	M/r	FX	0.000	S	C5	10004	FY	-0.2310	M/r	FX	0.0366	S
C6	10028	FY	-0.1768	M/r	FX	0.125	S	C6	10032	FY	-0.2310	M/r	FX	0.0366	S
C7	10024	FY	0.0000	M/r	FX	0.250	S	C7	10028	FY	-0.0957	M/r	FX	0.2134	S
C8	10020	FY	0.1768	M/r	FX	0.125	S	C8	10024	FY	0.0957	M/r	FX	0.2134	S
Total			0.0000	M/r		1.000	S				0.0000	M/r		1.0000	S

Tank full condition			
	S =	**353.8**	kN
	M =	**12338**	kNm
	r =	**6.05**	m

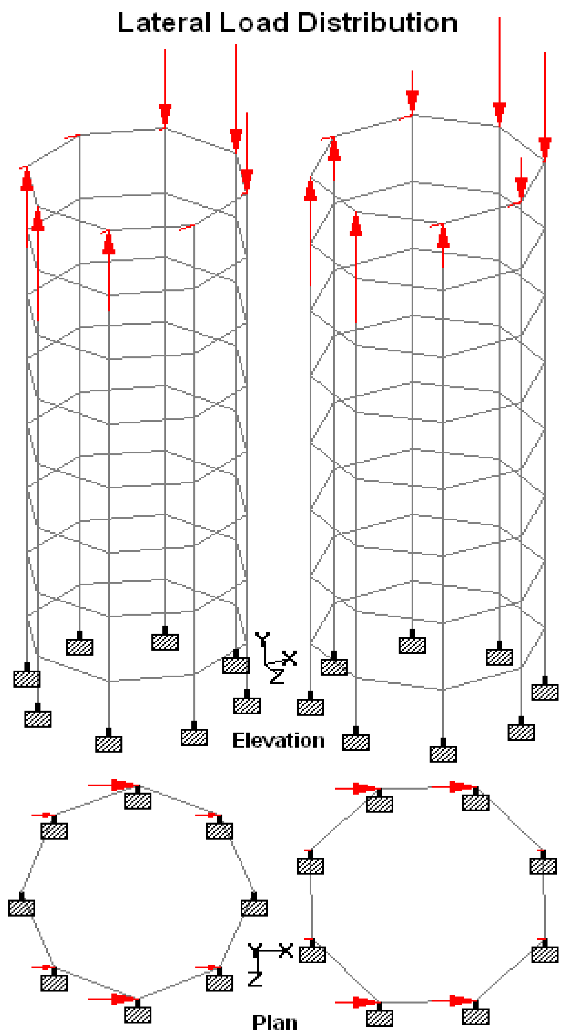
Per Tank Full Condition

Column No.	Node No.	Case1 Critical for Column					Node No.	Case 2 Critical for Bracing			
		Axial Force		Shear				Axial Force		Shear	
C1	10016	FY	**509.83**	FX	**0.00**	C1	10020	FY	**471.03**	FX	**12.95**
C2	10012	FY	**360.51**	FX	**44.23**	C2	10016	FY	**471.03**	FX	**12.95**
C3	10008	FY	**0.00**	FX	**88.45**	C3	10012	FY	**195.11**	FX	**75.50**
C4	10004	FY	**-360.51**	FX	**44.23**	C4	10008	FY	**-195.11**	FX	**75.50**
C5	10032	FY	**-509.83**	FX	**0.00**	C5	10004	FY	**-471.03**	FX	**12.95**
C6	10028	FY	**-360.51**	FX	**44.23**	C6	10032	FY	**-471.03**	FX	**12.95**
C7	10024	FY	**0.00**	FX	**88.45**	C7	10028	FY	**-195.11**	FX	**75.50**
C8	10020	FY	**360.51**	FX	**44.23**	C8	10024	FY	**195.11**	FX	**75.50**
Total			**0.00**		**353.80**				**0.00**		**353.80**

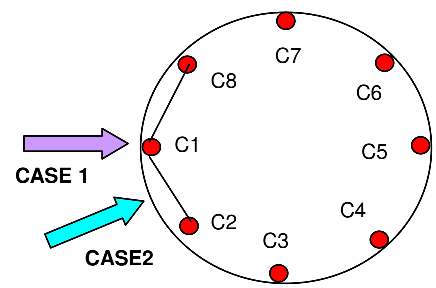
Tank empty condition					
	S =	**130.3**	kN	hcg=	35.071 -31.6
	M =	**4544**	kNm	hcg=	**3.471** m
	r =	**6.05**	m		

Per Tank Empty Condition

Column No.	Node No.	Case1 Critical for Column					Node No.	Case 2 Critical for Bracing			
		Axial Force		Shear				Axial Force		Shear	
C1	10016	FY	**187.77**	FX	**0.00**	C1	10020	FY	**173.48**	FX	**4.77**
C2	10012	FY	**132.77**	FX	**16.29**	C2	10016	FY	**173.48**	FX	**4.77**
C3	10008	FY	**0.00**	FX	**32.58**	C3	10012	FY	**71.86**	FX	**27.80**
C4	10004	FY	**-132.77**	FX	**16.29**	C4	10008	FY	**-71.86**	FX	**27.80**
C5	10032	FY	**-187.77**	FX	**0.00**	C5	10004	FY	**-173.48**	FX	**4.77**
C6	10028	FY	**-132.77**	FX	**16.29**	C6	10032	FY	**-173.48**	FX	**4.77**
C7	10024	FY	**0.00**	FX	**32.58**	C7	10028	FY	**-71.86**	FX	**27.80**
C8	10020	FY	**132.77**	FX	**16.29**	C8	10024	FY	**71.86**	FX	**27.80**
Total			**0.00**		**130.30**				**0.00**		**130.30**



	Axial force on each Column			
1	**Tank full condition**			
	Impulsive load	=	**-3943.9**	kN
	Live Load	=	**-43.65**	kN
2	**Tank Empty condition**			
	Axial load	=	**-1614.79**	kN
	Live Load	=	**-43.65**	kN



For Proposed Draft Code

Lateral Load Distribution For Peripheral and Diagonal Braced Trestle

Colum n No.	Node No.	Case1 Critical for Column					Node No.	Case 2 Critical for Bracing			
		Axial Force		Shear				Axial Force		Shear	
C1	10016	FY	0.2500 M/r	FX	0.000 S	C1	10020	FY	0.2310 M/r	FX	0.0366 S
C2	10012	FY	0.1768 M/r	FX	0.125 S	C2	10016	FY	0.2310 M/r	FX	0.0366 S
C3	10008	FY	0.0000 M/r	FX	0.250 S	C3	10012	FY	0.0957 M/r	FX	0.2134 S
C4	10004	FY	-0.1768 M/r	FX	0.125 S	C4	10008	FY	-0.0957 M/r	FX	0.2134 S
C5	10032	FY	-0.2500 M/r	FX	0.000 S	C5	10004	FY	-0.2310 M/r	FX	0.0366 S
C6	10028	FY	-0.1768 M/r	FX	0.125 S	C6	10032	FY	-0.2310 M/r	FX	0.0366 S
C7	10024	FY	0.0000 M/r	FX	0.250 S	C7	10028	FY	-0.0957 M/r	FX	0.2134 S
C8	10020	FY	0.1768 M/r	FX	0.125 S	C8	10024	FY	0.0957 M/r	FX	0.2134 S
Total			0.0000 M/r		1.000 S				0.0000 M/r		1.0000 S

	S =	**377.86** kN	h = (mi(hi*+hs)+mshcg)/(mi
Impulsive condition	**M =**	**13987** kNm	= 36.989 -31.6
	r =	**6.05** m	= **5.389** m

Peripheral and Diagonal Braced Tank Full Impulsive Condition											
Colum n No.	Node No.	Case1 Critical for Column					Node No.	Case 2 Critical for Bracing			
		Axial Force		Shear				Axial Force		Shear	
C1	10016	FY	**577.98**	FX	**0.00**	C1	10020	FY	**533.98**	FX	**13.83**
C2	10012	FY	**408.69**	FX	**47.23**	C2	10016	FY	**533.98**	FX	**13.83**
C3	10008	FY	**0.00**	FX	**94.47**	C3	10012	FY	**221.18**	FX	**80.63**
C4	10004	FY	**-408.69**	FX	**47.23**	C4	10008	FY	**-221.18**	FX	**80.63**
C5	10032	FY	**-577.98**	FX	**0.00**	C5	10004	FY	**-533.98**	FX	**13.83**
C6	10028	FY	**-408.69**	FX	**47.23**	C6	10032	FY	**-533.98**	FX	**13.83**
C7	10024	FY	**0.00**	FX	**94.47**	C7	10028	FY	**-221.18**	FX	**80.63**
C8	10020	FY	**408.69**	FX	**47.23**	C8	10024	FY	**221.18**	FX	**80.63**
Total			**0.00**		**377.86**				**0.00**		**377.86**

	S =	**251.1** kN	hc* = 8.1 m
Convective condition	**M =**	**9968.7** kNm	= **8** m
	r =	**6.05** m	

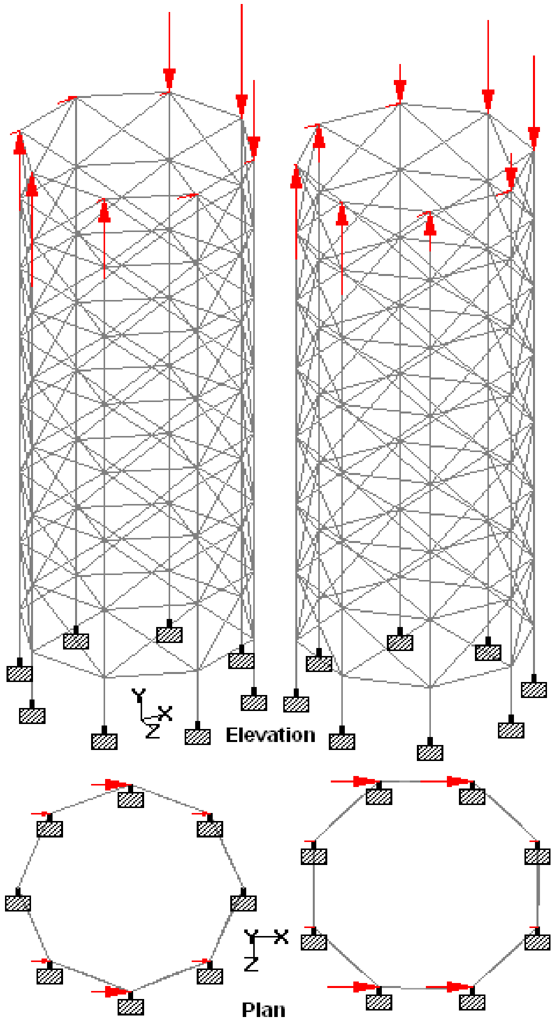
Peripheral and Diagonal Braced Tank Full Convective Condition											
Colum n No.	Node No.	Case1 Critical for Column					Node No.	Case 2 Critical for Bracing			
		Axial Force		Shear				Axial Force		Shear	
C1	10016	FY	**411.93**	FX	**0.00**	C1	10020	FY	**380.57**	FX	**9.19**
C2	10012	FY	**291.28**	FX	**31.39**	C2	10016	FY	**380.57**	FX	**9.19**
C3	10008	FY	**0.00**	FX	**62.78**	C3	10012	FY	**157.64**	FX	**53.58**
C4	10004	FY	**-291.28**	FX	**31.39**	C4	10008	FY	**-157.64**	FX	**53.58**
C5	10032	FY	**-411.93**	FX	**0.00**	C5	10004	FY	**-380.57**	FX	**9.19**
C6	10028	FY	**-291.28**	FX	**31.39**	C6	10032	FY	**-380.57**	FX	**9.19**
C7	10024	FY	**0.00**	FX	**62.78**	C7	10028	FY	**-157.64**	FX	**53.58**
C8	10020	FY	**291.28**	FX	**31.39**	C8	10024	FY	**157.64**	FX	**53.58**
Total			**0.00**		**251.10**				**0.00**		**251.10**

Tank Empty condition	S =	306.94	kN
	M =	10783	kNm
	r =	6.05	m

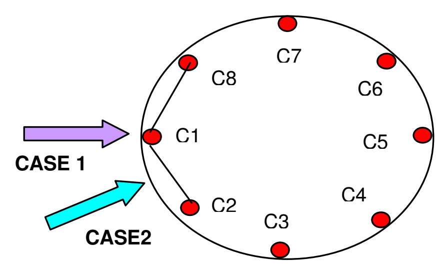
hcg= 35.071 -31.6
hcg= **3.471** m

Peripheral and Diagonal Braced Tank Empty Condition											
Column No.	Node No.	Case1 Critical for Column					Node No.	Case 2 Critical for Bracing			
		Axial Force		Shear				Axial Force		Shear	
C1	10016	FY	**445.57**	FX	**0.00**	C1	10020	FY	**411.65**	FX	**11.24**
C2	10012	FY	**315.07**	FX	**38.37**	C2	10016	FY	**411.65**	FX	**11.24**
C3	10008	FY	**0.00**	FX	**76.74**	C3	10012	FY	**170.51**	FX	**65.50**
C4	10004	FY	**-315.07**	FX	**38.37**	C4	10008	FY	**-170.51**	FX	**65.50**
C5	10032	FY	**-445.57**	FX	**0.00**	C5	10004	FY	**-411.65**	FX	**11.24**
C6	10028	FY	**-315.07**	FX	**38.37**	C6	10032	FY	**-411.65**	FX	**11.24**
C7	10024	FY	**0.00**	FX	**76.74**	C7	10028	FY	**-170.51**	FX	**65.50**
C8	10020	FY	**315.07**	FX	**38.37**	C8	10024	FY	**170.51**	FX	**65.50**
Total			**0.00**		**306.94**				**0.00**		**306.94**

Lateral Load Distribution



Axial force on each Column				
1	**Tank full Impulsive condition**			
	Impulsive load	=	**-2048.08**	kN
	Live Load	=	**-43.65**	kN
2	**Tank full Convective condition**			
	Convective load =	=	**-1531.53**	kN
	Live Load	=	**-43.65**	kN
3	**Tank Empty condition**			
	Axial load =	=	**-1346.68**	kN
	Live Load	=	**-43.65**	kN



For IS: 1893-1984

Lateral Load Distribution For Peripheral and Diagonal Braced Trestle											
Column No.	Node No.	Case1 Critical for Column					Node No.	Case 2 Critical for Bracing			
		Axial Force		Shear				Axial Force		Shear	
C1	10016	FY	0.2500 M/r	FX	0.000 S	C1	10020	FY	0.2310 M/r	FX	0.0366 S
C2	10012	FY	0.1768 M/r	FX	0.125 S	C2	10016	FY	0.2310 M/r	FX	0.0366 S
C3	10008	FY	0.0000 M/r	FX	0.250 S	C3	10012	FY	0.0957 M/r	FX	0.2134 S
C4	10004	FY	-0.1768 M/r	FX	0.125 S	C4	10008	FY	-0.0957 M/r	FX	0.2134 S
C5	10032	FY	-0.2500 M/r	FX	0.000 S	C5	10004	FY	-0.2310 M/r	FX	0.0366 S
C6	10028	FY	-0.1768 M/r	FX	0.125 S	C6	10032	FY	-0.2310 M/r	FX	0.0366 S
C7	10024	FY	0.0000 M/r	FX	0.250 S	C7	10028	FY	-0.0957 M/r	FX	0.2134 S
C8	10020	FY	0.1768 M/r	FX	0.125 S	C8	10024	FY	0.0957 M/r	FX	0.2134 S
Total			0.0000 M/r		1.000 S				0.0000 M/r		1.0000 S

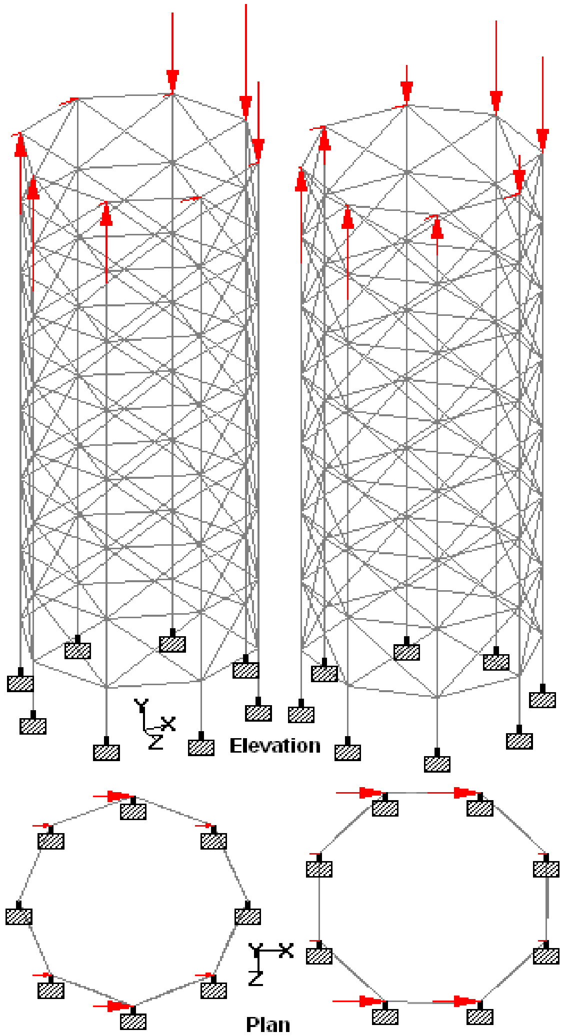
	S =	**353.8** kN	h = (mi(hi*+hs)+mshcg)/(mi+ms)	
Impulsive condition	**M =**	**12338** kNm	=	36.989 -31.6
	r =	**6.05** m	=	**5.389** m

Peripheral and Diagonal Braced Tank Full Impulsive Condition											
Column No.	Node No.	Case1 Critical for Column					Node No.	Case 2 Critical for Bracing			
		Axial Force		Shear				Axial Force		Shear	
C1	10016	FY	**509.83**	FX	**0.00**	C1	10020	FY	**471.03**	FX	**12.95**
C2	10012	FY	**360.51**	FX	**44.23**	C2	10016	FY	**471.03**	FX	**12.95**
C3	10008	FY	**0.00**	FX	**88.45**	C3	10012	FY	**195.11**	FX	**75.50**
C4	10004	FY	**-360.51**	FX	**44.23**	C4	10008	FY	**-195.11**	FX	**75.50**
C5	10032	FY	**-509.83**	FX	**0.00**	C5	10004	FY	**-471.03**	FX	**12.95**
C6	10028	FY	**-360.51**	FX	**44.23**	C6	10032	FY	**-471.03**	FX	**12.95**
C7	10024	FY	**0.00**	FX	**88.45**	C7	10028	FY	**-195.11**	FX	**75.50**
C8	10020	FY	**360.51**	FX	**44.23**	C8	10024	FY	**195.11**	FX	**75.50**
Total			**0.00**		**353.80**				**0.00**		**353.80**

	S =	**162.8** kN		
Tank Empty condition	**M =**	**5680** kNm	hcg=	35.071 -31.6
	r =	**6.05** m	hcg=	**3.471** m

Peripheral and Diagonal Braced Tank Empty Condition											
Column No.	Node No.	Case1 Critical for Column					Node No.	Case 2 Critical for Bracing			
		Axial Force		Shear				Axial Force		Shear	
C1	10016	FY	**234.71**	FX	**0.00**	C1	10020	FY	**216.84**	FX	**5.96**
C2	10012	FY	**165.97**	FX	**20.35**	C2	10016	FY	**216.84**	FX	**5.96**
C3	10008	FY	**0.00**	FX	**40.70**	C3	10012	FY	**89.82**	FX	**34.74**
C4	10004	FY	**-165.97**	FX	**20.35**	C4	10008	FY	**-89.82**	FX	**34.74**
C5	10032	FY	**-234.71**	FX	**0.00**	C5	10004	FY	**-216.84**	FX	**5.96**
C6	10028	FY	**-165.97**	FX	**20.35**	C6	10032	FY	**-216.84**	FX	**5.96**
C7	10024	FY	**0.00**	FX	**40.70**	C7	10028	FY	**-89.82**	FX	**34.74**
C8	10020	FY	**165.97**	FX	**20.35**	C8	10024	FY	**89.82**	FX	**34.74**
Total			**0.00**		**162.80**				**0.00**		**162.80**

Lateral Load Distribution



Axial force on each Column

1	**Tank full Impulsive condition**			
	Impulsive load	=	**-2048.08**	kN
	Live Load	=	**-43.65**	kN
2	**Tank full Convective condition**			
	Convective load =	=	**-1531.53**	kN
	Live Load	=	**-43.65**	kN
3	**Tank Empty condition**			
	Axial load =	=	**-1346.68**	kN
	Live Load	=	**-43.65**	kN