Seismic Analysis and Design of Liquid Storage Tanks

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Major Project

Submitted in partial fulfillment of the requirements

for the degree of

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(Computer Aided Structural Analysis and Design)

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Declaration

This is to certify that

a The major project comprises my original work towards the Degree of Master of Technology in Civil Engineering (Computer Aided Structural Analysis and Design) at Nirma University and has not been submitted elsewhere for a degree.

b Due acknowledgement has been made in text to all other material used.

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Romil A Hadwani

Certificate

This is to certify that the Major Project entitled **"Seismic Analysis and Design of Liquid Storage Tank"** submitted by **Romil A. Hadwani (18MCLC03)**, 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, Ahmedabad, is the record of work carried out by him under my supervision and guidance. The work submitted has in my opinion reached a level required for being accepted for examination. The results embodied in this Major Project, to the best of my knowledge, have not been submitted to any other university or institution for award of any degree or diploma.

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Abstract

Liquid Storage tank is one of the most critical components of petrochemical industries, nuclear plants and other critical plants. Analysis and design of such structures considering various critical load is very important for its safety. Due to importance of liquid storage tank being remain operational, such structure are designed following standardize codal provision guideline in most of the countries. However, analysis and design of liquid storage tank to seismic loading's is challenging and requires careful consideration, as damage to such structure has a greater impact on plants, environment and human life. There are various seismic design guidelines available for the liquid storage tank. Use of such guidelines warrant in depth understanding of dynamic behaviour of the liquid storage tank. Therefore, it is important to study the seismic behaviour of the liquid storage tank and to assess seismic hazard associated with it.

Major objective of the present study is to analyze and design a storage tank storing – product following various national and international codes. A tank of 22 MLD capacity located in seismic zone–v as per Indian seismic code IS:1893,is considered. Liquid stored inside the tank is modelled as a spring-mass analogy comprising of impulsive and convective modes of oscillation defined by IS:1893(Part-II)-2014. API 650 stipulations are mostly used world wide for gravity and seismic loading of the liquid storage tank. In the present study tank is designed using one-foot method following API 650 guidelines. However, seismic analysis is carried out using both IS:1893(Part-II)-2014 and API 650 guidelines with an objective to check efficiency of Indian seismic code. Seismic response parameters such as time period, base shear and bending moment are evaluated for liquid storage tank. It has been found that Indian seismic code guidelines are at par with API 650 guideline since seismic response parameters differs only by 7-8%. Tank is found to be self-anchored under API 650 guideline and found to be stable under seismic and wind loads. Erection plan and detailed structural drawings related to fabrication and erection are prepared for the liquid storage tank. A computational model of the liquid storage tank is prepared in a licensed commercial software SAP2000. Finite element analysis is carried out under the static conditions for the tank. A good agreement in terms of stress in the shell and thickness of the shell is found when the results of FEA and manual calculations are compared. Parametric studies in terms of type of liquid stored, geometric aspect ratio and seismic zone may be carried out that will be useful to day-to-day design of the tank.

Abbreviation and Notation

API .... American Petroleum Institute
H .... Height of the Tank
D .... Diameter of the Tank
E .... Modulus of Elasticity of tank wall
γ_w .... Mass Density of Water
J_r .... Joint Efficiency
S_d .... Maximum Allowable Design Stress
S_t .... Maximum Allowable Hydrostatic Stress
CA .... Corrosion Allowance
G .... Specific Gravity of Liquid to be Stored
m_i .... Impulsive Mass
m_c .... Convective Mass
m_b .... Mass of Base Plate
m_t .... Mass of Roof Slab
m_w .... Mass of Tank wall
I .... Importance Factor
R .... Response Reduction Factor
t_d .... Design Shell Plate Thickness
t_t .... Hydrostatic Test Plate Thickness
t_b .... Thickness of Base Slab
T_i .... Time period of Impulsive Mass
T_c .... Time period of Convective Mass
C_i .... Coefficient for Time Period of Impulsive Mode
C_c .... Coefficient for Time Period of Convective Mode
A_h .... Horizontal Seismic Coefficient
g .... Acceleration due to Gravity

Contents

List of Figures

List of Tables

Chapter 1

Introduction

1.1 General

Liquid storage tanks are always been an important structures for the storage and distribution of water, chemicals and refined petroleum products for the oil refineries and petrochemical industries. Industrial liquid containing tanks are used to store different types of materials like water, crude oil, kerosene, ethanol, Naptha, Petcoke, Special Boiling Point Spirit etc..

Figure 1.1: Oil Storage Tanks [20]

Different types of materials that are used for the construction of liquid storage tanks are concrete, steel, stone, plastic tank, polyethylene tank, carbon steel, fibreglass etc. Liquid storage tanks are mainly of two types:

1. Ground Supported Tanks.

2. Elevated Tanks.

Ground supported tanks are used by various industries for storing toxic materials, petrochemicals and water. Elevated tanks are used to store the water for public distribution. Liquid storage tanks have different shape such as Circular, Rectangular, Cylindrical etc. These tanks must remain functional in post earthquake period and toxic contents in them should not leak. Spillage of liquid out of the tanks leads to explosion and damage to the nearby. Hence seismic safety of liquid storage tanks is of considerable importance.
Liquid Storage tank consist of following section as shown in Figure [14]

1. Annular Plate.

2. Bottom Plate.

3. Shell Plate of the tank.

4. Roof of the Tank.

5. Foundation.

6. Accessories like Shell Manhole, Stairs, Railing, Piping, Top Stiffeners etc...

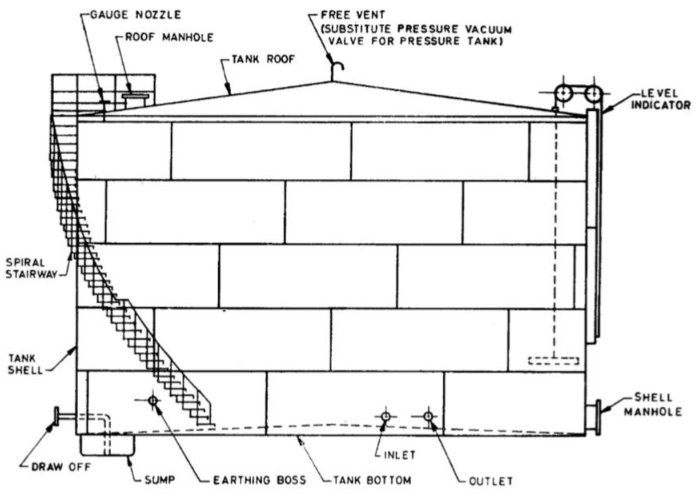

Figure 1.2: General Arrangement of Liquid Storage Tanks [2]

1.2 Types of Tanks Based on Roof

Different types of liquid storage tank used in petroleum industry and oil refineries are also classified based on the type of roof as described in the Figure 1.3

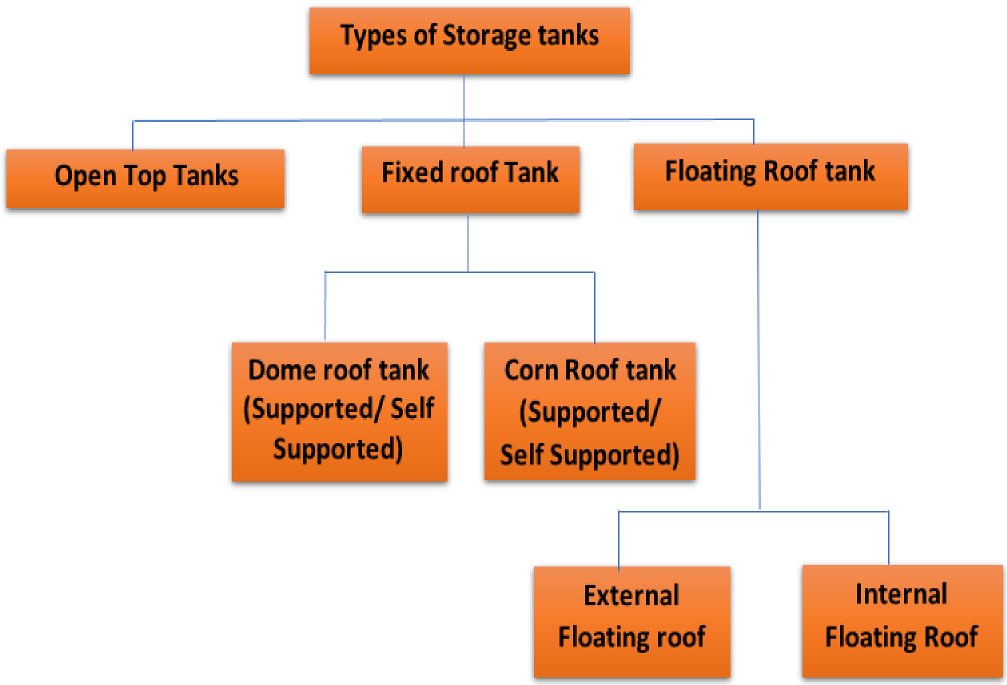

Figure 1.3: Types of Liquid Storage Tanks

Open Tank

Open tanks are the tanks which are directly opened to atmosphere and they does not have any type of roof.These tanks are also known as atmospheric tanks.Fire water/cooling water are stored in such types of tanks. These tanks shall not be used for storing the petroleum and other flammable products.

Fixed Roof Tank

These type of tanks has fixed roof. Generally they are further classified into Dome roof tank and Cone roof tank.These types of tank can be supported on rafters or trusses or self supported depending on the size of the tanks.Fixed roof can also be designed as atmospheric tank by providing free vent.

Floating Roof Tank

Floating roof tanks are the one where roof tanks with a floating roof which travels up and down along with the liquid level. They are further divided in to internal floating roof and external floating roof. External roof is the roof that floats on the liquid in tank open to the atmosphere and in the internal floating roof it floats on the liquid inside the fixed roof tank.

1.3 Design Codes and Standards

- API 650 (American Petroleum Institute 650) for Welded tanks for Oil Storage.
- Indian Standard 803:1976 Code of Practice of Design, Fabrication and Erection of Vertical Mild Steel Cylindrical Welded Oil Storage Tanks.
- API 620 Design and Construction of Large, Welded, Low-pressure Storage Tanks.
- API 653 Tank Inspection, Repair, Alteration, and Reconstruction.
- British Standards 2654 Manufacture of Vertical Storage Tank with Butt Welded Shell for Petroleum Industry
- Indian Standard 1893 (Part-II)-2014 Criteria for Earthquake Resistant Design of Liquid Retaining Tanks.

1.4 Need of the Study

Liquid storage tanks that where inadequately designed have suffered extensive damage during the past earthquake such as Alaska earthquake. Liquid storage tanks consists of highly toxic and hazardous material. During earthquake these tanks are prone to damage or in some of the cases leads to failure.To avoid the adverse effect such as fires, spillage of oil, overturning effect , environmental pollution and damage to human life better understanding of the behaviour of the tank under seismic excitation is necessary.The structural safety of liquid storage tanks even under seismic hazards is of an utmost importance and that is why seismic design and analysis is necessity in construction of any liquid storage tank.

1.5 Objectives of the Study

Major objectives of the present study are:

- To study the various national and international seismic codal stipulations and outline critical issues related to seismic analysis and design of liquid storage tank.
- To analyze and design liquid storage tank for various storage materials using seismic codal provisions and produce standardize design protocol.
- To study modelling approach for liquid storage tank and carry out seismic time history analysis.

1.6 Scope of the Work

In order to achieve above mentioned objectives, following scope of the work is proposed:

- Review, critically, various national and international seismic code for seismic analysis and design .
- Study various materials used in liquid storage tank and their characteristics.
- Understand fundamental behaviour of liquid storage tank through laboratory based experiments.
- Undertake exhaustive literature review on modelling, seismic analysis and failures of liquid storage tank.
- Conduct seismic analysis and design of liquid storage tank using API 650 and IS:1893(Part-II)-2014.
- Select appropriate mathematical model for liquid storage tank for seismic analysis.
- Carryout the analysis and design of liquid storage tank subjected to seismic excitation.
- Extract useful results and carryout discussion to derive important outcome of the study and to prepare technical report on the study.

1.7 Layout of the Report

This study is related to the Seismic Analysis and Design of Liquid Storage Tanks. The layout of the report is divided in to the following chapters:

- **Chapter 1** incorporates discussion about introduction and types of liquid storage tanks.Further it includes need of the study, objective of the study and scope of the work.

- **Chapter 2** deals with the literature review which contains the study of research papers and various design guidelines.It includes the study of behaviour of liquid tank under seismic loading.

- **Chapter 3** covers general introduction of design philosophy of API650 guidelines and problem formulation.It also includes gravity load analysis and design of different component of liquid storage tank.

- **Chapter 4** captures the seismic analysis of liquid storage tank as per API 650 standard and Indian Standard IS:1893(Part II)-2014.It also covers the erection plan of the liquid storage tank.

- **Chapter 5** deals with the finite element analysis of the liquid storage tank under static conditions.

- **Chapter 6** consists of summary, conclusion and recommendation of the future scope of the work that has been conducted during major project.

Chapter 2

Literature Review

2.1 General

Literature review of the Seismic Analysis and Design of Liquid Storage Tank is presented in this chapter.Various codal stipulations and research papers have been refereed to understand the basic behavior, advantages, modeling, analysis and design of oil storage tank under seismic event. Dynamic characteristics and failure modes are also reviewed.

2.2 Mathematical Modeling of Liquid Storage Tank

Housner[9] proposed a tool for modeling large liquid storage tanks experiencing seismic load. Availability of these analogues model has simplified the analysis. His spring-mass analogy system separates tank-liquid system into two parts:the Impulsive mass and the convective mass. As shown in the Figure 2.1 Impulsive mass is the lower portion of the liquid which is observed to move coincidentally with the tank while Convective mass is the upper portion of the liquid which generates waves when subjected to horizontal loading. Impulsive mass is modelled as a rigid link connected to the tank while convective mass is modelled as a mass connected to the tank by a spring with a given stiffness oscillating primarily in fundamental mode.

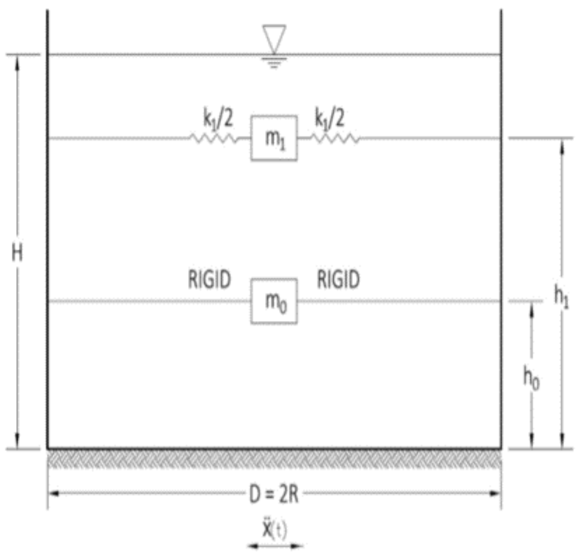

Figure 2.1: Spring-Mass Model of Ground Storage Tank Subjected to Horizontal Acceleration

Major drawback of the housner model is that he has assumed tank wall and foundation as a rigid but which is not applicable to the real cases. Housner model is used to design the storage by accounting for different types of failures that can occur during a seismic event.

2.3 Design Guidelines of Liquid Storage Tanks

Jaiswal et al [7] In this guideline, provisions of IBC 2000, AWWA, API 650, ACI, Eurocode 8 and NZSEE guidelines are reviewed, to evaluate the severity of design seismic forces for tanks.This estimation is done with respect to corresponding design seismic force for buildings.Such a comparative estimation helps in knowing how severe design seismic action for tank is, as compared to that for a building under similar seismic conditions.Liquid storage tanks are designed for higher seismic forces as compared to conventional buildings due to their low ductility and low energy absorbing capacity.Various conclusions drawn from the comparative assessment of provisions of different codes on seismic design of liquid storage tanks are that there is no similarity in types of tanks described in various guidelines.All this design guidelines have considered consider impulsive and convective modes of vibration in the seismic analysis of ground-supported tanks.

It is observed that for a particular type of tank with a short time periods ($T<0.6$ sec), ratio of base shear of tank and base shear of the building is almost similar in all the guidelines. This ratio is 6 to 7 for low ductility tanks and 3 to 4 for high ductility tanks.For a time period grater than 0.6sec large variations in this ratios are observed.API 650 and AWWA D-100 specify a constant value of base shear coefficient,for ground supported tanks which does not depend on time period.For steel tanks, it is quite likely that impulsive time period will be in the constant-acceleration range of spectra, and hence it suffices to specify this constant value of base shear coefficient.

Jaiswal et al.[8] This paper reviews provisions related with the analysis and modeling aspects of liquid storage tanks from various international codes.Estimation of hydrodynamic forces requires suitable modeling and dynamic analysis of tank liquid system, which is very complex. However, the availability of mechanical models of the liquid tanks has considerably simplified the analysis.These mechanical models, convert the tank-liquid system into an equivalent spring mass system. Design codes use these mechanical models to evaluate seismic response of tanks.

Hosseinzadeh et al.[13] determined the dynamic characteristics and failure mode using API650-2008 guidelines and compare it with FEM model analyses.In the first part the dynamic characteristics such as diamond shape buckling, slide bottom, elephant foot buckling, Sloshing, Uplift and structural period of vibrations are studied with their formulations from the API650-2008 (APPENDIX E) seismic provisions.In this study, 161 existing tanks in an oil refinery have been classified in to 24 groups and are investigated using API650-2008 guidelines and numerical finite element models using ANSYS software.Study concluded that impulsive and convective time period from API650 and FEM analyses are almost similar.Comparison of elephant foot buckling stresses obtained using both are similar for small diameter tanks.Code predicts higher stresses for larger diameter tanks as compared to FEM analyses.Although the API 650 guidelines agreed with the numerical analyses in some cases,this investigation highlights the major shortcomings of the code requirements in the design of liquid storage tanks.

Hamdan F.H.[12] reviewed the behaviour and design guidelines of cylindrical steel liquid storage tanks subjected to earthquake motion. Finite element analysis and published experimental results were used in this study coupled with field observations during past earthquakes to evaluate the accuracy of EUROCODE 8(EC8) Design guidelines.Special emphasis was provided on performance comparison of steel versus concrete tanks using field observations during past earthquakes.Various failure mechanisms occurring from interaction between fluid and structures such as, Elephant foot buckling, Diamond shape buckling, Uplifting of tanks and damage and collapse of roofs were summarized. A number of design guidelines for anchored and unanchored tanks such as, API 650, AWWA 1984,ASCE recommendations(1984), Austrian Design recommendations, Eurocode 8 (EC8), Part 4, New Zealand guidelines and Japanese guidelines were also discussed. Their assessment was done on the basis of variety of parameters like Sloshing wave height, Hydrodynamic pressures, Base shear and overturning moments and Elastic and elasto-plastic buckling strength of anchored tanks. And Finally, Behavior of steel tanks was compared to code predictions and FEM analysis in terms of number of governing factors like, Sloshing, Hydrodynamic pressures, overturning moment, base shear, Effect of tank-base support on axial compressive stresses, Effect of tank-base support on hoop stresses, Effect of roof strength and stiffeners on the buckling load, Presence of two earthquake components and Effects of tank inertia. The Areas where the guidelines need development were also briefly discussed. Results suggested that, in a few cases,there were some irregularities in code requirements that required further examination.

2.4 Dynamic Characteristics of Liquid Storage Tanks

Different types of characteristics of liquid storage tank when subjected to earthquake loading are: Elephant foot buckling, Diamond shape buckling, Structures period of vibrations, Bottom slide, Sloshing effect, Uplift of the tank [10].
Elephant-foot buckling
Buckling stress are caused by the internal fluid pressure and compression force acting on shell wall. Elephant foot buckling is an outward bulge just above the steel tank base as shown in Figure 2.2. It occurs in the tank with low height to radius ratio. Elephant foot buckling is also highly influenced by the uplift of the tank shell. Elephant foot buckling

occurs due to high tensile hoop stresses combined with the vertical compressive forces [13].

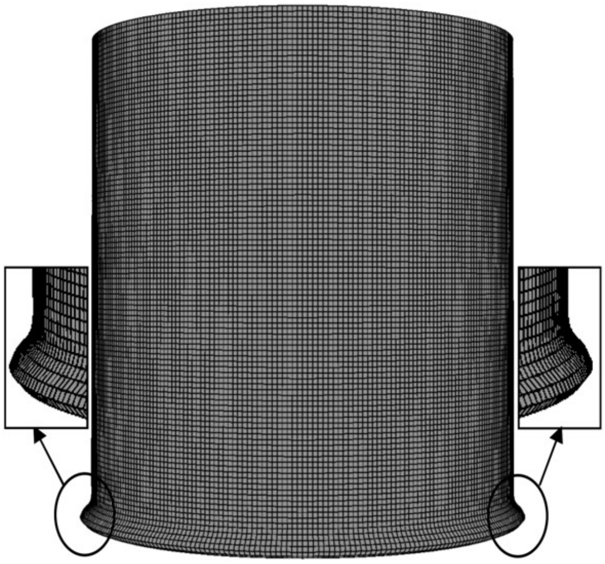

Figure 2.2: Elephant Foot Buckling [10]

Diamond Shape Buckling

Diamond shape buckling occurs at lower internal stress and higher the compressive stress as compared to elephant foot buckling.It tends to occur near the middle or top of the tank shell as shown in Figure 2.3. It also occurs at the location where thickness of the tank gets varied [13].

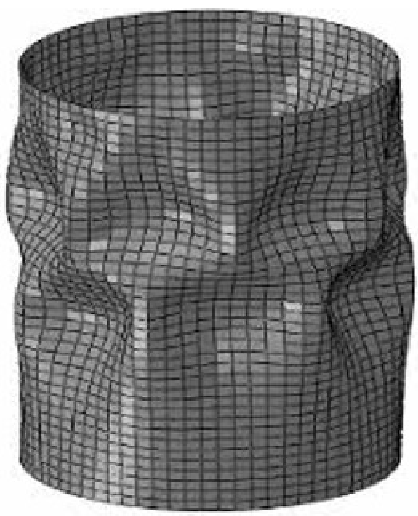

Figure 2.3: Diamond Shape Buckling[10]

Uplift of the tank

Tanks may undergo uplift when the magnitude of overturning exceeds design value.Uplift does not result in the collapse of the tank but includes the serious damage to the piping at the connection of the tank. Uplift can be limited by anchoring the tank to the foundation.Figure 2.4 shows the uplift of the tank occurring on the opposite face of elephant foot buckle which denotes an equal and opposite reaction to buckling mechanism [13].

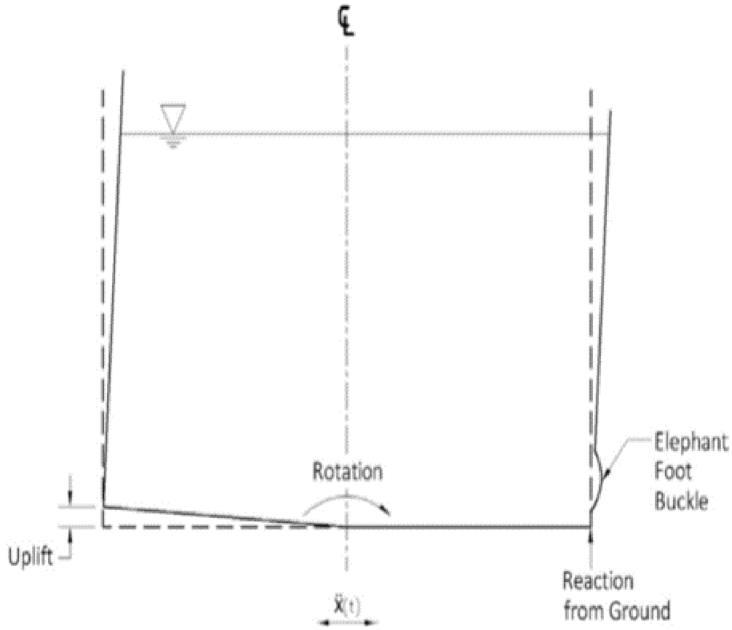

Figure 2.4: Tank Behavior During Horizontal Acceleration

2.5 Summary

In this chapter, literature review related to the analysis and design of storage tank has been studied.This review includes the study of design of real life tank structures using codal provisions and FEM analyses and also the dynamic response of the tanks under earthquake.

Chapter 3

Gravity Load Design of Liquid Storage Tank

3.1 General

Design of Liquid Storage Tank is carried out using API STANDARD 650 (American Petroleum Institute Standard 650). This standard apply only for the vertical, cylindrical, above-ground, closed and open welded oil storage tanks [1]. Liquid Storage Tank mainly consists of the following sections - Design of Shell, Tank Roof, Bottom Plate and Annular Plate Design.The design of shell consists of calculation of shell wall plate thickness, top and intermediate stiffener ring, stability against wind load and seismic load and design of anchorage.Bottom Plate Design consist of sizing up of the thickness of plates and welding requirements. Design of roof consists of roof stress design, accessories and fitting design. A cylindrical welded liquid storage tank having 22 million liters capacity is design is carried out using API 650 provisions.

3.2 Problem Formulation

Design of 22 Million liters capacity of Liquid Storage Tank used to store petrol is considered for the present study. Figure 3.1 shows the layout of the large diameter liquid storage tank. Following parameters are considered for the analysis and design of the tank; Diameter of the tank is 40 m, Height of the tank is 20 m, Material stored in the tank is Petrol, Specific Gravity of Petrol as 0.75, Density of liquid to be stored as 753 kg/m^3,

Joint efficiency factor as 0.85 due to spot radiography examination of joint, Allowable corrosion as 1.5 mm.
Maximum allowable design stress 164 N/mm^2, Maximum allowable hydrostatic stress as 176 N/mm^2.

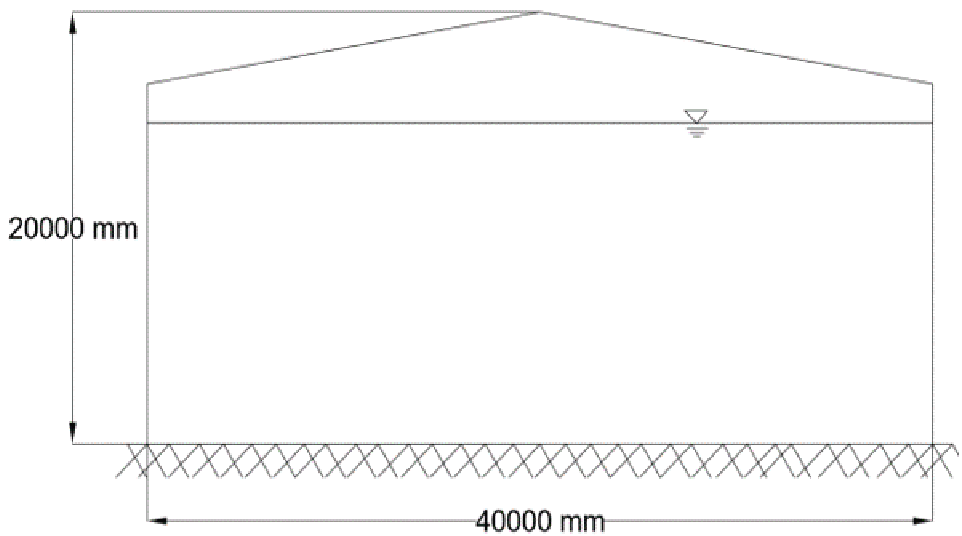

Figure 3.1: Line Diagram of the Tank

3.3 Material Selection

Material used for the construction of liquid storage tank should follow the specifications listed in the Section 4 **Materials** of API 650 Standard [1]. API 650 **clause 4.2** allows materials to be chosen from wide range of standard material specifications such as ASTM, CSA, ISO, EN and National standards. For this particular type of the tank, ISO Specification Grade in qualities E275C and E275D for maximum plate thickness of 40 mm is selected.These materials are chosen based on the types of liquid stored inside the tank and atmospheric conditions.Table 3.1 shows specification of the selected plate material and allowable stresses.

3.3.1 Allowable Stresses

The design stress of the material chosen S_d shall be either two-thirds the yield strength or two-fifth the tensile strength of the material, whichever is less. The corroded plate thickness shall be used in the calculation of design stress of material.While the hydrostatic stress S_t calculated should be either three-fourth the yield strength or three-sevenths the tensile strength, whichever is less.The nominal plate thickness shall be used in the calculation of this stress [1].

Table 3.1: Material Selection as per API 650

Material	Nominal Plate Thickness (mm)	Minimum Yield Strength (MPa)	Minimum Tensile Strength (MPa)	Design Stress (MPa)	Hydrostatic Stress (MPa)
ISO 630	16 <t <40	265	410	164	176

3.4 Design of Shell Course

API 650 guideline employs that the provided shell plate thickness should be greater of the design shell plate thickness considering corrosion allowances or hydrostatic test shell plate thickness. **Clause 5.6.1.1** of API 650 [1] employs minimum shell plate thickness based on the nominal tank diameter [1]. Shell plate thickness shall not be less than the value given in the Table 3.2. For this particular tank having diameter of 40 m the minimum shell thickness should be 8 mm. As per this guideline shell plate shall have minimum nominal width of 1800 mm unless otherwise specified by the purchaser [1]. The thickness of lower shell course should not be less than thickness of the upper shell course for design stress and hydrostatic stress conditions. The thickness of each shell course can be calculated by three different methods they are 1-Foot Method, Variable Design Point Method and Calculation of thickness by Elastic Analysis.

Table 3.2: Minimum Thickness of Shell Plates as per API 650

Nominal Tank Diameter (m)	Nominal Plate Thickness (mm)
<15	5
15 to <36	6
36 to 60	8
>60	10

3.4.1 Thickness by 1-Foot Method

As per **Clause 5.6.3** of API 650 guidelines the 1-foot method is used to calculate the requires thickness of shell at design points 0.3 m (1ft) above the bottom of each shall course [1].One-foot method shall not be used for liquid storage tank having diameter larger than 61 m (200ft).The required shell thickness shall be grater than the thickness calculated by following formula.
For Design Shell Thickness,

$$t_d = \frac{4.9D(H-0.3)G}{S_d} + CA \qquad \text{(Eq. 3.4.1)}$$

For Hydrostatic Test Shell Thickness,

$$t_t = \frac{4.9D(H-0.3)}{S_t} \qquad \text{(Eq. 3.4.2)}$$

Where,

t_d = Design shell plate thickness,in mm;

t_t = Hydrostatic test shell thickness,in mm;

D= Tank diameter,in m;

H= Design height of the liquid stored,in m

G= Specific gravity of liquid stored inside the tank;

CA= Corrosion allowance , in mm;

J_e= Joint efficiency factor;

S_d= Allowable design stress,in MPa

S_t= Allowable the hydrostatic test condition,in MPa

Shell Design Calculations

Material= ISO630-E275C

Diameter of the tank (D) = 40 m

Width of shell plate = 2 m

Design Condition G = 0.75

J_e= 0.85

S_d= 164 N/mm^2.

S_t= 176 N/mm^2

Shell Course - 10

Corrosion allowance CA_10 = 1.5 mm

Height (H)= 20 m

Design Shell Thickness

$$t_{d_{10}} = \frac{4.9D(H-0.3)G}{S_d \times J_e} + CA_{10}$$

$$t_{d_{10}} = \frac{4.9 \times 40 \times (20-0.3) \times 0.75}{164 \times 0.85} + 1.5$$

$$t_{d_{10}} = 22.30 \text{ mm}$$

Hydrostatic Shell Thickness

$$t_{t_{10}} = \frac{4.9D(H-0.3)G}{S_t \times J_e}$$

$$t_{t_{10}} = \frac{4.9 \times 40 \times (20-0.3) \times 1}{176 \times 0.85}$$

$$t_{t_{10}} = 25.80 \text{ mm}$$

The provided thickness of the Shell Course-10 is 28 mm.

Shell Course - 9

Corrosion allowance CA_2 = 1.5 mm

Height (H)= 18 m

Design Shell Thickness

$$t_{d_9} = \frac{4.9D(H-0.3)G}{S_d \times J_e} + CA_9$$

$$t_{d_9} = \frac{4.9 \times 40 \times (18-0.3) \times 0.75}{164 \times 0.85} + 1.5$$

$$t_{d_9} = 20.20 \text{ mm}$$

Hydrostatic Shell Thickness

$$t_{t_9} = \frac{4.9D(H-0.3)G}{S_t \times J_e}$$

$$t_{t_9} = \frac{4.9 \times 40 \times (18-0.3) \times 1}{176 \times 0.85}$$

$$t_{t_9} = 23.15 \text{ mm}$$

The provided thickness of the Shell Course-9 is 25 mm.

Shell Course - 5

Corrosion allowance $CA_5 = 1.5$ mm

Height (H)= 10 m

Design Shell Thickness

$$t_{d_5} = \frac{4.9D(H-0.3)G}{S_d \times J_e} + CA_5$$

$$t_{d_5} = \frac{4.9 \times 40 \times (10-0.3) \times 0.75}{164 \times 0.85} + 1.5$$

$$t_{d_5} = 11.72 \text{ mm}$$

Hydrostatic Shell Thickness

$$t_{t_5} = \frac{4.9D(H-0.3)G}{S_t \times J_e}$$

$$t_{t_5} = \frac{4.9 \times 40 \times (10-0.3) \times 1}{176 \times 0.85}$$

$$t_{t_5} = 12.69 \text{ mm}$$

The provided thickness of the Shell Course-5 is 15 mm.

Shell Course - 2

Corrosion allowance $CA_2 = 1.5$ mm

Height (H)= 4 m

Design Shell Thickness

$$t_{d_2} = \frac{4.9D(H-0.3)G}{S_d \times J_e} + CA_2$$

$$t_{d_2} = \frac{4.9 \times 40 \times (4-0.3) \times 0.75}{164 \times 0.85} + 1.5$$

$$t_{d_2} = 5.40 \text{ mm}$$

Hydrostatic Shell Thickness

$$t_{t_2} = \frac{4.9D(H-0.3)G}{S_t \times J_e}$$

$$t_{t_2} = \frac{4.9 \times 40 \times (4-0.3) \times 1}{176 \times 0.85}$$

$$t_{t_2} = 4.84 \text{ mm}$$

he provided thickness of the Shell Course-2 is 8 mm as minimum thickness as per codal provision needs to be provided.

Shell Course - 1

Corrosion allowance $CA_9 = 1.5$ mm

Height (H)= 2 m

Design Shell Thickness

$$t_{d_1} = \frac{4.9D(H-0.3)G}{S_d \times J_e} + CA_1$$

$$t_{d_1} = \frac{4.9 \times 40 \times (2-0.3) \times 0.75}{164 \times 0.85} + 1.5$$

$$t_{d_1} = 3.30 \text{ mm}$$

Hydrostatic Shell Thickness

$$t_{t_1} = \frac{4.9D(H-0.3)G}{S_t \times J_e}$$

$$t_{t_1} = \frac{4.9 \times 40 \times (2-0.3) \times 1}{176 \times 0.85}$$

$$t_{t_1} = 2.22 \text{ mm}$$

The provided thickness of the Shell Course-1 is 8 mm as minimum thickness as per codal provision needs to be provided.

The detail calculations for each shell course is attached in Appendix A. The calculation of thickness of each shell course is summarised in Table 3.4

Table 3.3: Design Summery of Shell Course Thickness as per 1-Foot Method

Shell Course (Bottom to Top)	**Height of Each Shell Course (mm)**	**Height (mm)**	t_{min} **(mm)**	**Design Shell Thickness (t_d) (mm)**	**Hydrostatic Shell Thickness (t_t)(mm)**	t_{max} **(mm)**	$t_{provided}$ **(mm)**
Shell Course 10	2000	20000	8	22.3	25.77	25.8	28
Shell Course 9	2000	18000	8	20.2	23.15	23.2	25
Shell Course 8	2000	16000	8	18.1	20.53	20.5	22
Shell Course 7	2000	14000	8	15.9	17.92	17.9	20
Shell Course 6	2000	12000	8	13.8	15.30	15.3	18
Shell Course 5	2000	10000	8	11.7	12.69	12.7	15
Shell Course 4	2000	8000	8	9.6	10.07	10.1	12
Shell Course 3	2000	6000	8	7.5	7.46	8.0	10
Shell Course 2	2000	4000	8	5.4	4.84	8.0	8
Shell Course 1	2000	2000	8	3.3	2.22	8.0	8

3.4.2 Shell Thickness by the Variable-Design-Point Method

Variable design point method normally gives a reduce shell-course thicknesses and total material weight as compared to one-foot method. Use of this method gives shell plate thicknesses at design points where the calculated stresses is being relatively close to the actual circumferential shell stresses. VDPM method may only be used when the following criteria is true or the use of 1-Foot Method has not been specified by purchaser:

$$\frac{L}{H} \leq \frac{1000}{6}$$

where,
L = $(500Dt)^{0.5}$, in mm;
D = Diameter of the tank in m;
t = Corroded thickness of the bottom course in mm;
H = Deign height of liquid,in m;

Shell courses thickness for both the design condition and the hydrostatic test condition are evaluated independently. For the calculation of bottom course thickness, firstly the values t_{pd} and t_{pt} for the design and hydrostatic test conditions shall first be calculated as per Eq. 3.4.1 and Eq. 3.4.2 respectively.
The bottom shell course thickness can be calculated using the following formula:
For design shell thickness,

$$t_{1d} = (1.06 - \frac{0.0696D}{H}\sqrt{\frac{HG}{S_d}})(\frac{4.9HDG}{S_d}) + CA \qquad \text{(Eq. 3.4.3)}$$

For hydrostatic shell thickness,

$$t_{1d} = (1.06 - \frac{0.0696D}{H}\sqrt{\frac{H}{S_t}})(\frac{4.9HD}{S_t}) \qquad \text{(Eq. 3.4.4)}$$

For calculating the next upper shell course thickness for both the design and the hydrostatic test condition, the value of the following ratio shall be calculated for the bottom course:

$$\frac{h_1}{(rt_1)^{0.5}}$$

where,

h_1 is bottom shell course height, in mm; r is the radius of the tank, in mm; t_1 corroded thickness of bottom shell course, in mm, used to calculate t_2(design).The calculated hydrostatic thickness of bottom shell course shall be used to calculate t_2 (hydrostatic test).

If ratio is less than or equal to 1.375

$t_2 = t_1$

If the ratio is greater than 2.625

$t_2 = t_{2a}$

If the ratio is greater than 1.375 and less than 2.625

$$t_2 = t_{2a} + (t_1 - t_{2a})[2.1 - \frac{h_1}{1.25(rt_1)^{0.5}}]$$

t_2= minimum thickness of the second shell course for design condition , in mm,

t_{2a} = corroded thickness of second shell course, in mm;

Further to find the upper shell course thickness for both the design condition and the hydrostatic test condition then a preliminary value t_u for the upper-course thickness shall be calculated using the Eq. 3.4.1 without any corrosion allowance. Now the distance x of the variable design point from the bottom of the course shall be calculated using the following formula:

$$x_1 = 0.61(rt_u)^{0.5} + 320CH$$

$$x_2 = 1000 \times C \times H$$

$$x_3 = 1.22 \times (rt_u)^{0.5}$$

Where,

t_u is upper shell course corroded thickness at the girth joint, in mm;

t_L is lower course corroded thickness at the girth joint, in mm;

H is the design liquid height, in m;

K equals t_L/t_u;

$$C = \frac{K^{0.5}(K-1)}{1+K^{1.5}}$$

Value of x is lowest among all three values obtained is used for the calculation of the minimum thickness t_x for the upper shell courses for both the design (t_{dx}) and the hydrostatic test condition.

$$t_{dx} = \frac{4.9D(H - \dfrac{x}{1000})G}{S_d} + CA \qquad \text{(Eq. 3.4.5)}$$

$$t_{tx} = \frac{4.9D(H - \dfrac{x}{1000})}{S_t} \qquad \text{(Eq. 3.4.6)}$$

Design Calculation

Bottom Course

Check Ratio

$$\frac{L}{H} \leq \frac{1000}{6}$$

$$L = (500 \times 40 \times 28)^{0.5} = 748.33 \text{ mm}$$

$$\frac{748.33}{20} \leq \frac{1000}{6}$$

$$37.42 \leq 166.67$$

Hence we can use Variable Design Point Method for the calculation of shell plate thickness. Preliminary value t_{pd} and t_{pt} obtain from Eq. 3.4.1 and Eq. 3.4.2 is 22.30 mm and 25.80 mm respectively.

For design shell thickness,

$$t_{1d} = (1.06 - \frac{0.0696 \times 40}{20}\sqrt{\frac{20 \times 0.75}{164}})(\frac{4.9 \times 20 \times 40 \times 0.75}{164}) + 1.5 = 20 \text{ mm}$$

t_{1d}= minimum of t_{1d} and t_{pd} = 20 mm

For hydrostatic shell thickness,

$$t_{1t} = (1.06 - \frac{0.0696 \times 40}{20}\sqrt{\frac{20}{176}})(\frac{4.9 \times 20 \times 40}{176}) = 22.68 \text{ mm}$$

t_{1t}= minimum of t_{1t} and t_{pt} = 22.68 mm

t_{min} = minimum nominal thickness required, the greater of t_{1d} and t_{1t}

t_{min}= 22.68 mm

t_{use} = 25 mm

Shell Course - 9

(a) Design Condition

h_1 = 2000 mm

r = 20 m

$$\frac{h_1}{(rt_1)^{0.5}} = \frac{2000}{(20000 \times 25)^{0.5}} = 2.82$$

Here the ratio is greater than 2.625 hence,

$t_9 = t_{9a}$

To find t_{9a}

$$t_u = \frac{4.9 \times 40(18 - 0.3) \times 0.75}{164} = 15.86 \text{ mm}$$

$$\text{K} = \frac{t_L}{t_u} = \frac{23.5}{15.86} = 1.47$$

$C = 0.20$

$x_1 = 0.61(rt_u)^{0.5} + 320CH = 1623.55$ mm

$x_2 = 1000 \times C \times H$= 4000 mm

$x_3 = 1.22 \times (rt_u)^{0.5} = 687.55mm$

x =$Min(x_1, x_2, x_3) = 687.55$

$$t_{9a} = \frac{4.9 \times 40(18 - \frac{687.55}{1000})}{164} = 15.56 \text{ mm}$$

$t_{9d} = t_{9a}$ = 15.56 + 1.5 = 18 mm

(b) Hydrostatic Condition

To find t_{9a}

$$t_u = \frac{4.9 \times 40(18 - 0.3)1}{176} = 19.71 \text{ mm}$$

K = $\frac{t_L}{t_u}$= $\frac{22.68}{19.71} = 1.15$

$C = 0.072$

$x_1 = 0.61(rt_u)^{0.5} + 320CH$ = 837.40 mm

$x_2 = 1000 \times C \times H$= 1440 mm

$x_3 = 1.22 \times (rt_u)^{0.5} = 765.98mm$

X =$Min(x_1, x_2, x_3)$ = 765.98 mm

$$t_{9a} = \frac{4.9 \times 40(18 - \frac{765.98}{1000})}{176} = 19.25 \text{ mm}$$

$t_{9t} = t_{9a}$ = 19.25 mm

$t_{9d} > t_{9t}$

Hence Shell Course-9 Thickness = 19.25 mm = 20 mm (Provided)

Shell Course - 8

(a) Design Condition

h_1 = 2000 mm

r = 20 m

$$\frac{h_1}{(rt_1)^{0.5}} = \frac{2000}{(20000 \times 25)^{0.5}} = 2.82$$

Here the ratio is greater than 2.625 hence,

$t_8 = t_{8a}$

To find t_{8a}

$$t_u = \frac{4.9 \times 40(16 - 0.3) \times 0.75}{164} = 14.07 \text{ mm}$$

$$\mathrm{K} = \frac{t_L}{t_u} = \frac{18}{14.07} = 1.27$$

$C = 0.13$

$\mathrm{x}_1 = 0.61(rt_u)^{0.5} + 320CH = 1155.55$ mm

$x_2 = 1000 \times C \times H = 2600$ mm

$x_3 = 1.22 \times (rt_u)^{0.5} = 647.17mm$

x $= Min(x_1, x_2, x_3) = 647.17$

$$t_{8a} = \frac{4.9 \times 40(16 - \frac{647.17}{1000})}{164} = 13.76 \text{ mm}$$

$t_{8d} = t_{8a} = 13.76 + 1.5 = 16$ mm

(b) Hydrostatic Condition

To find t_{8a}

$$t_u = \frac{4.9 \times 40(16 - 0.3)1}{176} = 17.50 \text{ mm}$$

$$\text{K} = \frac{t_L}{t_u} = \frac{20}{17.50} = 1.15$$

$C = 0.072$

$\text{x}_1 = 0.61(rt_u)^{0.5} + 320CH = 821.40$ mm

$x_2 = 1000 \times C \times H = 1440$ mm

$x_3 = 1.22 \times (rt_u)^{0.5} = 721.10mm$

$\text{X} = Min(x_1, x_2, x_3) = 721.10$ mm

$$t_{8a} = \frac{4.9 \times 40(16 - \frac{721.10}{1000})}{176} = 17.20 \text{ mm}$$

$t_{8t} = t_{8a} = 17.20$ mm

$t_{8d} > t_{8t}$

Hence Shell Course-8 Thickness = 17.25 mm = 18 mm (Provided)

The procedure for second shell course is repeated to get the thickness of all other remaining shell courses.

Table 3.4: Design Summery of Shell Course Thickness as per Variable Design Point

Shell Course (Bottom to Top)	**Height of Each Shell Course (mm)**	**Height (mm)**	t_{min} **(mm)**	**Design Shell Thickness (t_d) (mm)**	**Hydrostatic Shell Thickness (t_t)(mm)**	t_{max} **(mm)**	$t_{provided}$ **(mm)**
Shell Course 10	2000	20000	8	20	22.68	22.68	25
Shell Course 9	2000	18000	8	18	19.25	19.25	22
Shell Course 8	2000	16000	8	16.1	18.00	18.00	20
Shell Course 7	2000	14000	8	14.20	16.72	16.72	18
Shell Course 6	2000	12000	8	13.8	15.30	15.3	15
Shell Course 5	2000	10000	8	11.7	12.69	12.7	12
Shell Course 4	2000	8000	8	9.6	10.07	10.1	12
Shell Course 3	2000	6000	8	7.5	7.46	8.0	8
Shell Course 2	2000	4000	8	5.4	4.84	8.0	8
Shell Course 1	2000	2000	8	3.3	2.22	8.0	8

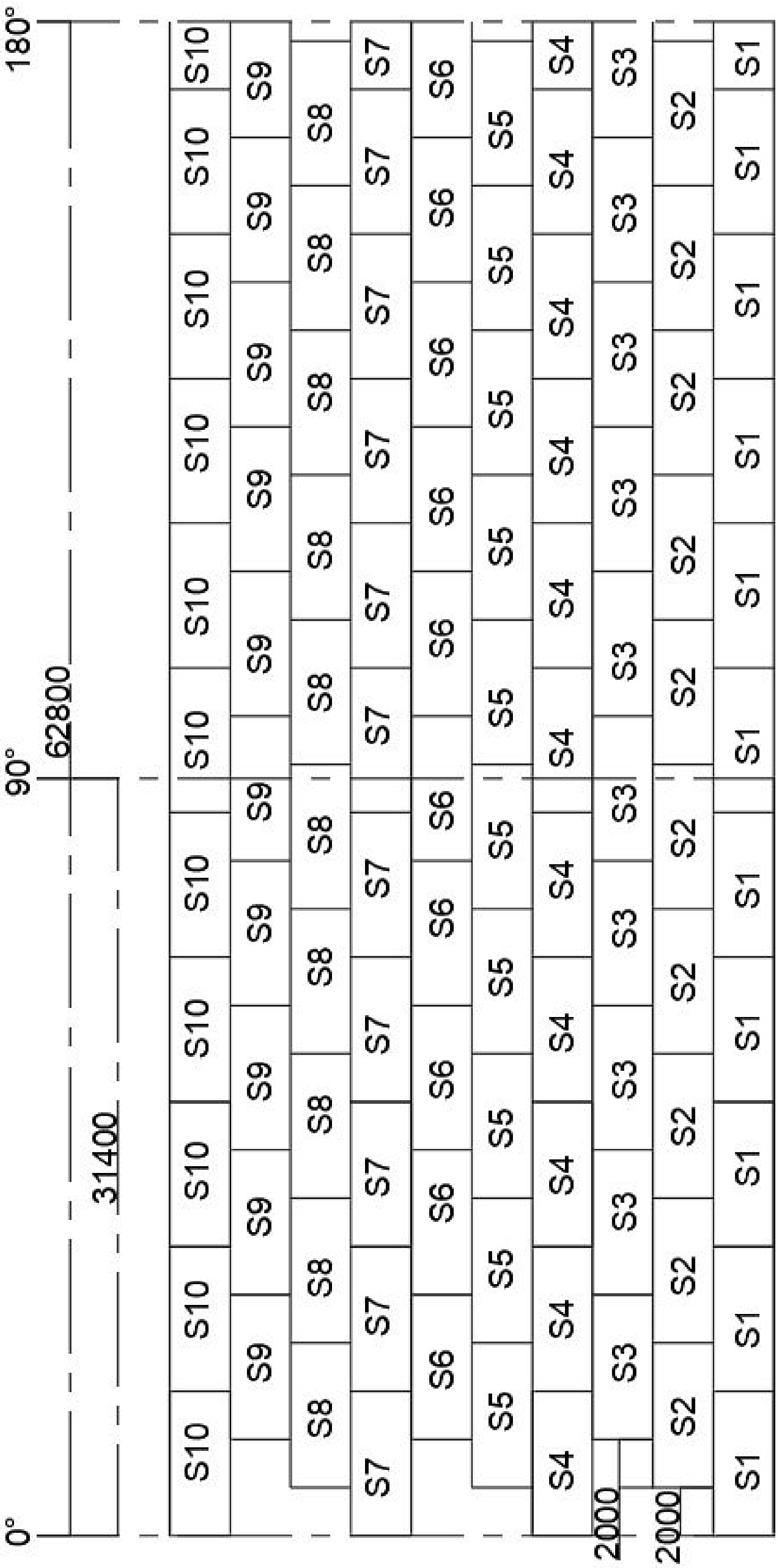

Figure 3.2: Shell Plate Layout-1

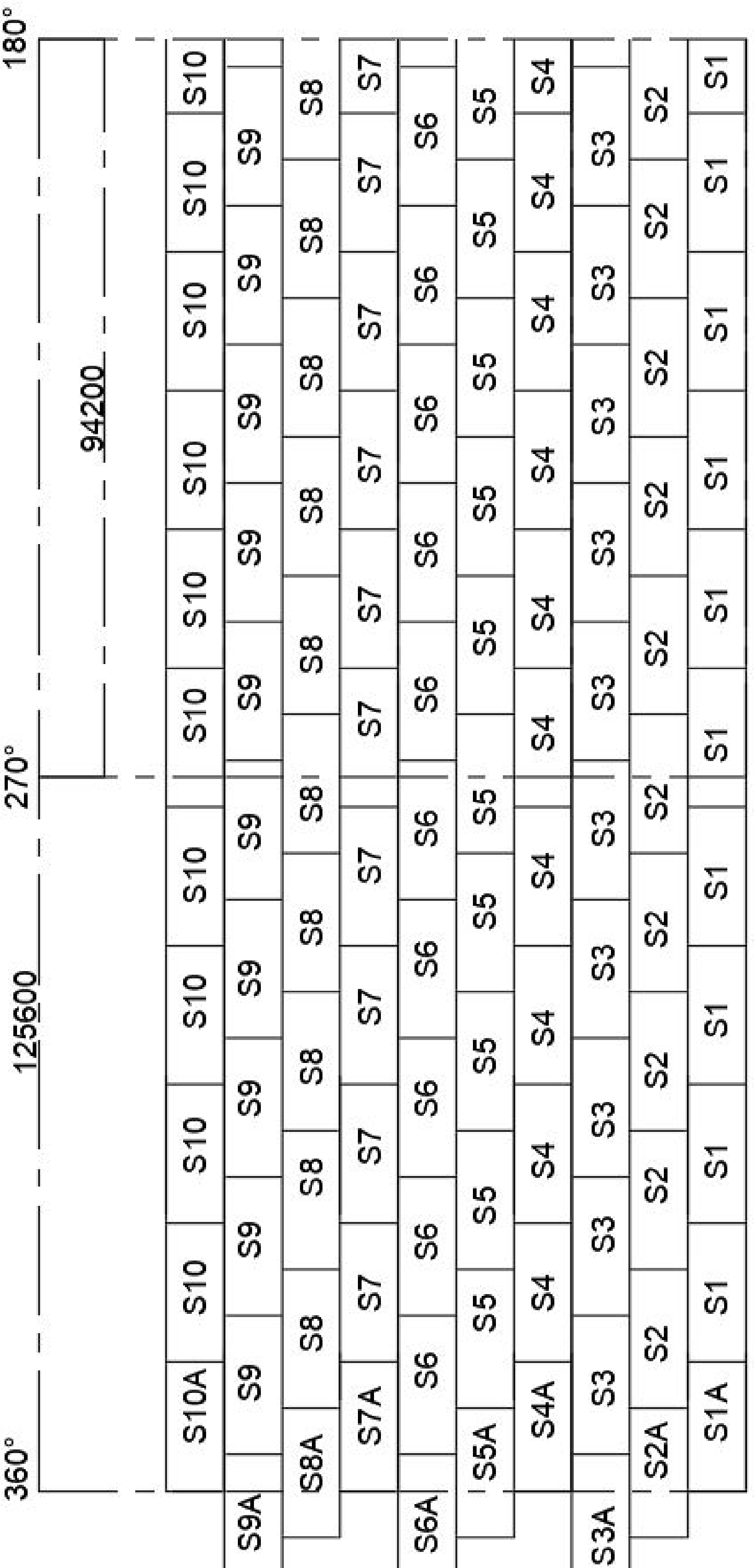

Figure 3.3: Shell Plate Layout-2

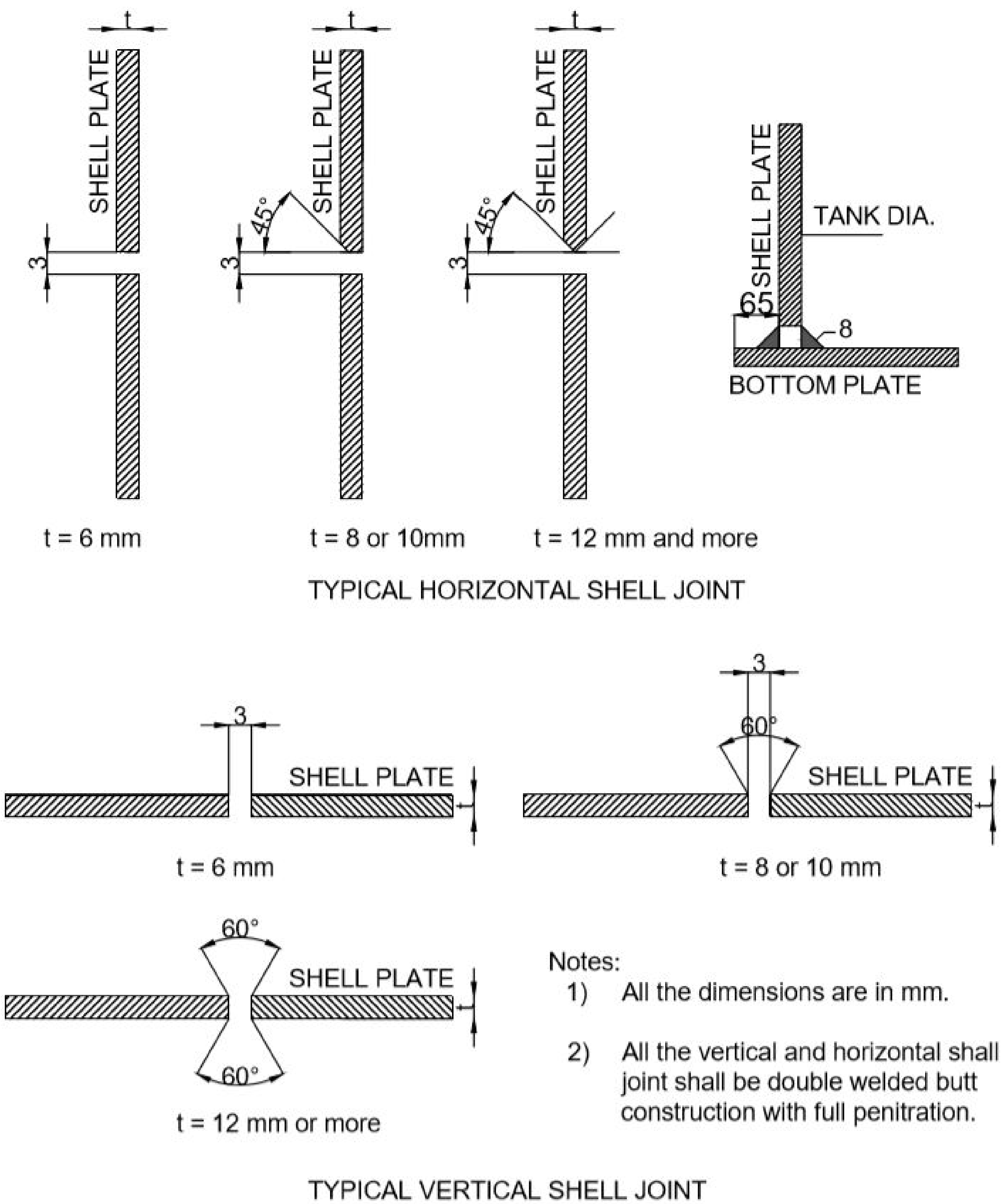

Figure 3.4: Shell Plate Joints-2

Table 3.5: Shell Plates Detail

Shell	**Length (mm)**	**Width (mm)**	**Nos**
S1-S9	6000	2000	20
S1A	5757	2000	1
S2A	5738	2000	1
S3A	5726	2000	1
S4A	5713	2000	1
S5A	5694	2000	1
S6A	5675	2000	1
S7A	5662	2000	1
S8A	5650	2000	1
S9A	5950	2000	1
S10	6000	1970	20
S10A	5950	1970	1

3.5 Bottom Plate Design

3.5.1 Bottom Plate

When the foundation of the tank gets completed and when the tank is ready for erection laying of bottom plates starts on the top of foundation and welded in sequence.As per API 650 **Clause 5.4** the corroded bottom plate thickness should not be less than 6 mm. Width of all the rectangular and sketch plates shall not be less than 1800 mm, unless otherwise specified by the purchaser.

Material= ISO630-E275C

Minimum thickness of bottom plates (t_{min})= 6 mm

Corrosion Allowance (CA_{bottom})= 1.5 mm

Total thickness of bottom plates = 6+1.5 = 7.5 mm

Provided thickness of bottom plates ($t_{provided}$) = 10 mm

3.5.2 Annular Bottom Plate

Annular plates are the thicker plates provided outside of the bottom floor under the shell plates.Annular plates are used for larger diameter tanks.The main purpose of the annular bottom plates is to support the weight of shell courses. Thickness of annular bottom plate should not be less than the greater thickness determined using Table 3.6 for product design or hydrostatic test design.

Table 3.6: Annular Bottom Plate Thickness (t_b)

Nominal Tank Diameter (m)	Stress in First Shell Course (MPa)			
	≤ 190	≤ 210	≤ 220	≤ 250
t ≤ 15	6	6	7	9
$19 \leq t \leq 25$	6	7	10	11
$25 \leq t \leq 32$	6	9	12	14
$32 \leq t \leq 40$	8	11	14	17
$40 \leq t \leq 45$	10	13	16	19

Table 3.6 is applicable for the effective product height of H $\times G \leq 23m$ where H is the height of the tank and G is the specific gravity of the liquid to be stored.Plate thickness in the table refers to the corroded shell plate thickness for product design stress and nominal plate thickness for hydrostatic test design.

Design Calculation

Height of the tank (H) = 20 m

Specific gravity of the liquid to be stored (G) = 1

Bottom (1st) shell course design thickness (t_{d1}) = 28 mm

Bottom (1st) shell course test thickness(t_{t1}) = 25.5

$t_{nominal}$= 26 mm Joint Efficiency Factor (J_r) : 0.85

Material= ISO630-E275C

Maximum Allowable Design Stress (S_d): 164 N/mm^2.

Maximum Allowable Hydrostatic Stress (S_t): 176 N/mm^2.

Corrosion Allowance (CA) = 1.5 mm

γ = Density factor of water MPa per meter SI: 9.81/1000

H $\times G \leq 23m$

20 $\times 0.75 \leq 23m$

$$\text{S1} = \frac{(t_{d1} - CA)}{t_{corroded}} \times S_d$$

S1 = 155 N/mm^2

$$\text{S2} = \frac{t_{t1}}{t_{nominal}} \times S_t$$

S2 = 172 N/mm^2

S = MAX(S1,S2)

S= 172 N/mm^2

Annular bottom plates shall have a radial width that provides at least 600mm between the inside of the shell and any lap-welded joint in the reminder of the bottom. Radial width of annular bottom plate is calculated as follow:

$$L = 2t_b\sqrt{\frac{F_y}{2\gamma GH}} \qquad \text{(Eq. 3.5.1)}$$

$$L = 2 \times 10\sqrt{\frac{265 \times 1000}{2 \times 9.81 \times 1 \times 20}}$$

L = 519.74 mm

Minimum required with of annular bottom plates is 600 mm.

Provided radial width = 600 mm

Annular Bottom Plate Thickness (t_b)= 6 mm

Corrosion Allowance (CA) = 1.5 mm

Final Thickness of Annular Bottom Plate = 10 mm

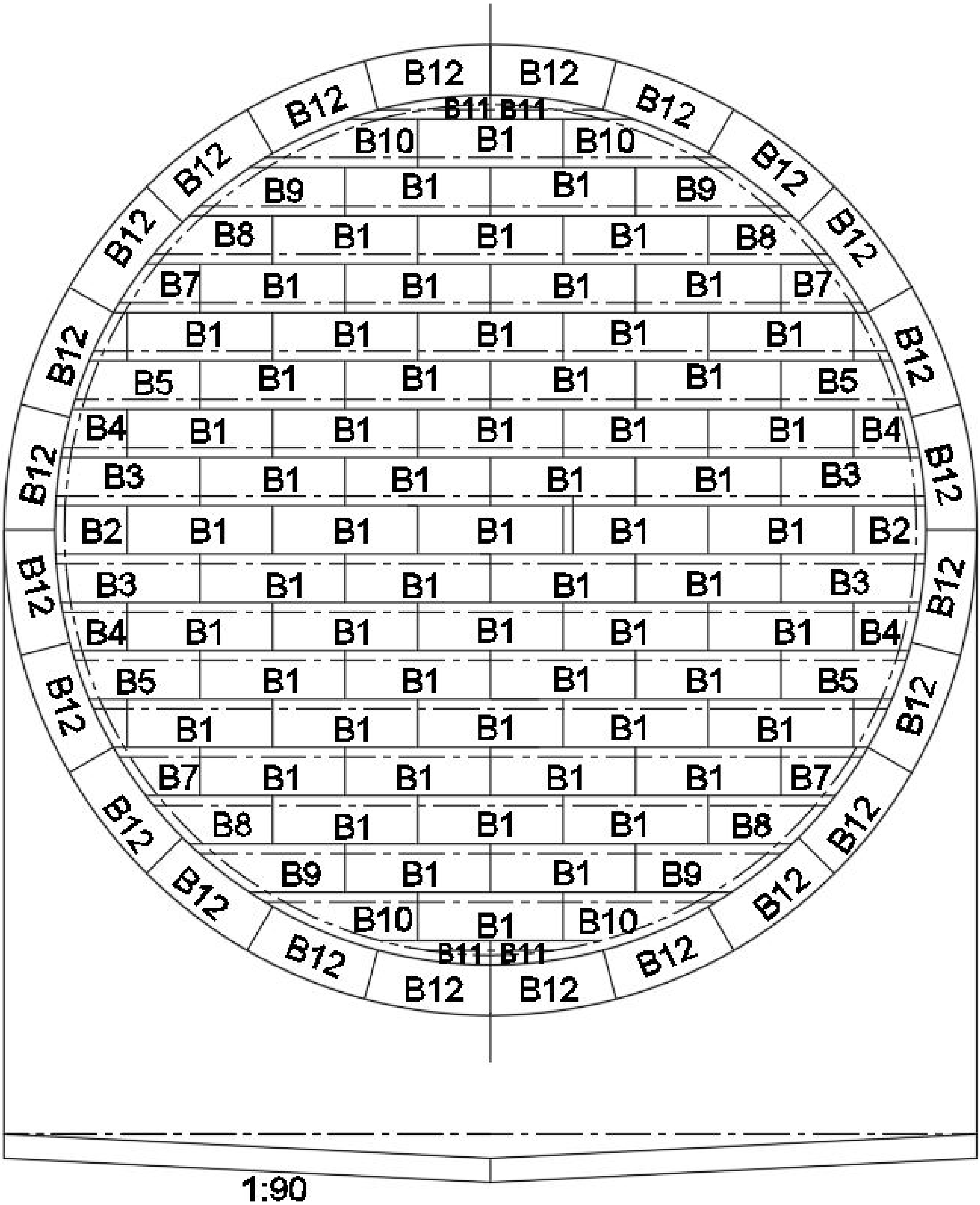

Figure 3.5: Bottom Plate Layout

3.6 Roof Design

Types of roof used to cover the liquid storage tank are:

1. Self/Structurally Supported Fixed Cone Roof.
2. Self/Structurally Supported Fixed Dome Roof.
3. External Floating Roof.
4. Internal Floating Roof.

All this above mentioned roof should be designed for dead load and uniform live load acting and should not less than 1 kPa as per API 650. When the diameter of the tank is large structure support to the roof in form of rafters, girders and column is provided. To design this structurally supported cone roof the following requirements needs to be followed:

1. As per API 650 standard Slope of the roof should not exceed 2:12 for tank having diameter more than 15m.
2. Roof plates of the cone roof should not attached to the the member supporting them.
3. Minimum thickness of roof plate should be of 5mm as per API 650.
4. Welding of the roof plates should be done on the top side with continues fillet weld for all the seams.
5. Structural members which are acting as support to rafters may be rolled or fabricated sections or trusses with or without support.
6. As per API 650 slenderness ratio of the columns should not exceed 180 while other compression member it should not exceed 200.For tension member the maximum value is 300.
7. The spacing of rafters measured along the circumference of the tank should not be more than 2.1 m for outer rings and 1.75 m for inner rings as per API 650 provisions
.

8. Length of rafter between two support should be limited to 7.5m for the economical design.

9. Length of the girder on which rafters are resting should not exceed 8.5m for the economical design.

Present study covers the design of structurally supported fixed cone roof.As the diameter of the tank is large, the structural supported are required to support the roof.Roof is divided in to three number of bays as shown in the Figure.The inside portion is termed as Bay-1 followed by Bay-2 and Bay-3 as we move outward.

3.6.1 Roof Load Calculation

Diameter of the tank (D) = 40 m

Height of the tank (H) = 20m

Radius of Bay-1 = 6.5 m

Radius of Bay-2 = 13 m

Radius of Bay-3 = 20 m

Total projected radius of the roof (R_p) = 20.04 m

Projected area of the roof (A_P) = 1261 m^2

Slope of the roof (θ) = 1:16

tan(θ) = 3.576

Dead load of the roof plates (DL):

DL = $\rho_s \times t_r \times A_p$

DL = $78.53 \times 0.005 \times 1261$

DL = 495 kN

DL = 0.40 kN/m^2

Live load on the roof plate (L_r):

L_r = 1.2 kN/m^2 (As per API)

Design external pressure (P_e)

P_e= 0.27 kPa (Assumed)

Load Combination (As Per API 650)

e1b. $= DL + L_r + 0.4P_e$

e1b. = 1.71 kPa.

e2b. $= DL + P_e + 0.4L_r$

e2b. = 1.12 kPa

T = Max(e1b, e2b)

T = 1.71 kPa.

Bay-1 Detail

Projected radius of the Bay-1 (r_1) = 6.52 m

Maximum length of rafters in Bay-1 (L_1) = 6.52 m

Maximum Spacing of rafters on outer shell ring (S_m)= 2 m. Length of girder in Bay-1 (L_{g1}) = 6.5 m Height of column in Bay-1 C_1c= 20 m

Number of rafters in Bay-1 (N1)

$$N1 = \frac{2 \times \pi \times r_1}{S_m}$$

$$N1 = \frac{2 \times \pi \times 6.5}{2} = 20.41$$

Number of Provided rafters are 24

Actual spacing of rafter (S_1)

$$S_1 = \frac{2 \times \pi \times 6.5}{24} = 1.70\text{m}$$

Load acting on the rafter R1

Total area of Bay-1

$= \pi \times r_1^2$

$= \pi \times (6.52)^2$

$= 133.48\ m^2$

Area per rafter (A_{p1})

= Total area/No. of rafters

$= 133.48/24$

$= 5.56\ m^2$

Dead load acting on each rafter

$= \dfrac{DL \times A_{p1}}{L_1}$

$= \dfrac{0.40 \times 5.61}{6.52} = 0.38$ kN/m

Live load acting on each rafter

$= \dfrac{L_r \times A_{p1}}{L_1}$

$= \dfrac{1.20 \times 5.61}{6.52} = 1.03$ kN/m

Bay-2 Detail

Projected radius of the Bay-2 (r_2) = 13.02 m

Maximum length of rafters in Bay-2 (L_2) = 6.52 m

Maximum Spacing of rafters on outer shell ring (S_m)= 2 m. Length of girder in Bay-2 (L_{g2}) = 9 m Height of column in Bay-2 C_2= 20 m

Minimum number of rafters in Bay-2 (N2)

N2 $= \dfrac{2 \times \pi \times r_2}{S_m}$

N2 $= \dfrac{2 \times \pi \times 13.02}{2} = 40.82$

Number of provided rafters are 48

Actual spacing of rafter (S_2)

$$S_2 = \frac{2 \times \pi \times 13}{48} = 1.70\text{m}$$

Load acting on the rafter R2

Total area of Bay-2

$= \pi \times r_2^2 - \pi \times r_1^2$

$= \pi \times (13.02)^2 - \pi \times (6.52)^2$

= 397.18 m^2

Area per rafter (A_{p2})

= Total area/No. of rafters

= 397.18/48

= 8.29 m^2

Dead load acting on each rafter

$$= \frac{DL \times A_{p2}}{L_2}$$

$$= \frac{0.40 \times 8.29}{6.52} = 0.53 \text{ kN/m}$$

Live load acting on each rafter

$$= \frac{L_r \times A_{p2}}{L_2}$$

$$= \frac{1.20 \times 8.29}{6.52} = 1.53 \text{ kN/m}$$

Bay-3 Detail

Projected radius of the Bay-3 (r_3) = 20.04 m

Maximum length of rafters in Bay-2 (L_3) = 7.5 m

Maximum Spacing of rafters on outer shell ring (S_m)= 2 m. Height of column in Bay-3 C_3= 20 m

Minimum number of rafters in Bay-3 (N3)

$$\text{N3} = \frac{2 \times \pi \times r_3}{S_m}$$

$$\text{N3} = \frac{2 \times \pi \times 20.04}{2} = 62.92$$

Number of provided rafters are 72

Actual spacing of rafter (S_3)

$$S_3 = \frac{2 \times \pi \times 20.04}{72} = 1.75\text{m}$$

Load acting on the rafter R3

Total area of Bay-3

$= \pi \times r_3^2 - \pi \times r_2^2$

$= \pi \times (20.04)^2 - \pi \times (13.02)^2$

$= 728.73\ m^2$

Area per rafter (A_{p3})

= Total area/No. of rafters

= 728.73/72

$= 10.12\ m^2$

Dead load acting on each rafter

$$= \frac{DL \times A_{p3}}{L_3}$$

$$= \frac{0.40 \times 10.12}{7.5} = 0.55 \text{ kN/m}$$

Live load acting on each rafter

$$= \frac{L_r \times A_{p3}}{L_3}$$

$$= \frac{1.20 \times 10.12}{7.5} = 1.63 \text{ kN/m}$$

Analysis and Design of the Roof

The roof of the liquid storage tank is modelled in a STADD Pro Connect edition software for the analysis and design.Analysis is done in several stages:

1. Geometry and modelling of the structure is carried out as per drawing.

2. Material property for each material are defined from the database.

3. Appropriate constrains are provided to the structure.

4. Loading is provided as per above calculation done.

5. Model is set to run.

6. Post Processing has been carried out.

Table 3.7: Structural Member Detail of the Roof

Sr No	Member	Section Provided	Nos.
1	Rafter R1	ISMC 150	24
2	Rafter R2	ISMC 175	48
3	Rafter R3	ISMC 300	72
4	Girder G1	ISMC 300	6
5	Girder G2	ISMC 300	8
6	Center Column C1	ISMC 350 D SP 0.15	1
7	Column C2	ISMC 250 D SP 0.18	6
8	Column C3	ISMC 250 D SP 0.18	8

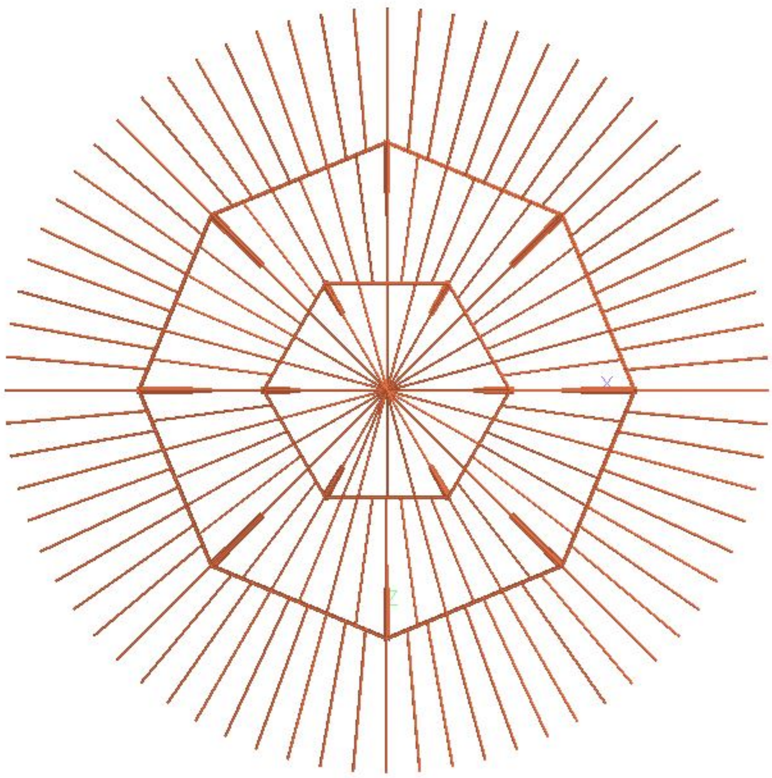

Figure 3.6: Plan of the Roof

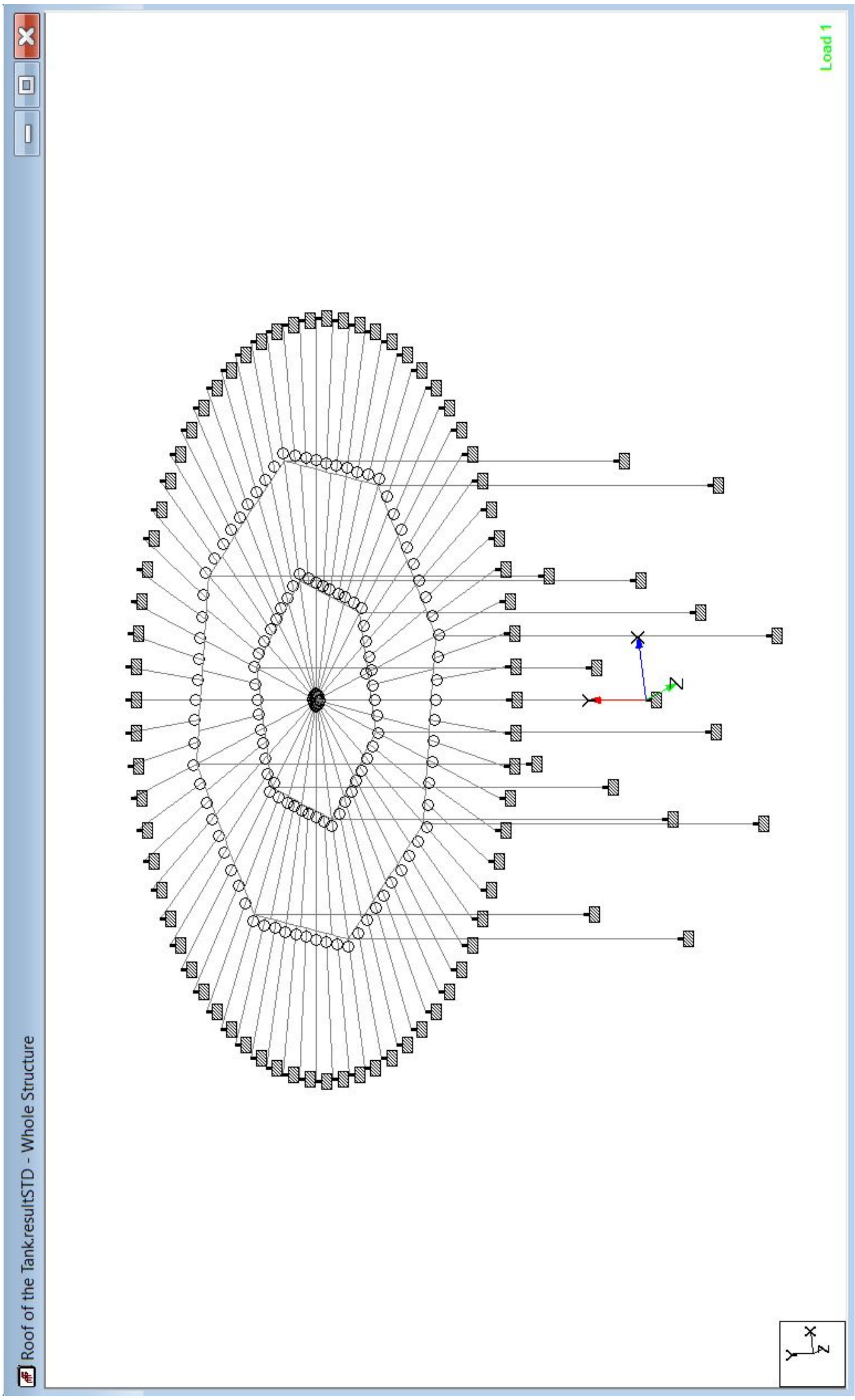

Figure 3.7: Modeling of the Roof in Stadd Pro

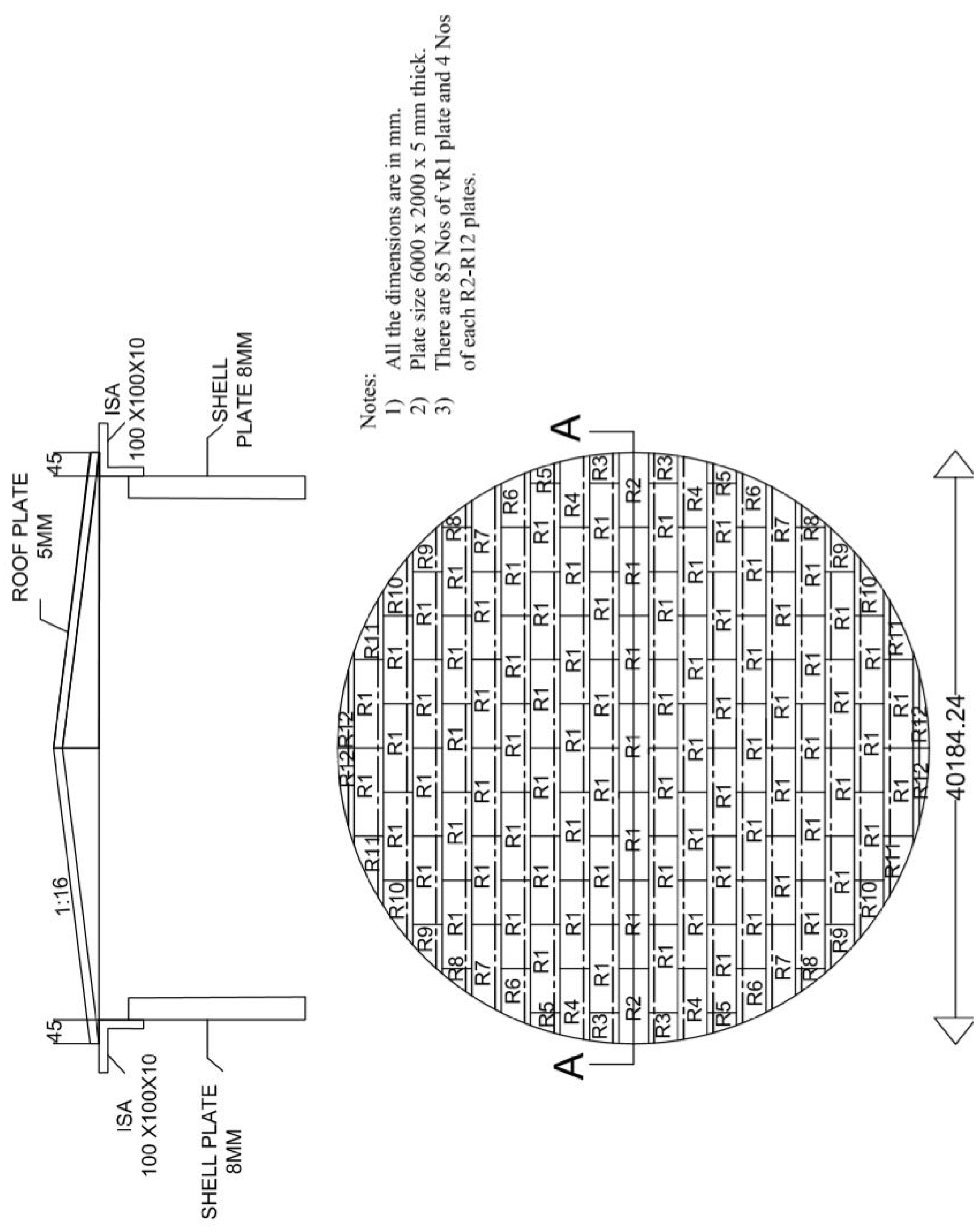

Figure 3.8: Roof Plate Layout

3.7 Wind Analysis of Liquid Storage Tank

Wind analysis of the liquid storage tank is carried out to check the stability of the tank against wind pressure.A tank subjected to wind pressure if not designed properly then it can overturn or slide a tank off its foundation or can cause to the collapse.Mostly the empty storage tanks are more vulnerable to wind forces. Effect of wind force increases with increase in height of the tank.Wind girders are provided as per the requirement to sustain against these wind loads.

3.7.1 Wind Load Calculations

Location of the tank = Bhuj (Gujarat)
The 3-sec gust design wind speed for the tank location = 50m/s(Bhuj) i.e. 180 kmph
The wind pressure used for the analysis is calculated as follows:
Wind pressure acting on the vertical projected area of the shell:
$P_{ws} = 0.86kPa\left(\frac{V}{190}\right)^2 =$ 0.77kPa
Wind pressure acting on the roof:
$P_{wr} = 1.446kPa\left(\frac{V}{190}\right)^2 =$ 1.29kPa

3.7.2 Top and Intermediate Stiffening Rings

A stiffness to the liquid storage tank is provided in two ways:one is by increases the thickness of the tank shell and second is by providing stiffening rings to tank when it is subjected to wind loads.Mostly it is uneconomical to increase the thickness of the shell plates hence second method is followed. These stiffening rings are welded outside the tank shell .Stiffening rings used as a girder can be structural sections, formed plate sections,welded section or combination sections.Stiffening rings are generally referred as wind girders.As per API 650 standard the minimum size of angle used for the stiffening ring shall be 65 × 65 × 6 mm. The minimum plate thickness used for formed or built up rings shall be 6 mm as per API guidelines.

When the wind girder are located more than 0.6 m below from the top of the shell then it is necessary to be provided a curb angle with the $65 \times 65 \times 6mm$ for shell having 5mm thickness and with $75 \times 75 \times 6mm$ angle for shell more than 5mm thickness or with member of equivalent section modulus.

Stiffening Rings as a Walkways

If wind girder or any portion of it is used as a walkways then its width should not be less than 710 mm clear projections including the angle on the top of tank shell. If the tank is not covered with the fixed roof, the girder or rings used as a walkways may be located 1100 mm below the top curb angle. Standard railing on the unprotected side and at the end of the section used as a walkways must be provided.

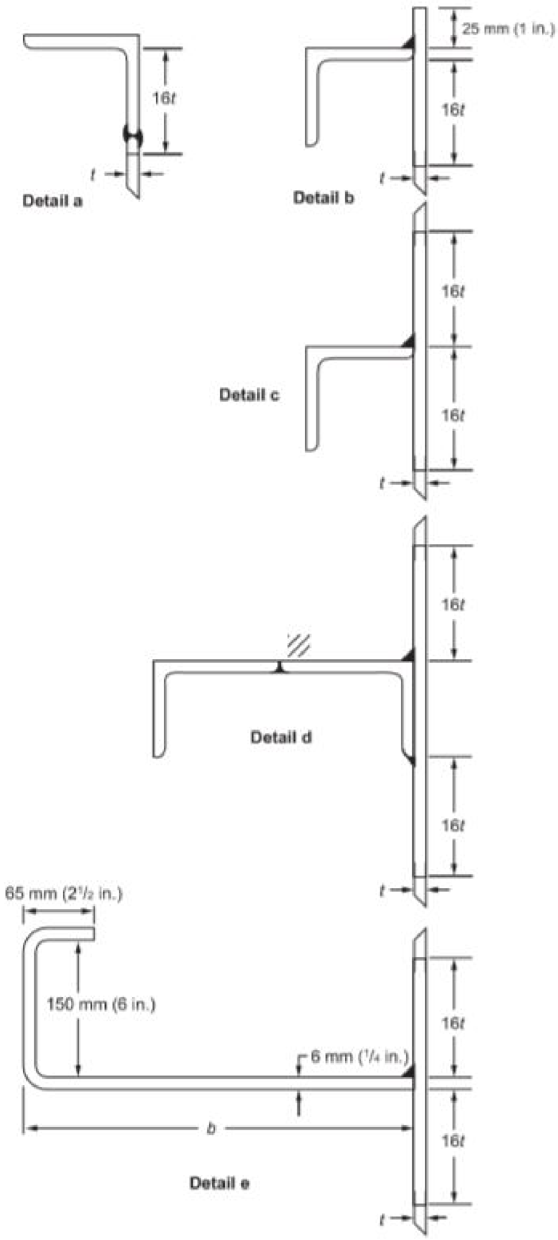

Figure 3.9: Typical Stiffening Ring Sections for Tank Shells [1]

Top Wind Girder

As per API 650 standard all the liquid storage tanks must be provided with the stiffening rings located at or near the top course of the shell.These stiffening rings are top wind girder which are provided to prevent the deformation of the shell and to maintain the roundness of the tank when it is subjected to wind loads. The minimum section modulus of the top wind girder which need to be provided is determined by following equation :

$$Z = \frac{D^2 H_2}{17}\left(\frac{V}{190}\right)^2 \qquad \text{(Eq. 3.7.1)}$$

$$Z = \frac{40^2 \times 20}{17}\left(\frac{180}{190}\right)^2$$

Z = 1689 cm^3

Where,

Z is the required section modulus in cm^3;

D is tank diameter in m;

H_2 is the tank shell height in m including any free-board required in m;

V is the design wind speed (3-sec gust) in kmph

Hence the required section modulus for the top wind girder that need to be provided is 1689cm^3 and the built up thickness of the section provided is 8mm.

Intermediate Wind Girder

The situation where only the top wind girder is not enough to provide sufficient stiffness for the given geometry then the intermediate wind girder should be provided. This intermediate wind girder is installed between the top wind girder and the tank bottom to reduce the unstiffened length of the shell and to prevent the buckling of the shell.

Following steps needed to be performed to check weather the intermediate stiffner is to be provided or not.

Firstly, the maximum height of the unstiffened shell shall be calculated using following equation.

$$H_1 = 9.47t\sqrt{(\frac{t}{D})^3}\left(\frac{190}{V}\right)^2 = 7.55m \qquad \text{(Eq. 3.7.2)}$$

$$H_1 = 9.47 \times 8 \times \sqrt{\left(\frac{8}{40}\right)^3 \left(\frac{190}{180}\right)^2}$$

H_1= 7.55 m where,

H_1 = Vertical distance between intermediate girder and the top curb angle of the shell in m;

t = the thickness of the thinnest shell course in mm;

D = Tank diameter in m;

The thickness of the shell decreases with increase in tank height hence analysis is complex. The equivalent shell method is used to convert multi-thickness shell course into equivalent thickness as to the top shell course of the storage tank.

a) After calculating the maximum height of the unstiffened shell H_1 , the width of each transformed shell course is calculated as:

$$W_{tr} = W\sqrt{\left(\frac{t_{uniform}}{t_{actual}}\right)^5} \qquad \text{(Eq. 3.7.3)}$$

Where

W_{tr} is the transposed width of each shell course, in millimeters (inches)

W is the actual width of each shell course, millimeters

$t_{uniform}$ is the nominal thickness, unless otherwise specified, of the thinnest shell course, in mm

t_{actual} is the nominal thickness, unless otherwise specified, of the shell course for which the transposed width is being calculated, in millimeters .

b) In the second step transformed width of each shell course is summed up and this sum of transposed widths of the each shell courses will give the total height of the transformed shell (H_2) as shown in Table 3.8 .

c) If the height of the transformed shell (H_2) is found to be greater than the maximum unstiffened shell height (H_1),then an intermediate wind girder is to be provided.

d) To keep the equal stability above and below the intermediate wind girder, the location of girder should be at the mid-height of the transformed shell.

The maximum unstiffened shell height be calculated is 7.55 m which is greater than the transformed shell height 7.11 m. Hence there is no need to provide intermediate wind girder .

If intermediate girder needs to be provided then the section modulus girder can be determined using the following equation:

$$Z = \frac{D^2 H_1}{17} \left(\frac{V}{190} \right)^2 \quad \text{(Eq. 3.7.4)}$$

Where,

Z is the required section modulus in cm^3;

D is tank diameter in m;

H_1 is the vertical distance between the top angle and intermediate wind girder in m;

V is the design wind speed (3-sec gust) in kmph

Table 3.8: Height of Transformed Shell Course

Shell Course	**W (mm)**	$\mathbf{t}_{uniform}$ **(mm)**	$\mathbf{t}_{actual}$ **(mm)**	$\mathbf{W}_{tr}$ **(mm)**
10	2000	8	28	87.27
9	2000	8	25	115.85
8	2000	8	22	159.48
7	2000	8	20	202.38
6	2000	8	18	263.37
5	2000	8	15	415.46
4	2000	8	12	725.77
3	2000	8	10	1144.86
2	2000	8	8	2000
1	2000	8	8	2000
Total				7.11 m

3.7.3 Stability against Overturning

Overturning Stability is calculated using the design wind pressure calculate as per Equation to determine stability of the tank with or without anchorage. As per API 650 standard unanchored storage tanks are stable if it meets the following requirement:

1. $0.6M_w + M_{Pi} < \left(\frac{M_{DL}}{1.5}\right) + M_{DLR}$

2. $M_w + F_P(M_{Pi}) < \left(\frac{M_{DL} + M_F}{2}\right) + M_{DLR}$

3. $M_{ws} + F_P(M_{Pi}) < \left(\frac{M_{DL}}{1.5}\right) + M_{DLR}$

Where,
M_W is the wind overturning moment about the shell to bottom joint and is due to the wind loading acting on the shell and roof of the tank.It is calculated as:
$M_w = F_r L_r + F_s L_s$.
M_w = (1620.25)(20.005) + (616)(10)
$M_w = 385.72 \times 10^5$ N.m

F_r is the wind load acting on the roof and is calculated as $P_{wr} \times \frac{\pi}{4} \times D^2$ and is equal to 1620.25 kN. F_s is the wind load acting on the shell plates and is calculated as $P_{ws} \times D \times H$ and is equal to 616 kN.

L_r and L_s are the height from the bottom of the tank to roof and shell centre and is equal to 20.005 m and 10 m respectively.

M_{Pi} is the moment due to design internal pressure about the shell-to-bottom joint.

M_{DL} is the moment due to weight of the shell at the shell-to-bottom joint which is calculated as $0.5 \times D \times W_{DL}$ and is equal to 671.60 $\times 10^5$ N.m

M_{DLR} is the moment due to weight of the roof and attached structural acting at the shell-to-bottom joint which is calculated as $0.5 \times D \times W_{DLR}$ and is equal to 130 $\times 10^5$ N.m.

F_p is the pressure combination factor and is equal to 0.4.

M_F is the moment due to weight of the liquid

$M_F = (w_L \pi D)(\frac{D}{2})$

$M_F = (6875 \times 3.14 \times 40)(\frac{40}{2})$

$M_F = 1726 \times 10^5$ N.m

w_L is the resisting weight of the tank liquid per unit length of shell circumference.

$w_L = 59 \times t_b \times \sqrt{F_{by} H}$

$w_L = 59 \times 16 \times \sqrt{265 \times 20}$

$w_L = 6875$ N/m

where,

F_{by} is the minimum yield stress of the annular bottom plate, in MPa;

H is the design liquid level in m;

D is the tank diameter in m;

t_b is the thickness of the annular bottom in mm;

Criteria 1

$$0.6M_w + M_{Pi} < \left(\frac{M_{DL}}{1.5}\right) + M_{DLR}$$

$385.72 \times 10^5 N.m < 577 \times 10^5 N.m$

Criteria 2

$$M_w + F_P(M_{Pi}) < \left(\frac{M_{DL} + M_F}{2}\right) + M_{DLR}$$

$231 \times 10^5 N.m < 1328 \times 10^5 N.m$

Criteria 3

$$M_{ws} + F_P(M_{Pi}) < \left(\frac{M_{DL}}{1.5}\right) + M_{DLR}$$

$231 \times 10^5 N.m < 466 \times 10^5 N.m$

Hence above all the criteria are satisfied and tank is structurally stable without anchorage under wind loads.

3.8 Summary

Design of 22 millions liters capacity liquid storage tank having various components such as shell design, bottom plate design, annular plate design has been carried out using API650 codal provisions. Analysis and design of structurally supported cone roof tank is also performed using Stadd Pro software.The wind loads acting on the tank has also been evaluated.The section modulus of the top wind girder and the intermediate wind girder which are need to be provided has also been calculated.Stability of liquid storage tank against wind load is checked and it is found to be stable.Erection drawing of the shell plates, bottom plates and roof plates are drawn in Autocad software.

Chapter 4

Seismic Analysis of Liquid Storage Tank

4.1 General

After the gravity load analysis of the storage tank in previous chapter the seismic analysis of the storage tank is performed.This chapter deals with the different codal provisions used for the seismic analysis of storage tanks. Seismic design of a liquid storage tank using both API 650 guideline and Indian standard code IS:1893(Part-2):2014 are covered. Storage tank of 22 million liters capacity is considered in highest seismic zone of the country as per Indian codal provisions.The tank is assumed to be self-anchored on a foundation. Further the design of foundation, erection plan of the tank along with erection drawing are discussed.

4.2 Spring-Mass Model of the Tank

When a liquid storage tank having liquid inside is subjected to vibration,then the liquid inside the storage tank exerts some extra pressure on the wall and base of the storage tank along with that of hydrostatic pressure.This extra pressure acting is hydrodynamic pressure.To evaluate this hydrodynamic pressure housner has fond a spring-mass analogy model of tank.In Spring-Mass analogy model liquid inside the tank is modeled in to two parts: impulsive component and convective component.Impulsive component is modelled as a rigid link while convective component is modelled as spring with given stiffness oscillating in fundamental mode. Both IS:1893(Part-2)-2014 and API 650 Standard considers this spring-mass analogy for the modelling of liquid of the tank while performing the seismic analysis as shown in Figure 4.1.

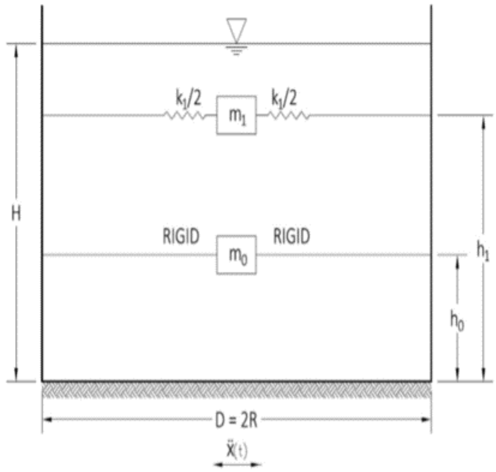

Figure 4.1: Spring-Mass Analogy Model of Liquid Storage Tanks

4.3 Major Analysis to be Performed for Seismic

There are major three analysis to be performed for the seismic design of the liquid storage tank.They are as follows:

(i) Stability against Overturning - The overturning moment of the tank is evaluated to check whether tank is stable or not under seismic loads.It is also used to determine whether tank can be self anchored or need any mechanical anchorage devices.

(ii) Maximum Base Shear Calculation.

(iii) Freeboard Requirement - It is necessary to provide freeboard to the storage tank to stop the spillage of the product and also to prevent the roof damage due to sloshing of liquid.

4.4 Problem Formulation

Design of 22 Million liters capacity of Liquid Storage Tank used to store petrol is considered for the present study. Following parameters are considered for the analysis and design of the tank; Diameter of the tank is 40 m, Height of the tank is 20 m, Material stored in the tank is Petrol, Specific Gravity of petrol 0.75, Density of liquid to be stored as 753 kg/m^3, Joint efficiency factor as 0.85 due to spot radiography examination of joint, Allowable corrosion as 1.5 mm and Tank is situated in the Seismic zone V as per Indian codal provisions.

4.5 Code base Seismic Analysis of the Tank

4.5.1 Seismic Analysis of the Tank as per IS:1893(Part-2)-2014

IS:1893(Part-2)-2014 covers the criteria for earthquake resistance design of ground supported as well as elevated liquid storage tanks[3].Seismic parameters: Time period - impulsive and convective, Spectral acceleration, Base Shear, Base Moment and Overturning Moment are evaluated. Hydrodynamic pressure acting on the tank and sloshing wave height of the liquid inside the tank are also evaluated.

4.5.1.1 Seismic Parameters

Impulsive Time Period

Impulsive mass is the lower portion of the liquid which is observed to be move with the tank and time required for it is termed as impulsive time period. It is further use to calculate impulsive base shear and bending moment of the tank.Time period of impulsive mode of the ground supported circular tank is calculated by using following IS code equation [3]

$$T_i = C_i \times \frac{h\sqrt{\rho}}{\sqrt{\frac{t}{d}}\sqrt{E}} \quad \text{(Eq. 4.5.1)}$$

C_i = Coefficient of impulsive modes; h = Maximum height of liquid stored; D = Diameter of circular tank; t = Wall thickness of tank ; E = Modulus of elasticity of shell plate ; ρ= Density of the liquid stored.

Convective Time Period

Convective mass is upper portion of the liquid which is observed to move coincidentally with tank. Convective time period is further used for the calculation of convective base shear and moment. Time period of convective mode can be calculated using following formula [3].

$$T_c = C_c \times \sqrt{\frac{D}{g}} \tag{Eq. 4.5.2}$$

C_c= Coefficient of time period of convective mode; D= Inner diameter of the tank; g= Gravitational acceleration;

Damping

As per codal provisions value of damping coefficient of convective mode for all the types of liquid storage tank is 0.5% of the critical and that of impulsive mode damping for steel tanks is 2% of the critical. Value of multiplying factor used is 1.4 for 2% and 1.75 for 0.5% of damping. [3].

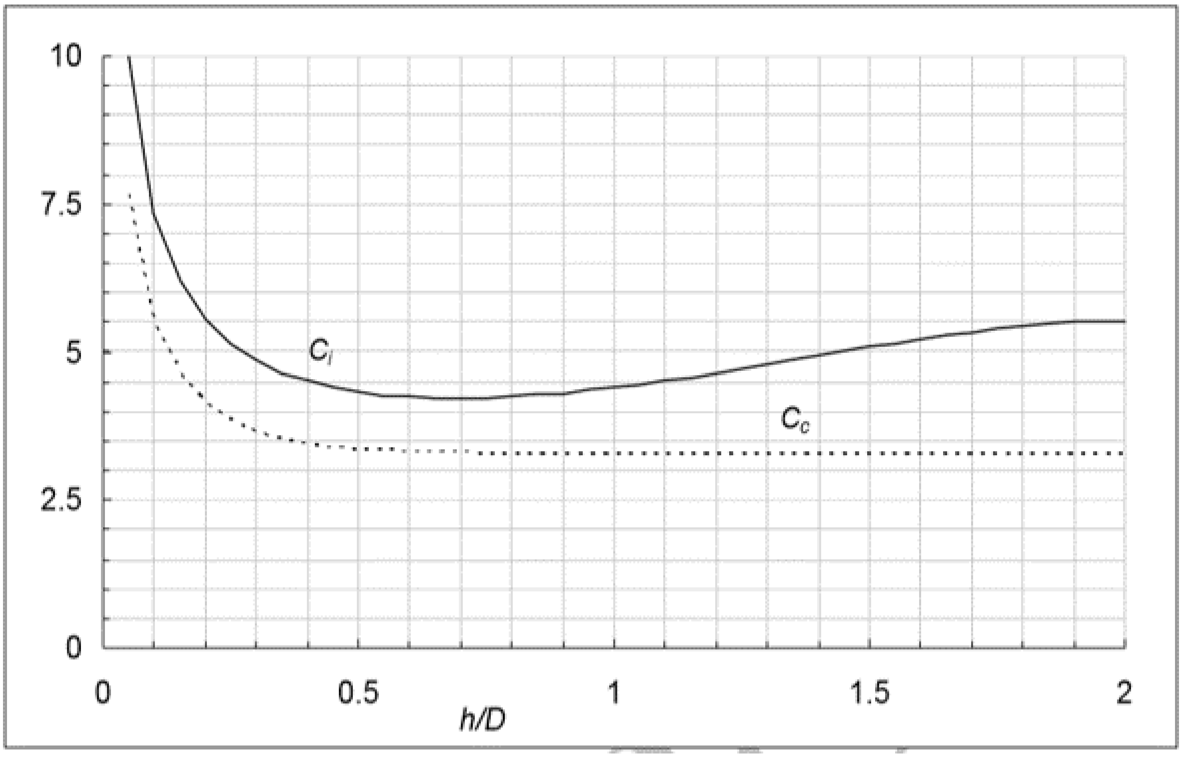

Figure 4.2: Coefficient of Time Period for Impulsive & Convactive Mode of Circular Tank

Horizontal Design Seismic Coefficient

This seismic coefficient is calculated separately for impulsive mode and convective mode [4].

$$A_h = \frac{Z}{2} \times \frac{S_a}{g} \times \frac{I}{R} \tag{Eq. 4.5.3}$$

Z= zone factor; I= Importance factor; R= Response reduction factor.
$\dfrac{S_a}{g}$= Average acceleration coefficient

Base Shear

Impulsive base shear at the base of the tank is given by:

$$V_i = (A_h)(m_i + m_w + m_t)g \tag{Eq. 4.5.4}$$

Convective base shear at the base of the tank is given by:

$$V_c = (A_h)_c m_c g \tag{Eq. 4.5.5}$$

Total base shear is given by:

$$V = \sqrt{V_i^2 + V_c^2} \tag{Eq. 4.5.6}$$

Base Moment

Impulsive bending moment at the bottom of the tank wall is given by:

$$M_i = (A_{h_i})(m_i h_i + m_w h_w + m_t h_t)g \tag{Eq. 4.5.7}$$

Convective bending moment at the bottom of the tank wall is given by:

$$M_c = (A_{h_c})m_c h_c g \tag{Eq. 4.5.8}$$

Impulsive overturning moment & convective overturning moment used for checking stability at the bottom of base plate is given by:

$$M_i^* = (A_h)_i)[m_i(h_i^* + t_b) + m_w(h_w^* + t_b) + m_t(h_t^* + t_b) + \frac{m_b t_b}{2}] \tag{Eq. 4.5.9}$$

$$M_c^* = (A_h)_c m_c(h_c^* + t_b) \tag{Eq. 4.5.10}$$

Total moment can be obtained by:

$$M = \sqrt{M_i^2 + M_c^2} \tag{Eq. 4.5.11}$$

M_i= impulsive mass of liquid; M_w= Mass of the tank wall; M_t= mass of roof slab; g = gravity acceleration ; t_b= Thickness of base plate; h_w= height of centre of gravity of wall mass;h_t= height of cg of wall mass from bottom; h_i^* = height of impulsive component from the bottom of tank wall; h_c^* = height of convective component from the bottom of tank wall; t_b= Thickness of base slab.

Weight Calculation

(i) Weight of the tank wall

$= \pi \times D \times t \times \rho \times h$

= 3198 kN

Mass of the tank wall (M_w)

= (3198 × 1000)/9.81

= 325991 kg

Note: as the thickness of the wall decrease as the height of the tank increases.Hence the varying thickness has been taken to calculate weight of the tank wall.Detail Calculation are shown in Appendix.

(ii) Weight of the base plate

$=\pi \times r^2 \times t \times \rho$

$=\pi \times (20.025)^2 \times 0.010 \times 78.53$

= 986.37 kN

Mass of the tank wall (M_b)

= (986.37 × 1000)/9.81

= 100544 kg

(iii) Weight of the liquid

= Volume of liquid ×9.81

= 162512.46 kN

Mass of the liquid (M)

= (162512.46 × 1000)/9.81

= 16566000 kg

(iv) Weight of the roof and accessories

= 650 kN

Mass of the roof

$= (650 \times 1000)/9.81$

$= 66259$ kg

Parameters of Spring-Mass Model

Height of the storage tank (H) = 20 m

Diameter of the storage tank (D) = 40 m

$$\frac{H}{D} = \frac{20}{40} = 0.5$$

$$\frac{M_i}{M} = 0.54$$

$$M_i = 0.54 \times 16566000 = 8984011.88kg$$

$$\frac{M_c}{M} = 0.44$$

$$M_c = 0.44 \times 17600000 = 7245401.27kg$$

$$\frac{h_i}{h} = 0.375$$

$$h_i = 7.5 \text{ m}$$

$$\frac{h_c}{h}=0.603$$

$$h_c = 12 \text{ m}$$

$$\frac{h_i^*}{h} = 0.797$$

$$h_{i^*} = 15.93 \text{ m}$$

$$\frac{h_c^*}{h}=0.782$$

$$h_{c^*} = 15.64 \text{ m}$$

Note: From the above study it was observed 54% mass is excited in impulsive mode & 44% mass is excited in convective mode.

Time Period Calculation

Parameters required for the time period calculation are: $C_i = 4.33$; $C_c = 3.36$; h=20 m; $\rho = 800$ kg/m^3; t= 28 mm; D= 40 m; g= 9.81m/s^2; $E = 2 \times 10^{11}$

(i) Impulsive mode time period (From Eq. 4.5.1)

$$=\frac{4.33 \times 20 \times \sqrt{800}}{\sqrt{\frac{0.028}{40}} \times \sqrt{2 \times 10^{11}}} = 0.20 \text{ sec}$$

(ii) Convective mode time period (From Eq. 4.5.2)

$$=3.36 \times \sqrt{\frac{40}{9.81}} = 6.78 \text{ sec}$$

Design Horizontal Acceleration

(i) For Impulsive Mode (From Eq. 4.5.3)

$$(A_h)_i = \frac{0.36}{2} \times \frac{1.5}{2.5} \times (2.5 \times 1.4) = 0.378$$

(ii) For convective mode (From Eq. 4.5.3)

$$(A_h)_c = \frac{0.36}{2} \times \frac{1.5}{2.5} \times (0.25 \times 1.75) = 0.064$$

Base Shear Calculation

(i) Impulsive Base Shear (From Eq. 4.5.4)

$V_i = (0.378) \times (8984011.88 + 325991 + 66259) \times 9.81 = 34768.86$ kN

(ii) Convective Base Shear (From Eq. 4.5.5)

$V_c = (0.064) \times 7245401.27 \times 9.81 = 4548.95$ kN

(iii) Total Base Shear (From Eq. 4.5.6)

V $= \sqrt{(34768.86)^2 + (4548.95)^2} = 35067.58$ kN

Total base shear is approximately 20 %

Bending Moment at the Bottom of Wall

(i) Impulsive Bending Moment (From Eq. 4.5.7)

$M_i = 0.378[(8984011.88 \times 7.5) + (325991 \times 10) + (66259 \times 20.0025)] \times 9.81 = 266860.45$ kN.m

(ii) Convective Bending Moment (From Eq. 4.5.8)

$M_c = 0.064 \times 7245401.27 \times 12 \times 9.81 =$ 55111.75 kN.m

(iii) Total Bending Moment (From Eq. 4.5.11)

M=$\sqrt{(266860.45)^2 + (55111.75)^2}$= 272.50 MN.m

Overturning Moment

(i) In Impulsive Mode (From Eq. 4.5.9)

$M_i^* = 0.378 \times [8984011.88(16 + 0.025) + 325991(10 + 0.025) + 66259(20.005 + 0.025) + \frac{100544 \times 0.010}{2}] \times 9.81 = 548.37$ MN.m

(i) In Convective Mode (From Eq. 4.5.10)

$M_c^* = 0.064 \times 7245401.27 \times 16.025 \times 9.81 = 71.49$ MN.m

(iii) Total Bending Moment (From Eq. 4.5.11)

M=$\sqrt{(548.37)^2 + (71.49)^2}$= 553 MN.m

4.5.1.2 Hydrodynamic Pressure

When the tank is subjected to seismic loading then hydrodynamic pressure needs to be calculated in addition to that of hydrostatic pressure. This hydrodynamic pressure are impulsive and convective hydrodynamic pressure at the wall and base of the storage tank.As per codal provisions they are calculated using following equation Eq. 4.5.12 and

Eq. 4.5.13. **Impulsive Hydrodynamic Pressure**

$$P_{iw} = Q_{iw}(y) \times (A_h)_i \times \rho \times g \times h \times \cos\phi \qquad \text{(Eq. 4.5.12)}$$

$$Q_{iw}(y) = 0.866 \times [1 - (\frac{y}{h})^2] \times \tan h \times (0.866\frac{D}{h}) \qquad \text{(Eq. 4.5.13)}$$

$Q_{iw} = 0.813$

$P_{iw}(y = 0) = 0.813 \times 0.378 \times 753 \times 9.81 \times 20 \times 1$

$P_{iw} = 45.43 kN/m^2$

Maximum pressure occurs at $\phi = 0$

Convective Hydrodynamic Pressure

$P_{cw} = Q_{cw}(y) \times (A_h)_c \times \rho \times g \times D \times [1 - 0.33\cos^2\phi] \times \cos\phi$

$$Q_{cw} = 0.5625 \times \frac{\cosh 3.676\frac{y}{D}}{\cosh 3.674\frac{h}{D}}$$

$Q_{cw} = 0.175$

$P_{cw}(y = o) = 0.175 \times 0.064 \times 753 \times 9.81 \times 20 \times 0.67 \times 1$

$P_{cw} = 2.21$ kN/m^2

Pressure Due to Wall Inertia

$$P_{ww} = (A_h)_i \times t \times \rho_m \times g \qquad \text{(Eq. 4.5.14)}$$

$P_{ww} = 0.378 \times 0.028 \times 78.53 \times 9.81 = 8.15 kN/\mathrm{m}^2$

Effect of Vertical Ground Acceleration

$$P_v = A_v \times \rho \times g \times h \times (1 - \frac{y}{h})$$

$$A_h = \frac{2}{3} \times \frac{Z}{2} \times \frac{S_a}{g} \times \frac{I}{R}$$

$$A_v = \frac{2}{3} \times \frac{0.36}{2} \times 3.5 \times \frac{1.5}{2.5} = 0.25$$

At base of the wall at y=0

$$P_v = 0.25 \times 753 \times 9.81 \times 20 \times (1 - \frac{0}{20}) = 37.23 \text{ kN}/m^2$$

Maximum Hydrodynamic Pressure

$P = \sqrt{(P_{iw} + P_{ww})^2 + P_{cw}^2 + P_v^2}$

$P = \sqrt{(45.43 + 8.15)^2 + 2.21^2 + 37.23^2}$

P = 65.20 kN/m^2

Hydrostatic pressure at the base of the wall = ρ g h= 147.73 kN/m^2

Note: Form the above study it was noted that the value hydrodynamic pressure is about 44% of that of hydrostatic pressure of the product.Hence we cannot neglect the pressure due to the hydrodynamic effect while performing seismic analysis.

4.5.1.3 Sloshing Wave Height

Sloshing wave of high amplitude causes damage to the roof and make them temporarily unserviceable. Spillage of liquid over the roof causes the fires.The height of this sloshing wave is calculated using Eq. 4.5.15.

$$d_{max} = (A_h)_c \times R \times \frac{D}{2} \qquad \text{(Eq. 4.5.15)}$$

$d_{max} = 0.064 \times 2.5 \times \dfrac{40}{2}$

$d_{max} = 3.21$ m

4.5.2 Seismic Analysis of the Tank as Per API 650

API 650 standard covers the design of welded steel liquid storage tanks.Annex E of API 650 standard deals with the minimum requirements for the design of welded steel storage tanks subjected to the seismic ground motion [1].Provisions of the said Annex E is based on the allowable stress design (ASD) method using response spectra analysis [1].The design procedure outlined adopt the 5% damped response spectra for impulsive component and 0.5% damped response spectra for the convective component.

4.5.2.1 Seismic Parameters

Time Period Calculation

Parameters required for the time period calculation are as follows:

Coefficient of impulsive time period (C_i) = 6.35

Coefficient of convective time period (K_s)= 0.6

Maximum depth of liquid (H) = 20m;

Thickness of tank wall (t) = 0.028m

Modulus of elasticity of tank wall (E) = $2 \times 10^{11} N/mm^2$

Mass density of the liquid (ρ) = 753 kg/m^3

(i) Impulsive Time Period of the Tank

$$T_i = \left(\frac{1}{\sqrt{2000}}\right)\left(\frac{C_i H}{\frac{\sqrt{t_u}}{D}}\right)\left(\sqrt{\frac{\rho}{E}}\right) \quad \text{(Eq. 4.5.16)}$$

$$T_i = \left(\frac{1}{\sqrt{2000}}\right)\left(\frac{6.35}{\frac{\sqrt{0.028}}{40}}\right)\left(\sqrt{\frac{753}{2 \times 10^{11}}}\right) = 0.21 \text{ sec}$$

(ii) Convective Time Period of the Tank

$$T_c = 1.8 K_s \sqrt{D} \quad \text{(Eq. 4.5.17)}$$

$$T_c = 1.8.6 \times \sqrt{40} = 6.83 \text{ sec}$$

Where $K_s = \dfrac{0.578}{\sqrt{\tanh\left(\frac{3.68H}{D}\right)}}$ =0.6

Weight Calculation

(i) Weight of the tank wall (W_s)

$= \pi \times D \times t \times \rho \times h$

= 3198 kN

(ii) Weight of the base plate (W_f)

$=\pi \times r^2 \times t \times \rho$

$=\pi \times (20)^2 \times 0.010 \times 78.53$

=986.34 kN

(iii) Weight of the liquid (W_p)

= Volume of liquid $\times 9.81 \times \rho$

= 162512 kN

(iv) Weight of the roof (W_r)

= 650 kN

Effective Weight of the Product

Height of the tank (H) = 20 m

Diameter of the storage tank (D) = 40 m

For $\dfrac{H}{D} = \dfrac{20}{40} = 0.5$

For $\dfrac{D}{H} = \dfrac{40}{20} = 2.0$

When ratio of $\dfrac{D}{H}$ is greater than 1.333 then the impulsive weight of the product is given by:

$$W_i = \frac{\tanh\left(0.866\dfrac{D}{H}\right)}{0.866\dfrac{D}{H}} W_p \qquad \text{(Eq. 4.5.18)}$$

The convective weight of the product is given by:

$$W_c = 0.230\frac{D}{H}\tanh\left(\frac{3.67H}{D}\right) W_p \qquad \text{(Eq. 4.5.19)}$$

Impulsive Weight = 88133.16 kN

Convective Weight =71041.33 kN

Centre of Action for Effective Lateral Forces

The height of centre of action of impulsive mode and convective mode from base of the tank is obtained from the following equation. (i) **Ring-wall Moment**

The height of the centre of action of lateral forces due to Impulsive Weight from the tank bottom is given by X_i.

When $\dfrac{D}{H}$ is ≥ 1.333 then,

$$X_i = 0.375H \qquad \text{(Eq. 4.5.20)}$$

$X_i = 7.5$ m

The height of the centre of action of lateral forces due to Convective Weight from the tank bottom is given by X_c.

$$X_c = \left[1 - \frac{\cosh\left(\frac{3.67H}{D} - 1\right)}{\frac{3.67H}{D}\sinh\frac{3.67H}{D}}\right] H = 12.10m \qquad \text{(Eq. 4.5.21)}$$

(ii) For Slab Overturning Moment

When $\dfrac{D}{H}$ is ≥ 1.333 then,

$$X_{is} = 0.375\left[1.0 + 1.333\left(\frac{0.866\frac{D}{H}}{\tanh\left(\frac{0.866D}{H}\right)} - 1.0\right)\right] H \qquad \text{(Eq. 4.5.22)}$$

X_{is}= 15.94 m

The height X_{cs} is given by:

$$X_c = \left[1 - \frac{\cosh\left(\frac{3.67H}{D} - 1.937\right)}{\frac{3.67H}{D}\sinh\frac{3.67H}{D}}\right] H \qquad \text{(Eq. 4.5.23)}$$

$X_c = 15.45$ m

Height from the tank bottom to centre of gravity of roof in m;(X_r) = 20.05 m.

Height from the tank shell bottom to centre of gravity of shell in m;(X_s)= 10 m

Design Parameters

Seismic Use Group = III

Zone V = 0.36

Importance Factor (I) = 1.5

Response Modification Factor for Self Anchored Tanks:

For Impulsive Mode (R_{wi})= 3.5

For Convective Mode (R_{wc})= 2.0

Scaling Factor (Q) = 1 (ASCE-07 methods does not apply)

Acceleration based site coefficient (0.2 sec) (F_a)= 0.9

Velocity Based Site Coefficient (F_a) = 1.52

Coefficient to adjust acceleration from 5% to 0.5% damping (K) = 1.5

Regional dependent transition period for longer period ground motions (T_L) = 4sec (Region outside ASCE-07).

Maximum considered earthquake spectral response acceleration parameter at shorter time periods of 0.2 sec for 5% damping, %g (S_s)= 0.9

Maximum considered earthquake spectral response acceleration parameter for time period of 1 sec for 5% damping, %g (S_1)= 0.49

Design spectral response acceleration parameter at shorter periods (0.2 sec) for 5% damping, %g (S_{DS}) = QF_aS_s = 0.81

Design spectral response acceleration parameter at period of 1 sec for damping of 5%, %g (S_{DS}) = QF_vS_1 = 0.74

Mapped,maximum considered earthquake spectral response acceleration parameter at period of 0 sec for 5% damping, %g (S_s)= 0.36

Design Response Accelerations

(i) Spectral Acceleration for Impulsive Mode (A_i)

$$A_i = 2.5QF_aS_0\left(\frac{I}{R_{wi}}\right) = 0.347 \qquad \text{(Eq. 4.5.24)}$$

(ii) Spectral Acceleration for Convective Mode (A_c)

When $T_c > T_L$

$$A_c = 2.5KQF_aS_0\left(\frac{T_sT_L}{T_c^2}\right)\left(\frac{I}{R_{wc}}\right) = 0.072 \qquad \text{(Eq. 4.5.25)}$$

Base Shear Calculation

The base shear for seismic is calculated as per Square Root of Sum of Squares (SRSS) method for impulsive and convective components separately.

$V_i = A_i(W_s + W_f + W_r + W_i)$ = 32272.99kN

$V_c = A_c(W_c)$= 5103.35 kN

$$V = \sqrt{V_i^2 + V_c^2} \qquad \text{(Eq. 4.5.26)}$$

V = 32673.97 kN

Ringwall Overturning Moment

The seismic overturning moment at the bottom of the tank shell shall be the SRSS summation of impulsive and convective component multiplied by their moment arms to the centre of action of forces.It is used to determine the shell compression, anchorage forces and load on the foundation.

$$M_{rw} = \sqrt{[A_i(W_iX_i + W_rX_r + W_sX_s)]^2 + [A_c(W_cX_c)]^2} \qquad \text{(Eq. 4.5.27)}$$

M_{rw}= 252.73 MN.m

Slab Overturning Moment

This overturning moment is important for the designing anchorage and also to determine the number and size of the anchor bolt required for the storage tank. It is used while designing foundation for the storage tank.

$$M_s = \sqrt{[A_i(W_{is}X_i + W_rX_r + W_sX_s)]^2 + [A_c(W_cX_{cs})]^2} \qquad \text{(Eq. 4.5.28)}$$

M_s= 509.35 MN.m

4.5.2.2 Dynamic Liquid Hoop Forces at the Base

If the ratio of D/H greater than equal to 1.333 the impulsive dynamic hoop force is given by the following equation:

$$N_i = 8.48A_iGDH\left[\frac{Y}{H} - 0.5\left(\frac{Y}{H}\right)^2\right]\tanh\left(0.866\frac{D}{H}\right) = 829.17N/mm \qquad \text{(Eq. 4.5.29)}$$

For convective mode the dynamic hoop force is given by:

$$N_c = \frac{1.85A_cGD^2\cosh\left[\dfrac{3.68(H-Y)}{D}\right]}{\cosh\left[\dfrac{3.68H}{D}\right]} = 49.64N/mm \qquad \text{(Eq. 4.5.30)}$$

Horizontal hoop stress is given by:

$$N_h = 4.9DG(H - 0.3) = 2896N/mm \qquad \text{(Eq. 4.5.31)}$$

Total combined hoop stress in the shell is:

$$\sigma_T = \sigma_h \pm \sigma_s \quad \text{(Eq. 4.5.32)}$$

$$\sigma_T = \frac{N_h \pm \sqrt{N_i^2 + N_c^2}}{t} \quad \text{(Eq. 4.5.33)}$$

$\sigma_T < 0.9F_yE$ or $\sigma_T < 1.33S_d$

Here the total combined hoop stress is found to be 134 N/mm^2 which is less than the maximum allowable hoop stress that is 202.72 N/mm^2 and 218 N/mm^2 obtained from equation

4.5.2.3 Freeboard Requirement

The height of sloshing wave above product design height can be estimated by:

$$\delta_s = 0.42DA_f = 1.6m \quad \text{(Eq. 4.5.34)}$$

For SUG III,

$T_c > T_L$

$A_f = 2.5KQF_aS_0\left(\frac{T_sT_L}{T_C^2}\right) = 0.095$

4.6 Resistance to Overturning

As per API 650 standard the overturning moment due to seismic is resisted by the following three component:

(i) Anchorage Requirement.

(ii) Annular Plate Width.

(iii) Shell Compression at the Bottom

Anchorage Requirement

The overturning moment (ringwall moment) acting at the shell base is resisted by:

- The self-weight of the shell plates, weight of the roof and the weight of the product inside the tank for unanchored tanks.

- Mechanical anchorages.

Anchorage ratio (J) is used to determine the anchorage requirement as per API 650 standard.Anchorage ratio criteria are discussed in the table 4.1 will use to determine tank is self anchored or mechanically anchored [1].

Table 4.1: Anchorage Ratio Criteria

Anchorage Ratio (J)	**Criteria**
J≤ 0.785	No uplift for design overturning moment
0.785<J≤ 1.54	Tank is uplifting but the tank is stable for the design load providing the shell compression. Tank is Self anchored
J>1.54	Tank is not stable. Modify the annular ring if L<0.035D is not controlling or add mechanical devices.

The anchorage ratio J as per API 650 is[1] :

$$J = \frac{M_{rw}}{D^2[w_t(1-0.4A_v)+w_a-0.4w_{int}]} \qquad \text{(Eq. 4.6.1)}$$

Where

w_t=Weight of the tank shell and roof supported by the shell in N/m

$$w_t = \left[\frac{W_s}{\pi D} + w_{rs}\right]$$

$$w_t = \left[\frac{3198000}{3.14 \times 40} + 5175\right] = 30636.7 \text{ N/m}$$

$$w_a = 99t_a\sqrt{F_y H G_e} \leq 201.1HDG_e$$

A_v=Vertical seismic acceleration parameter

A_v= 0.47 S_{DS}= 0.38

M_{rw}=Ring-Wall Moment at the base = 252.73 MN.m

w_{rs} =Roof load acting on the shell including 10% snow load in N/m

w_a =Resisting force of the annular ring in N/m = 91964.8 N/m

F_y=Minimum yield strength of bottom annular plate in MPa = 265 MPa

t_a=Bottom annular plate thickness in mm = 16 mm

H = Height of the tank in m = 20 m

D = Diameter of the tank in m = 40 m

G_e= Effective specific gravity including vertical seismic effect

G_e= G(1-0.4A_v)=0.636

The anchorage ratio J is found to be 1.33 which is less than 1.54 so as per the anchorage criteria as shown in the Table4.1 tank is stable for the design load and can be self anchored.

Annular Bottom Plate Requirement

Before going for the anchor bolt design width of the annular plate should be checked for the stability against seismic load.

For the tanks located in SUG III, butt-welded annular plates shall be required. Annular plate exceeding 10 mm thickness should be butt-welded.

The annular plate thickness t_a should not exceed the thickness of the bottom shell course less corrosion allowances.

When the bottom plate located below the shell plate is thicker than the remaining tank bottom then the minimum projection L of the annular bottom plate inside the tank wall shall be greater of 450mm or determined by following equation. L should not be greater than 0.0035D.Provided thickness of annular bottom plate t_a is 16 mm

$$L = 0.01723 t_a \sqrt{\frac{F_y}{HG_e}}$$

L = 1260 mm $\leq$ 1400mm

Shell Compression

Maximum shell compression at shell bottom in a self anchored storage tanks having anchorage ratio grater than 0.785 is given by:

$$\sigma_c = \left(\frac{w_t\left(1 + 0.4A_v\right) + w_a}{0.607 - 0.18667[J]^{2.3}} - w_a \right) \frac{1}{1000 t_s} \qquad \text{(Eq. 4.6.2)}$$

The maximum compression of shell should be less than the seismic allowable stress F_C which is determined as follow:

When $\dfrac{GHD^2}{t^2}$ = 30.65 <44 then,

$$F_c = \frac{83 t_s}{2.5D} + 7.5\sqrt{GH}$$

$$F_c < 0.5 F_{yt}$$

The maximum longitudinal shell compression σ_c is calculated to be 16 N/mm^2.and F_c is found to be 52.28 N/mm^2which is less than 0.5 times the specific yield stress of the bottom shell.Hence the tank is structurally stable.

4.7 Erection Plan

Erection plan of the tank has been developed after having the full knowledge of the site condition, availability of material and equipment.

Establishing Orientation:

Before start of the construction of the tank orientation of the tank pad and location of fittings of the tank should be established.

Inspection of Foundation:

Foundation of the tank should be in level to maintain the smooth roundness of the shell and to avoid the settlement.

Locating Centre:

Before laying of the bottom plate the centre of the tank must be located:

1. Diameter of the tank is measured at 4 different places.
2. Average the diameter and calculate radius out of it.
3. Now hold the tape at one end on the outer edge of the grade and mark an arc across the centre grade.
4. Repeat the step 3 at other three locations which are approximately at 90 degrees.
5. Mark the stake at a point where all these 4 arcs cross each other.
6. After the location of the centre measure the radius of the tank to check weather the centre is located correctly or not.

Installation of Draw off Sump

Before laying of bottom plates place the sump in the foundation and check the location and elevation.Compact thoroughly around the sump.

Laying of Bottom Plates

1. As per the erection drawing start the laying of annular plates and weld annular to annular plate joint.
2. Then laying of bottom plate will start from the centre to the outward direction.
3. All the bottom plates and sketch plates are laid and tacked on the shorter seams.
4. Suitable jig and fixtures are used to hold the sketch plates with bottom plates.
5. Now check all the bottom plates are fully locked or not and after that tacked the long seams.
6. Start the welding of T joints. When all the T joints are welded start the full welding of short seams followed by full welding of long seams from centre to outwards.
7. At the last sketch plates are welded to the annular plate.

Shell Erection

1. Start the erection of top most shell course in jig plates and spacers.
2. Tack the 1st vertical seams weld strongly.
3. Install the jacking system and necessary equipment.
4. Weld the top wind girder and curbe angle to the top shell at desired location.
5. Start jacking up the shell and install the next shell course.
6. Weld all the vertical and horizontal seams of that course.
7. Repeat the procedure until all the shell course are installed.
8. Now complete shell to bottom plate joint.

Laying of Roof Plates

1. As per the erection drawing after the laying of column , girders and rafters the laying of roof plates begins from centre to outward direction.

2. Overlap each roof plate over the last one laid and check the distance from the center line to the edge of each course.

3. To make the necessary adjustment tack all the plates lightly.

4. Weld the sketch plate to the curb angle before welding to all other sketch plates.

5. Now complete the full welding of all the seams.

4.8 Summary

The larger diameter liquid storage tank of 22 million liters capacity is analyzed through IS:1893(Part2) and API 650 guidelines for the seismic design using their respective design philosophy. Seismic parameters: Time period - impulsive and convective, Spectral acceleration, Base Shear, Base Moment and Overturning Moment are evaluated through both design guidelines.Stability check against seismic loads has been performed and tank is found to be stable.Erection plan of various component of the liquid storage tank has been discussed.

Chapter 5

Finite Element Analysis of Liquid Storage Tank

5.1 General

Analysis and design of liquid storage tank is utmost important due to its hazardous nature under seismic condition. To carry out finite element analysis a computational model of the liquid storage tank is prepared in a licensed commercial software SAP2000.Finite element analysis is carried out under the static conditions for the tank.A tank of 22 MLD capacity located in seismic zone–v as per Indian seismic code IS:1893, is considered for the present study. The 4 noded shell element is used for modeling of these problems.Comparison of stresses and thickness of shell plate by finite element analysis results of liquid storage tank with manual calculations has been done.

5.2 F.E Modeling and Analysis of the Tank

A tank of 22 MLD capacity has been modeled in a software SAP2000 to perform the finite element analysis.The 4 node shell element with six degrees of freedom at each node, three translations and three rotations is choosen to modeled the tank. The analysis is carried out by using full shell behaviour of 4 noded shell element. Tank is analysed by with full shell behaviour means it utilize both membrane bending behaviour. The results obtained from finite element analysis are compared with manually calculated results.The hydrostatic pressure applied on the shell elements.

5.2.1 Problem Description

A above ground cylindrical liquid storage tank storing petrol is modeled in a software. Diameter of the tank is 40 m , height of the tank is 20 m and the thickness of the plate is 28mm as shown in the Figure 5.1.The thickness of the bottom plate provided is 10mm. The density of liquid stored is 753 kg/m^3.Material used for the shell model is steel having poison ratio 0.3 and minimum yield stress of 265 N/m^2.For the FEA modelling of the tank tank is divided into number of shell element segment. Tank is divided into 30 number of divisions radially and 40 number of divisions height wise as shown in the figure. The circular cylindrical tank analysed for static condition with fixed base.

Steps for Modeling and Analysis of the Tank

1. Modeling of the tank as 4 noded shell element with specified dimensions has been done as shown in the Figure 5.1.

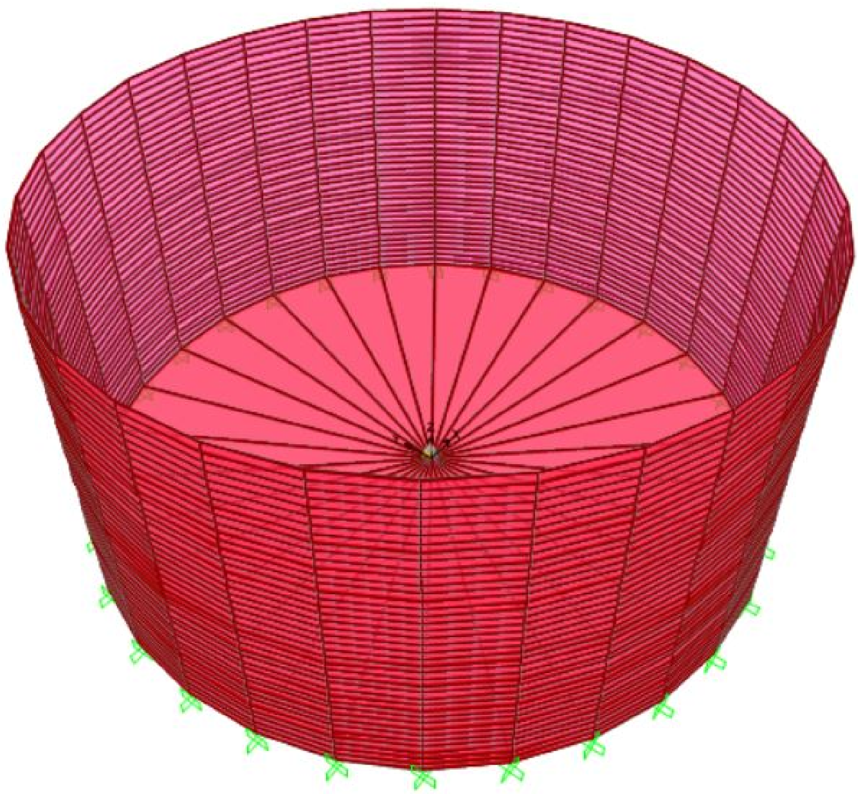

Figure 5.1: FEA model of the Liquid Storage Tank

2. Material and section properties for each member are defined as shown in Figure 5.2.
3. Constrains are provided to the model.Tank is considered fixed at the base.
4. Load patterns and load combination are defined.As the static analysis is done firstly hence only dead load and hydrostatic load are considered.

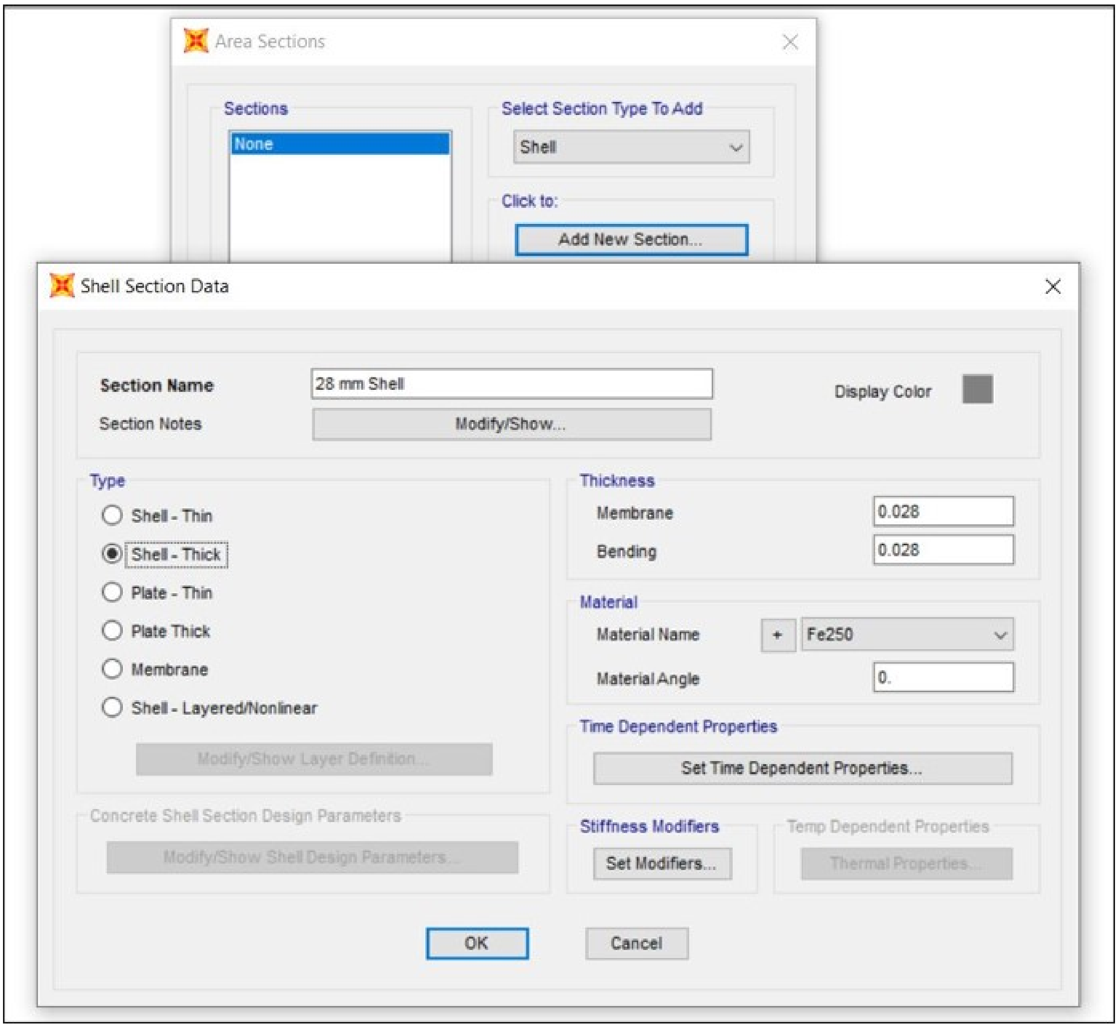

Figure 5.2: Material Properties of the Tank

5. Joint patterns are defined and assigned to the tank as shown in Figure 5.3 and Figure 5.4 respectively.Calculation for the joint pattern and hydrostatic load is done as follows:

6. Hydrostatic load is assigned as **Assign >Area Loads >Surface Pressure >Hydrostatic Pattern** in SAP 2000 .

7. Model is set to run and results are obtained.

In SAP 2000 Pressure Pattern Value (P) needs to be calculated for the definition of joint pattern.Where,
(P) = Ax + By + Cz + D
Here A ,B , C and D are the constants.
In this case hydrostatic load is interpolated along the Z axis and hence the value of x and y will be zero.
Hydrostatic Pressure
P = Ax + By + Cz + D
P = Cz + D
At z = 0,
P = $\rho \times g \times h$ = 147.7 kN/m^2
D = 147.70

At z = 20 m,
P = 0 kN/m^2
0 = C (20) + 147.70
C = - 7.385
The value of the constant obtained from the above calculation are assigned to the software as shown in the Figure 5.4.

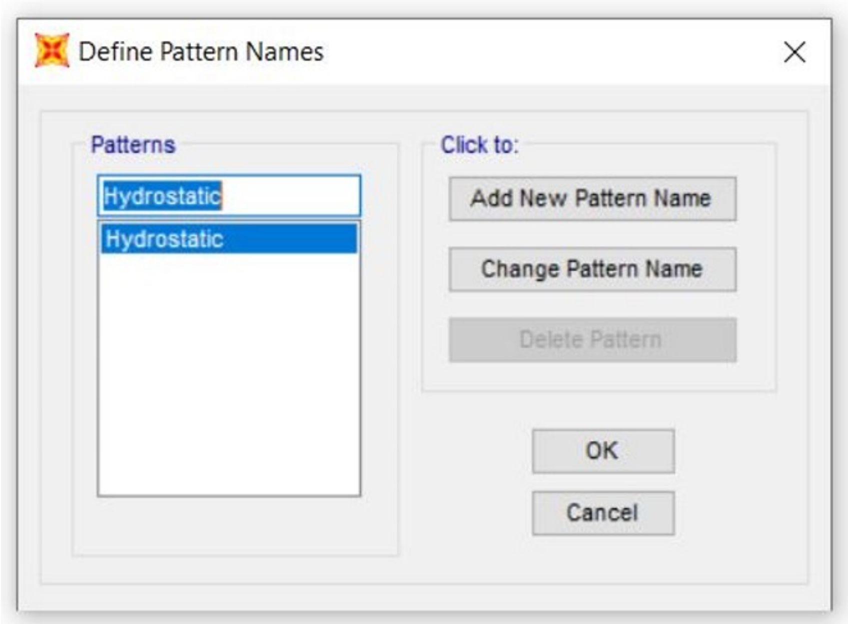

Figure 5.3: Joint Pattern Definition

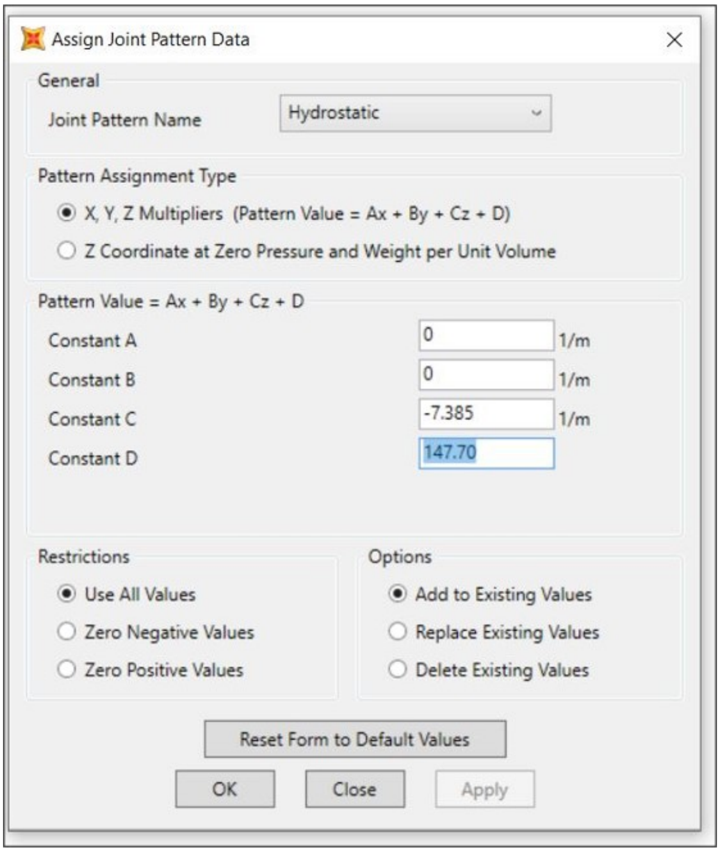

Figure 5.4: Assignment of Joint Pattern to the Tank Model

5.3 Results and Discussion

Here the Finite element analysis of the tank is carried out under the static conditions. And it shows a good agreement in terms of variation of stress in the shell and thickness of the shell is when the results of FEA and manual calculations are compared.

Table 5.1: Comparison of Shell Plate Thickness by FEA Results and Manual Calculations

Sr No	Shell Course	Design Thickness Provided in SAP	Design Thickness Calculated Manually
1	ShellCourse 10	28	28
2	ShellCourse 9	28	25
3	ShellCourse 8	25	22
4	ShellCourse 7	22	20
5	ShellCourse 6	20	18
6	ShellCourse 5	15	15
7	ShellCourse 4	15	12
8	ShellCourse 3	15	10
9	ShellCourse 2	10	8
10	ShellCourse 1	10	8

One more study of the storage tank is also carried out by changing the thickness of the shell plates in FEA model.It was observed from the table 5.2 that as the thickness of the plates get increased the value of maximum stress in it get reduced and it comes with in the design limit when the provided thickness is 28 mm. The thickness of the plates get reduces as the height of the tank increases and stress gets decrease.

Table 5.2: Value of Maximum Stress for Applied Thickness

Shell Thickness (mm)	**Stress (Mpa)**
8	459.7
10	358.47
12	290.65
15	203.3
18	178.83
20	165.12
22	157.7
25	143.2
28	138.57

5.4 Summary

This chapter discuss about the modeling of a liquid storage tank having 22 MLD capacity in a commercial software SAP2000. Finite element analysis of the tank has been performed and stress results are obtained.How to apply the hydrostatic load in SAP2000 software is also discussed.

Chapter 6

Summary, Conclusion and Future Scope of the Work

6.1 Summary

Analysis and design of liquid storage tank plays an important role due to its hazardous nature. Study of the various national and international seismic code for the seismic analysis and design of liquid storage tank has been carried out. In the present study above ground cylindrical steel tanks of 22 million litres capacity located in highest seismic zone, is analysed and designed using API 650 guidelines.

Firstly, the gravity load design of large diameter liquid storage tank has been carried out following API 650 standard. Analysis and design of structurally supported cone roof of storage tank has been performed using Stadd Pro software by following IS 803 and API 650 specification. Wind analysis of the storage tank is performed using API 650 guidelines. In second part the same liquid storage tank having large diameter is analysed through IS:1893(Part2):2014 and API 650 guidelines for the seismic design using their respective design philosophy. Various seismic parameters such as time period, base shear and bending moment are evaluated. The finite element analysis of the tank has been performed using SAP 2000 software. Erection drawing of various component of tank has also been prepared along with the erection plan.

6.2 Conclusions

- The maximum calculated thickness of bottom shell course as per one-foot method is 28mm and it gets reduces as the stress gets reduce along with increase in height. The thickness finalized for the bottom plates and annular plates are 10 mm and 16 mm respectively using API 650 provisions.
- It has been found that the IS code based seismic analysis is 7-8% conservative as compared to Annex E of API 650 standard.
- The anchorage ratio of the considered tank as per API 650 provision is 1.33 and is found to be stable under seismic loads.
- The tank also satisfies the wind criteria of the API code and found to be stable against wind load and hence no mechanical devices are required to anchor the tank.
- The FEM analysis of the tank has been performed using 4 node shell elements in a FEA based software i.e. SAP 2000 and the design stresses of the plate are found to be within the limit for the provided plate thickness.
- To study the behaviour of the tank a free vibration analysis of FEM model has been done in SAP 2000 software

6.3 Future Scope of the Work

- Parametric study related to variety of liquid which can be stored inside tank be worked upon.
- The analysis of the tank can also be performed using soil structure interaction.
- Linear time history seismic analysis may be conducted using commercial software to observe critical behaviour of liquid storage tank.
- In present study the type of roof used is fixed cone roof but for future work the design of floating roof can also be performed.

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Appendix A

Sesmic Analysis of Tank as per IS Code

Seismic Analysis			
Design Code		IS 1893(Part 2):2014	
			Unit
Volume of the Liquid to be Stored	**V**	22000	m3
Tank Shape		Cylindrical or Circular	
Diameter of the tank	**D**	40	m
Nominal Diameter of the tank	**D**	40.028	
Radius of the Tank	**r**	20	m
Height of the tank	**H**	20	m
Density of Steel Plates	**steel**	78.53	kN/m3
Liquid to be Stored		Water	
Density of Stored Liquid		753	kg/m3
Specific Gravity of Stored Liquid	**G**	0.75	

Thickness of Steel Shell Plates	**t**	28	mm
Thickness of Steel Shell Plates	**t**	0.028	m
Thickness of Bottom Plates	**tb**	0.01	m
Modulus of Elasticity	**E**	2E+11	N/m2
Gravitational Acceleration	**g**	9.81	m/s2
Weight Calculation			
(i) Weight of the Tank Wall			
D t H		3198	kN
(a) Mass of the tank wall	**mw**	325991	kg
(ii) Weight of the Base Plate			
r2 t		986	kN
(b) Mass of the Base Plate	**mb**	100544	kg
(iii) Weight of the Liquid		162512.46	kN
(c) Mass of the Liquid	**m**	16566000	kg
Assuming roof plate of (5mm)		0.005	m
(iv) Weight of the roof plate		650	kN
(d) Mass of the Roof Plate	**mt**	66259	kg

Parameters of Spring Mass Model			
Height of the Tank	**h**	20	m
Diameter of the Tank	**D**	40	m
For h/D		0.5	
For D/h			
mi/m		0.54	
mc/m		0.44	
hi/h		0.375	
hc/h		0.603	
hi*/h		0.7970	
hc*/h		0.7822	
Impulsive Mass	**mi**	8984011.887	kg
Convective Mass	**mc**	7245401.277	kg
Height of the Impulsive Mass (Without Base Pressure)	**hi**	7.5	m
Height of the Convective Mass (Without Base Pressure)	**hc**	12.06624087	m

Height of the Impulsive Mass (With Base Pressure)	**hi***	15.93942351	m
Height of the Convective Mass (With Base Pressure)	**hc***	15.64354951	m
Height of Centre of Gravity of Wall Mass	**hw**	10	m
Height of Centre of Gravity of Roof Mass above tank Wall	**ht**	20.0025	m
Time Period Calculation			
Coefficeint of Impulsive Mode	**Ci**	4.33	
Coefficeint of Convective Mode	**Cc**	3.36	
(a) Impulsive Time Period	**Ti**	0.20	sec
(b) Convective Time Period	**Tc**	6.78	sec

Design Seismic Horizontal Coefficent			
Zone Factor (Z)	**5**	0.36	
Importance Factor	**I**	1.5	For Petroleum Products
Response Reduction Factor	**R**	2.5	For Unananchored Tank Base
Average Response Acceleration Coff	**Sa/g**		
Impulsive Mode		2.5	
Convective Mode		0.34	
Damping			Multiplying Factor
Impulsive Mode		2%	1.4
		5%	1
Convective Mode		0.50%	1.75
(a) For Impulsive Mode	**(Ah)i**	0.378	
(b) For Convective Mode	**(Ah)c**	0.064	
Base Shear Calculation			
(a) Impulsive Base Shear	**Vi**	34768.865	kN
(b) Convective Base Shear	**Vc**	4567.433	kN
Total Base Shear	**V**	35067.584	kN

Base Moment Calculation			
(a) Impulsive Base Moment	**Mi**	266860.43	kN.m
(b) Convective Base Moment	**Mc**	55111.75	kN.m
Total Moment	**M**	272.49	MN.m
Overturning Moment Calculation			
(a) Impulsive Overturn Moment	**Mi***	548.3637528	MN.m
(b) Convective Overturn Moment	**Mc***	71.49653638	MN.m
Total Overturning Moment	**M***	553.0050272	MN.m
Hydrodynamic Pressure Calculation			
(a) Impulsive Hydrodynamic Pressure			
Lateral Hydrodynamic Pressure	**Piw(y)**	45.43	kN/m2
Vertical Distance of a Point or Tank Wall from the Bottom of Tank	**y**	0	m
Maximum Pressure will Occure at	**Ø**	0	
Constant	**Qiw**	0.813	

(b) Convective Hydrodynamic Pressure			
Lateral Pressure	**Pcw**	2.21	kN/m2
Constant	**Qcw**	0.175	
(c) Pressure due ro Wall Inertia	**Pww**	8.153694511	kN/m2
(d) Effect of Vertical Ground Acceleration	**Pv**	37.2301272	kN/m2
Vertical Acc Coefficent	**Av**	0.252	
Maximum Hydrodynamic Pressure	**P**	65.282	kN/m2
Anchorage Requirement			
H/D		0.5	
1/(Ah)i		2.65	
Anchorge Required		No	
Sloshing Wave Height			
(a) For Circular Tank	**dmax**	3.21	m

Appendix B

Seismic Analysis as per API 650

Sesismic Design by API 650						
			Unit			
Volume of the Liquid to be Stored	**V**	22000	m3			
Location		Kutch				
Tank Shape		Cylindrical or Circular				
Diameter of the tank	**D**	40	m			
Nominal Diameter of the tank	**D**	40.028				
Radius of the Tank	**r**	20	m			
Height of the tank	**H**	20	m			
Density of Steel Plates	**steel**	78.53	kN/m3			
Liquid to be Stored		Petrol				
Density of Stored Liquid		753	kg/m3			
Specific Gravity of Stored Liquid	**G**	0.75				
Thickness of Steel Shell Plates	**t**	28	mm			
Thickness of Steel Shell Plates	**t**	0.028	m			

Thickness of Bottom Plates	**tb**	0.01	m			
Modulus of Elasticity	**E**	200000	N/m2			
Gravitational Acceleration	**g**	9.81	m/s2			
Weight Calculation						
(i) Weight of the Tank Wall		3197.97	kN			
		3197968.24	N			
Shell Course (Bottom to Top)	**Height of Each Shell Course (mm)**	**Height (mm)**	**tmin (mm)**	**tmax (mm)**	**Weight**	
Shell Course 10	**2000**	20000	8	26	513.25	
Shell Course 9	**2000**	18000	8	24	473.77	
Shell Course 8	**2000**	16000	8	22	434.29	
Shell Course 7	**2000**	14000	8	20	394.81	
Shell Course 6	**2000**	12000	8	18	355.33	
Shell Course 5	**2000**	10000	8	14	276.37	
Shell Course 4	**2000**	8000	8	12	236.89	
Shell Course 3	**2000**	6000	8	10	197.41	
Shell Course 2	**2000**	4000	8	8	157.92	
Shell Course 1	**2000**	2000	8	8	157.92	
					3197.97	kN

(i) Weight of the Tank Wall	**Ws**	3197968.244	N	3197.97	KN	
D t H						
(ii) Weight of the Base Plate	**Wf**	986337	N	986.337	KN	
r2 t						
(iii) Weight of the Liquid	**Wp**	162512460	N	162512	KN	
Assuming roof plate of (10mm)						
(iv) Weight of the roof plate	**Wr**	650000	N	650	KN	
Earthquake Parameters						
Mapped, MCE 5% damped spectral response acceleration parameter at short periods (0.2 sec), %g	**Ss**	0.9				
Mapped, MCE 5% damped spectral response acceleration parameter at period of 1 sec, %g	**S1**	0.49				

Scaling Factor from MCE to the design level spectral acceleration	**Q**	1				
Acceleration based site coefficient (0.2 sec)	**Fa**	0.9				
Velocity based site coefficient (1 sec)	**Fv**	1.52				
Design 5% damped spectral response acceleration parameter at short periods (0.2 sec), %g	**SDS**	0.81				
Design 5% damped spectral response acceleration parameter at period of 1 sec, %g	**SD1**	0.7448				
Mapped, MCE 5% damped spectral response acceleration parameter at periods (0 sec), %g	**S0**	0.36				
Time Period Calculation						

Coefficeint of Impulsive Mode	**Ci**	6.35				
Coefficeint of Convective Mode	**Ks**	0.6				
(a) Impulsive Time Period	**Ti**	0.21	sec			
(b) Convective Time Period	**Tc**	6.83	sec			
Regional dependent transition period for longer period ground motions	**TL**	4	sec			
	T0	0.183	sec			
	Ts	0.91	sec			
Coefficient to adjust the spectral accelerationn from 5% to 0.5	**K**	1.5				
Spectral Acceleration Coefficient						
Zone Factor (Z)	**5**	0.36				

Importance Factor (I)	**III**	1.5				
Response Reduction Factor (R)	**Self-anchored**					
a) Impulsive	**Rwi**	3.5				
b) Convective	**Rwc**	2				
(a) For Impulsive Mode	**(Ah)i**	0.347	% g			
(b) For Convective Mode	**(Ah)c**	0.072	% g			
Parameters of Spring-Mass Model						
a) Impulsive Weight	**Wi**	88133156.61	N			
b) Convective Weight	**Wc**	71041336.96	N			
	Total	159174493.6	N	ok		
Centre of Action for Ringwall Overturning Moment						
a) Height of the Impulsive Mass	**Xi**	7.5	m			

b) Height of Convective Mass	**Xc**	12.10	m			
	0.60506					
Centre of Action for Slab Overturning Moment						
a) Height of the Impulsive Mass	**Xis**	15.937	m			
b) Height of Convective Mass	**Xcs**	15.45	m			
	0.77232					
c) Height from the bottom of the tank shell to roof and roof appurtenances centre of gravity	**Xr**	20.005	m			
d) Height from the bottom of the tank shell to shell centre of gravity	**Xs**	10	m			
Base Shear Calculation						
(a) Impulsive Base Shear	**Vi**	32272990.26	N			
(b) Convective Base Shear	**Vc**	5103355.302	N			
Total Base Shear	**V**	32673997.85	N			
		32673.99785	KN			

Overturning Moment Calculation						
a) Ringwall moment	**Mrw**	252.737781	MN .m			
Impulsive Moment	**Mrwi**	245.0764722	MN .m			
Convective Moment	**Mrwc**	61.75685196	MN .m			
b) Slab Moment	**Ms**	509.3513092	MN .m			
	Msi	503.2143604	MN .m			
	Msc	78.82933206	MN .m			
Resistance to Design Loads						
a) Anchorage Ratio	**J**	1.339				
Bottom annular plate thickness	**ta**	16	mm			
				Ok		
Thickness of bottom shell course	**ts**	26				
Bottom Annular Radial Width	**Ls**	960.24	mm	1400	mm	Ok
Minimum Specified Yield Strength of Bottom Ring	**Fy**	265	MPa			

Ring Wall Overturning Moment	**Mrw**	252737781	N			
Vertical Acceleration Coefficient	**Av**	0.38				
Tank and Roof Weight acting at Shell Base	**Wt**	30.63668984	N/mm	30636.7	N/m	
Resisting Force of Annular Ring	**wa**	91.9648083	N/mm	102.32	N/mm	Ok
Height of the tank	**H**	20	mm	91964.8	N/m	
Diameter of the Tank	**D**	40000	mm			
	wrs	5.175159236	N/mm	5175.16	N/m	
Effective Specific Gravity including Vertical Acceleration	**Ge**	0.636				
Minimum width of the butt weld anular ring to be provided	**L**	1258.301289	mm			