ANALYSIS AND DESIGN OF SHAFT TYPE WATER TOWERS USING FINITE ELEMENT METHOD AND DRAFT CODE PROVISIONS

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CERTIFICATE

This is to certify that the Major Project entitled "**Analysis and Design of Shaft Type Water Towers Using Finite Element Method and Draft Code Provisions**" submitted by **Mr. Mishra Pritesh R. (04MCL006)**, towards the partial fulfillment of the requirements for the degree of **Master of Technology in Civil Engineering** (**Computer Aided Structural Analysis and Design**) of Nirma University of Science and Technology, Ahmedabad is the record of work carried out by him under my supervision and guidance. In my opinion, the submitted work has reached a level required for being accepted for examination. The results embodied in this major project, to the best of my knowledge, haven't been submitted to any other university or institution for award of any degree or diploma.

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ABSTRACT

Overhead water tanks or elevated service reservoirs are one of the most important components of any efficient water distribution system. The basic purpose of elevated water tanks is to secure constant water supply. There post earthquake functionality makes these structures even more important. There vulnerability to severe earthquakes due to their configuration, involving large mass concentrated at the top of relatively slender staging, is a major concern for all the designers.

In the past earthquakes some of the shaft type of water tanks has shown some distress and a very few has collapsed. The poor performance of such structures should be studied considering various aspects like design considerations, construction quality, quality of post construction maintenance etc. In the proposed draft code the level of forces has been increased to bring it to the level of other international codes and to compensate for the poor performance of shafts. Analysis and design of the supporting structure and foundation of a case study has been carried out by IS: 1893-1984 and Proposed draft for IS: 1893 (Part II) and comparison is made for the design forces and quantities of physical quantities.

When thickness of a shaft is very small compared to its diameter, the shaft behaves as a membrane structure. For access, for maintenance, air ventilation and for the inlet and outlet pipes of water, openings are frequently required in shafts. To minimize the effects of such openings and to have smooth flow of stresses around the openings the opening sizes should be as small as possible. This helps in maintaining constant shell thickness of shaft over the height of openings. Even after all such measures, stress concentration on sides of the openings and stress release above and below the openings are bound to occur. Here an effort has been made to study and understand the effects of stress concentration around such openings on shafts. Further the effects of stress release on foundation of shaft especially due to the inlet and outlet openings located very near to the foundation has been studied.

The most important aspect in the construction of a shaft is to maintain verticality, circularity and uniformity of thickness. Any variation in geometry would result into additional stresses. Generally the workmanship is of good quality, many times (which may not be always the case), some construction error is bound to occur. As a good design engineer it is a good practice to consider effect of such errors by way of minimum tolerances and include them in the design itself. But the Indian Standard code has been silent about these tolerances in the shafts of elevated water tanks. An effort has been made to study the effects of tolerances in the shafts, especially the tolerances in plumb ness of the shafts. Further the addition stresses due to such tolerances has been worked out and probable boundary for such tolerances have been suggested to restrict the additional stresses on the shafts due to poor workmanship.

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NOMENCLATURE

A	=	Cross Sectional area
A_h	=	Design horizontal seismic coefficient
$(A_h)_c$	=	Design horizontal seismic coefficient in convective mode
$(A_h)_i$	=	Design horizontal seismic coefficient in impulsive mode
a	=	Lever arm
C_c	=	Coefficient of time period in convective mode
C_i	=	Coefficient of time period in impulsive mode
d_{max}	=	Maximum sloshing wave height
D_1	=	Inside Diameter of water tank
D_2	=	Inside Diameter of shaft
DL	=	Dead load
d	=	Effective depth
E_c	=	Modulus of elasticity of concrete
E_s	=	Modulus of elasticity of steel
e	=	Eccentricity
f_{ck}	=	Characteristic compressive strength of concrete
f_y	=	Characteristic strength of steel
h_c	=	Height of convective mass above bottom of tank (neglecting base pressure)
h_i	=	Height of convective mass above bottom of tank (neglecting base pressure)
h_s	=	Structural height of staging measured from top of the footing to the bottom of container
h^*_c	=	Height of convective mass above bottom of tank considering base pressure
h^*_i	=	Height of impulsive mass above bottom of tank considering base pressure
h_{cg}	=	Height of center of gravity of empty container of elevated tank measured from base of staging
I	=	Moment of Inertia of section
K	=	Stiffness of member
LL	=	Live load
M	=	Bending moment

m = Modular ratio

m_c = Convective mass of liquid

m_i = Impulsive mass of liquid

m_s = Structural mass i.e. mass of container of elevated tank and one third mass of staging

M^* = Overturning moment at the base

M^*_c = Overturning moment in convective mode at the base

M^*_i = Overturning moment in impulsive mode at the base

M_r = Radial moment

M_t = Tangential moment

P = Axial load on a member

r = Radius

R = Response reduction factor

T_c = Time period in convective mode

T_i = Time period in impulsive mode

S_a/g = Average response acceleration coefficient

V = Total base shear or shear force

w = distributed load per unit area

W = Total vertical or concentrated load

Z = Seismic zone factor

σ_{cb} = Permissible stress in concrete in bending compression

σ_{cc} = Permissible stress in concrete in direct compression

σ_{sc} = Permissible stress in steel in compression

σ_{st} = Permissible stress in steel in tension

τ_c = Permissible shear stress in concrete

Ø = Diameter of steel bar

θ, α = Angles

β = Ratio

δ = Displacement

CHAPTER 1 INTRODUCTION

Storage reservoir is a term used for structures, designed to store water, petroleum products and similar other liquids. The structural analysis of all reservoirs is similar irrespective of the chemical nature of the product being stored. Such structures are important public utility structures more particularly in high seismic zones. For such structures, one of the main consideration, besides strength, is that they should be leak proof hence it should be ensured during design stage that concrete does not crack on the liquid face or crack width is within permissible limit. The concrete used for such structures should be well graded and well compacted, so that the tensile strength is high and the porosity is low.

Such reservoirs are very important part of drinking water distribution system. In water distribution system, water is first stored in underground sump storage reservoirs, usually two to three times the capacity of the elevated reservoir, and is chlorinated before being pumped up into reservoirs for distribution. Elevated reservoirs are used to meet demand during peak supply hours.

Reservoir is a common term used for liquid storage structures and it can be classified as:

- Underground Reservoirs
- Partial underground Reservoirs
- Reservoirs resting on ground
- Elevated Reservoirs

The purpose of the present work is to study the performance of elevated reservoirs (supported on shaft) along with analysis and design criteria for gravity loads as well as lateral (earthquake) load.

1.1 ELEVATED WATER RESERVOIRS:

Overhead water tanks or elevated service reservoirs are one of the most important components of any efficient water distribution system. The basic purpose of elevated water tanks is to secure constant water supply with sufficient flow to wide area by gravity. The height of the elevated tank depends on the area to be covered for the water supply. Wider the area to be served higher will be the required elevation of the tank.

Elevated tanks can be classified in a variety of ways.

- Classification based on shape of container.
- Classification based on supporting system.

Based on shape of the container elevated tanks can be classified as:

- Square Tank.
- Rectangle Tank.
- Circular Tank.
- Conical Tank.
- Intze Tank.

Based on supporting system elevated tanks can be classified as:

- Shaft supported Elevated Tank.
- Trestle supported Elevated Tank.

➢ **Intze Tank:**

For large capacity of the tank generally intze type of tank is preferred compared to any other shape. Intze tank can be termed as improved version of cylindrical tanks. In case of cylindrical tank when dome with small rise is used only compressive stresses are produced which helps in making the water retaining structure water tight. But in case of cylindrical tank when load on the bottom dome is heavy and its diameter is large, the ring beam becomes very heavy and needs large amount of reinforcement. In such situations the more economical option could be to reduce its diameter by introducing one additional member in form of a conical dome. Such tank with additional member in form of conical shell is known as intze tank.

- **Shaft supported Tank:**

A hollow circular shaft is the most common type of staging to support an elevated water tank. This type of staging holds good structural point of view as well as aesthetic value point of view. A hollow circular shaft is preferred for being economical (for relatively tall staging). When slip form process of casting is used, construction becomes quite rapid. The major difficulty with hollow circular shaft is to maintain verticality, circularity and uniformity of thickness. Being a stiffer structure a hollow circular shaft attracts higher seismic forces, on the other hand being stronger, lateral deformations are reduced. The height of the shaft depends on the water system requirements and site elevation, and varies from minimum of 10 m to maximum of 25 to 30 m. A rigid raft type slab foundation is a very common foundation to support shaft type staging.

FIGURE: 1.1 shows a tank of 300000 Gallon capacity located in the Lal Darwaja area of Ahmedabad. Hemispherical container is supported on a cylindrical shaft type staging. Spherical dome is used as roof of the container.

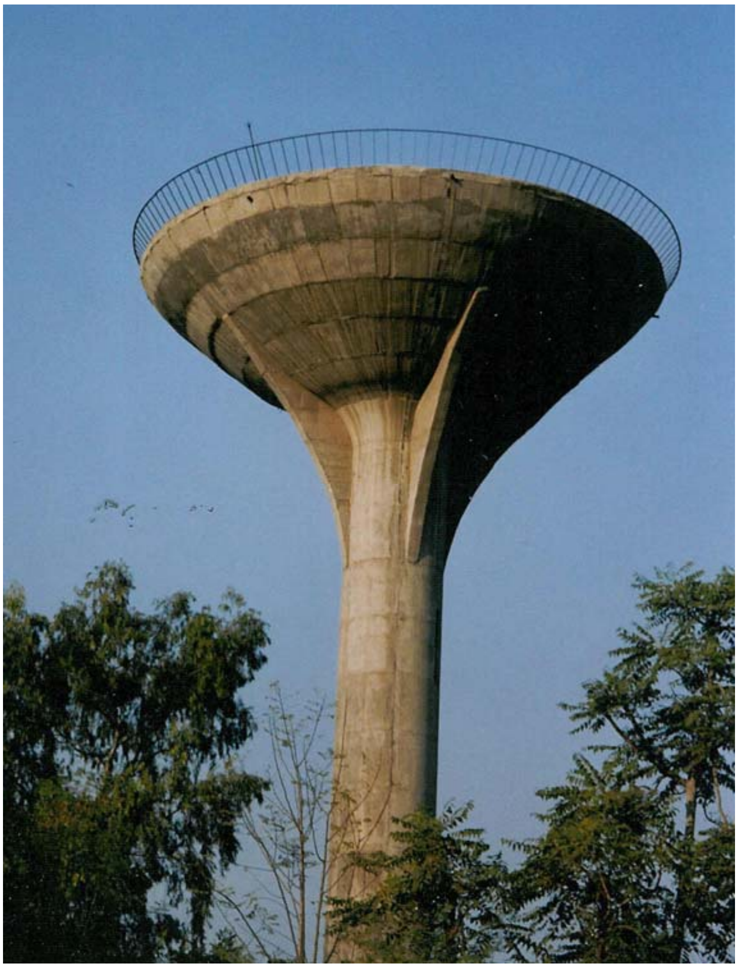
FIGURE 1.1: AT LAL DARWAJA, AHMEDABAD

HEMISPHERICAL CONTAINER SUPPORTED ON CYLINDRICAL SHAFT

FIGURE: 1.2 shows tanks having conical shaped container supported on shaft. Conical or funnel shaped over head water tanks are often preferred to other shapes mainly due to their aesthetic and superior architectural features. Conical tanks have advantage at the bottom junction where inward force will try to bring in compression and gives less tension. The disadvantage of such type of tanks is more tension in the top ring beam.

FIGURE 1.2: TANK WITH CONICAL CONTAINER SUPPORTED ON CYLINDRICAL SHAFT AT MANINAGAR, AHMEDABAD

FIGURE: 1.3 shows a semispherical container supported on cylindrical shaft. The shaft has been widened at bottom in a conical shape. Bottom of the container is usually a spherical dome which provides a better alternative than a flat slab.



SEMISPHERICAL CONTAINER SUPPORTED ON CYLINDRICAL SHAFT
FIGURE 1.3: WATER TANK AT PRL GUEST HOUSE, AHMEDABAD.

FIGURE: 1.4 shows the inside access provided to go up to container with the help of central column and tie beams connected at two levels.

FIGURE: 1.4 INSIDE VIEW OF SHAFT SUPPORTED WATER TANK

FIGURE: 1.5 shows the spiral stair case constructed outside the tank to provide access to top which is supported by tie beams and column.

FIGURE: 1.5 SPIRAL STAIRCASE OUT SIDE THE SHAFT

The openings in the shaft are the region where the stresses, due to gravity or lateral load, are high. The amount of stress concentration depends mainly on capacity of the storage reservoir and zone of earthquake or wind where the structure is located. These regions could cause vulnerability to the shafts. In the past earthquake of 2001 it was observed that shafts were damaged near such regions of stress concentration. Fig 1.6 represents the problem of stress concentration around opening and near the base.

Quality and accuracy in construction are equally important aspects of such structures. For the shaft type of staging the verticality of shaft is the most important aspect. Any disturbance in shaft at the time or placement of formwork or at the time of casting of concrete may cause some local moments and circumferential forces on the shaft. Although the effect of such problems may be localized but many times it may lead the structure, in extreme cases, to the failure.

FIGURE: 1.6: DAMAGE TO SHAFT AROUND OPENING

1.2 NEED OF STUDY:

Overhead water tanks or elevated service reservoirs are one of the most important components of any efficient water distribution system especially with their post earthquake functionality. Water is the most important requirement after air; therefore it is necessary to ensure that water supply is not interrupted after the earthquake. Further, fire is very common after an earthquake. To handle such situations also, uninterrupted water supply is very important. It may happen that because of earthquake the pumping stations have collapsed or may not remain functional (and need some retrofitting to become functional again). In such situations the elevated tanks may prove most handy tool for the purpose of water distribution and fire protection.

The basic configuration of the elevated water tank includes large mass concentrated on top of the supporting structure just like an inverted pendulum-type structure which is relatively slender and resists lateral forces by the flexural strength and stiffness of the circular hollow shaft staging. In high seismic zones lateral force design is by and large governed by seismic forces especially with tank full condition. During severe earthquakes large horizontal and overturning forces are induced in the tank, and because of that the water tank may get damaged. But the damage should not affect its functionality as far as possible. In the extreme case, complete collapse of the elevated tank should be avoided.

Those damaged tanks include both, cylindrical shaft supported as well as trestle supported tanks. In view of the fact that in Gujarat most of the elevated tanks are supported on cylindrical shaft, quite a few case of failure or severe damage of cylindrical shaft supported elevated water tanks came into knowledge. The focus of this dissertation is to analyze and study the behavior and performance of shaft supported elevated tank under different loads like gravity loads, seismic load. Further behavior of shaft supported staging under the influence of different openings and stress concentration around these opening is also studied.

1.3 OBJECTIVE OF STUDY:

On 26th January 2001, Bhuj earthquake most of the structures have shown some distress. The amount of distress may vary from structure to structure. Three elevated water tanks located in a radius of approximately 125 km from the epicenter collapsed completely, and many more were damaged. RC cylindrical shafts developed circumferential flexural cracks near the base. Similar damage to support shafts has also been observed in the past earthquakes also. The distress could be because of faulty design, improper detailing, poor construction practice or some deteriorating factor. The poor performance of such important public utility requires attentive approach.

At present, IS 1893: 1984 describes the seismic force criteria for elevated water tanks. This code does not count the convective hydrodynamic pressures in the analysis of tank wall and assumes the tank as a single degree of freedom idealization. However, the accurate approach for analysis of water tanks as practiced in most countries is to model the tank with two masses representing the impulsive as well as convective components of liquid.

Each and Every structure must perform well throughout its design life without any serviceability problems. Especially all important public utility structures, like water tanks, should withstand in all circumstances. Since the RC cylindrical shafts have shown some damage in form of circumferential flexural cracks near the base, the reasons of the same should be properly studied and some solution should be found out to come out of such problem.

Present study is related to the elevated shaft supported water tanks only. The main objective of this study is to study effects of different openings provided in the RC cylindrical shafts. The nature of stress distribution around the openings is studied and suitable measures are suggested. Other then the openings, the problems related to construction inaccuracies are also studied to find its role in distress of RC cylindrical shafts, if any.

1.4 OUTLINE OF DISSERTATION:

In the present work, analysis and design of shaft type of staging and foundation system for elevated water tank (Intze Type) under the effect of dead load, live load, wind load and seismic load has been carried out. Wind load calculations have been performed as per IS: 875 (Part-III) – 1985. Seismic load calculations have been performed as per IS: 1893 – 1984 and Proposed draft for IS 1893 (Part II) and comparison has been made of design forces and physical quantities required for the execution of the work. Further the effects of different openings on performance and design of shaft and foundation system have been studied. The construction inaccuracies in the shaft of water tank, for which the Indian Standard code is silent, are studied and outcomes are summarized to get idea for the same.

First chapter includes understanding about elevated water tanks along with their importance in the water distribution system, classification of the same and need of study of the subject along with outline of the dissertation.

Second chapter presents several research papers published till date which are related to modeling, lateral load distribution and analysis and design part of elevated water tank especially shaft type of water tanks.

Third chapter includes some of the important criteria for analysis and design of elevated water tanks along with codal provisions for the same. Further this chapter includes a brief review of present code for seismic analysis and design of elevated reinforced concrete water tanks IS: 1893-1984 and Proposed draft for IS 1893 (Part II).

Fourth chapter include analysis and design of supporting structure (shaft type staging) of elevated water tank and foundation system for the same as per IS: 1893 – 1984 and Proposed draft code for IS: 1893 (Part II). Further comparison of forces and physical quantities for supporting structure and foundation system as per IS: 1893 – 1984 and Proposed draft code for IS: 1893 (Part II) is done.

Fifth chapter includes the effects of various openings, present in the shaft at various levels, on the performance of shaft along with the foundation system. Study of nature of stress distribution around the openings has been studied for the same.

Sixth chapter covers the construction inaccuracies in shaft of water tanks. The study on effects of inaccuracies in shaft wall construction (certain section of wall going out of plumb at various levels) has been carried out. IS code for staging of shaft does not give permissible tolerances in shaft construction.

Seventh chapter addresses the conclusion and future scopes of the present study.

2 LITERATURE REVIEW

Construction of overhead water tower is required for storing water and maintaining pressure in the pipe line. Cracking in container will not serve the functional purpose while cracking in staging will no longer be safe for the water tower to carry on for the rest of its design life. This chapter is the compilation of codes and technical papers which are very important in load distribution, analysis and design of the water towers.

In this chapter research papers published till date for various aspects of elevated water towers are studied. Some of the useful technical papers describing distribution of lateral force, analysis of circular hollow concrete column with different types of reinforcement and time period of supporting structure are presented.

2.1 DISTRIBUTION OF LATERAL AND GRAVITY LOADS ON SUPPORTING TOWER:

Before analysis or design of any structure the load transfer pattern within the structure needs to be studied carefully. Especially in important structures like elevated water tanks, where very heavy loads are accumulated on top of the staging. Such structures are loaded with permanent loads like self weight of structure itself, load of water, weight of staircase, other services and live load if any. Add to such permanent loads structure is subjected to infrequent lateral loads like wind load and earthquake loads. Permanent loads which are vertical in nature are distributed evenly on the supporting structure but the lateral loads are distributed on various parts of supporting structure based on its location from neutral axis and axis of bending. Therefore distribution of lateral load will not remain uniform at all locations of the shaft.

S. K. Kundoo [Indian Concrete Journal, February-1977] has worked in the same direction and carried out work for distribution of vertical and horizontal loads on supporting structure of elevated water tank. The trestle type of supporting structure of reinforced concrete overhead reservoirs is highly indeterminate. Such supporting structures with closely spaced horizontal and vertical members are sufficiently rigid and therefore it is quite understandable that the vertical load due to self weight of container, water, supporting structure and other members will be distributed equally on all the columns. The same principal can be adopted for the shaft type of staging replacing the columns of trestle by number of nodes of elements of shaft. Therefore the vertical load in form of self weight of structure, weight of water, other services and live load if any is distributed equally on each node of the shaft. But the magnitude and sign of vertical reactions on column for overturning moment due to lateral loads (Seismic load or wind load) varies directly with the distance from the neutral axis of the column group. Similarly for the shaft the magnitude and sign of vertical reactions at various nodes of the shaft for overturning moment due to lateral loads (Seismic load or wind load) varies directly with the distance from the neutral axis of the shaft. The purpose of this study is to evaluate the distribution of shear due to lateral loads on the frame. Here the distribution co-efficient are worked out for vertical columns only and the points of contra flexure for all the columns are assumed to be at mid-height between horizontal bracings.

Based on above methodology the vertical forces due to permanent loads are distributed equally at each node of the shaft. The axial forces due to moment and horizontal forces because of lateral loads (Seismic load or governing load) are distributed in form of axial compression or tension and shear force. The distribution of axial compression or tension is similar to distribution of bending stress and distribution of shear is very similar to distribution of shear stress across the section.

2.2 STRENGTH AND DUCTILITY OF CIRCULAR HOLLOW REINFORCED CONCRETE COLUMN SECTIONS:

Shaft is a hollow column, which will be of either cylindrical or hyperbolic shape. When its thickness is very small compared to its diameter it behaves like a membrane structure. Ductility of RC shafts is under question by the Proposed draft for IS 1893 (Part II). In the proposed code the R (Response Reduction Factor for tanks) value is considered as low as 1.8 for the RC shaft. When this value is compared with the R value of masonry shaft (R = 1.5) it gives and impression that ductility of RC shaft and masonry shaft are similar. It is pointed out in the commentary to the proposed code that building frames have greater number of redundancies as compared to shaft and hence they can dissipate seismic energy to a greater extent. This argument can be extended to shafts, which are continuous membranes and like all other membrane structures have infinite redundancies due to enormous number of alternative paths for transmission of loads.

It is interesting to know that ATC – 19 [4] committee notes in its first concluding remark that, “There is no mathematical basis for the response modification factors tabulated in modern seismic codes in the United States”. It may be pointed out that there have been more collapses in normal times than during earthquakes due to poor design and construction. It is important to understand that level of available skills for construction will vary widely from place to place. One can not escape the fact that poor workmanship and associated problems of durability are far greater problems than the performance during earthquakes.

F. A. Zahn, R. Park, M. J. N. Priestley [ACI Structural Journal, March-April 1990] have investigated the flexural strength and ductility available from circular hollow reinforced concrete column sections with single layer longitudinal and transverse reinforcement placed near outside face of the section. For column members of smaller cross-sectional size, it is convenient to place the longitudinal and transverse reinforcement in single layer only near outside face of the cross-section. Such six columns with different axial load ratios were tested and those results were compared with analytical results. The experimental results confirmed the analytical prediction that, circular hollow columns with low axial load, moderate longitudinal reinforcement percentage, and thick wall performs in a ductile manner, where as, circular hollow columns with high axial load, high longitudinal reinforcement percentage, and thin wall performs in a brittle manner. Because of single layer reinforcement near the outside face only, the inside face of the concrete is not properly confined and therefore the concrete near the inside face of the hollow circular section gets crushed.

Further Mander, Priestley and Park and Whittaker, Park and Carr have studied and investigated the flexural strength and ductility available from rectangular and circular hollow reinforced concrete column sections with layers of longitudinal and transverse reinforcement placed near both the inside and outside faces of the section and tied through the wall thicknesses by transverse reinforcement. Such columns, when properly detailed, were shown to perform in a ductile manner during cyclic lateral loading in the inelastic range, since the core of the tube walls was well confined by the reinforcement. It has been well established that well-confined concrete members can sustain large concrete compressive strains without significant loss of compressive strength.

2.3 ANALYSIS AND DESIGN:

Construction of overhead water towers is required for storing water and maintaining pressure in the pipe line. A combination of such heavy loads, construction problems and inadequate soil investigations may lead the structure to functional or design life problems.

- The water load goes on varying everyday. The stresses in concrete vary from 30% to 100% both ways. This variation causes more fatigue.
- Inadequate foundation investigations. Study of ground water table is not taken into design properly.
- In absence of ductile detailing, water tank may fail due to earthquake;

2.3.1 Modeling:

➢ G. Tripathi et al. [Proc. Structural Engineering Convention-SEC 2000] carried out seismic analysis of a typical liquid storage tank. For modeling impulsive and convective masses, multiple impulsive-convective mass lumping scheme has been used. This multiple impulsive-convective mass lumping scheme has been found more realistic over the conventional approaches used by different design codes for the hydrodynamic modeling of water mass inside the liquid storage tanks, and needs less computational efforts compared to the other approaches. In this study tank geometry has been modeled using 4-noded plate and shell elements and water mass inside the tank is distributed appropriately to consider hydrodynamic forces developed due to impulsive as well as convective effects. Further, for the analysis response spectrum analysis has been performed using FEM and the forces and moments obtained are compared with the forces and moments obtained with conventional design code for water mass modeling.

➢ In the Proposed draft for IS 1893 (Part II) the parameters for the spring mass model are given for circular and rectangular tanks only. For tanks of shapes other than these, provision has been made to use equivalent circular tank of same volume and diameter of tank at top level of liquid.

This provision is an outcome of the work done by Sanjay P Joshi [ISET Journal of Earthquake Technology, Technical Note: Vol 32, March-September 2000, Page 39-47]. In this work equivalent mechanical model for rigid type tanks for horizontal vibration is developed. Parameters of the model are evaluated for a wide range of shapes of the tank. The objective of this paper is to develop a mechanical model for intze type tanks and explore the possibility of using for their design values already available for cylindrical tanks with necessary modifications. For the intended purpose comparison of different shapes with those of cylindrical tank model is done. These comparisons have shown that errors associated with the use of equivalent cylindrical tank model in place of the intze tank model are small.

Therefore use of parameters for equivalent cylindrical tank for other shapes is logical.

2.3.2 Lateral Stiffness and Time Period of Overhead Tower:

Analysis of water tank structures for earthquake or dynamic wind loads using gust factor method requires calculation of the fundamental time period. The relevant Indian code does not provide any expression for finding out either lateral stiffness or time period of the water tower structures. It has been as common practice among designers to use approximate procedure for analysis and design of water tank structures. Exact calculation of lateral stiffness of lateral stiffness can be made using stiffness matrix approach applying a lateral load at the top of tank. The stiffness is evaluated by the ration of load applied to the deflection of tower at top. This method, though exact, requires computing facility, software and also time. In this paper Shri. R. K. Ingle [The Indian Concrete Journal, September 1997, Page 497-499] has proposed an equation to determine lateral stiffness of overhead tank, which is used in finding out time period of the tank structure. This paper deals with approximate calculation of lateral stiffness and fundamental time period for water tank structures, with rectangular configuration of columns and braces in plan.

2.3.3 Analysis and Design of Elevated Water Tanks:

- The overall axisymmetric structural geometry and mass distribution of such structures may leave only a small accidental eccentricity between centre of stiffness and centre of mass. Such a small accidental eccentricity is not expected to cause a torsional failure. But torsional failure of some reinforced concrete as well as steel elevated water tanks has occurred in past earthquakes. The latest failure of this kind was the torsional failure of a reinforced concrete elevated tank during 1993 Killari, India earthquake. In this case, the tank container collapsed vertically. This vertical collapse suggests that torsional vibration may have been the primary cause of failure.

Sekhar Chandra Dutta, C.V.R Murthy and Sudhir K. Jain [Structural Engineering and Mechanics, Vol. 9, No. 6 (2000), Page 615-636] did work for seismic torsional vibration in elevated tanks. This paper studies the possibility of torsional behavior of elevated water tanks due to such small accidental eccentricity in the elastic as well as inelastic range. The study has shown that the presence of a small eccentricity may lead to large displacement of the staging in the elastic range. The tanks supported on staging with less number of columns and panels are found to have greater torsional vulnerability.

- Elevated liquid storage tanks are structurally flexible in horizontal direction. However it was observed in the past that the elevated liquid storage tanks were damaged due to earthquakes. The protection against is ensured by two techniques, namely conventional and base isolation. The conventional technique involves the strengthening of the structure by means of increasing the size of different component members. The base isolation approach involves the implanting the isolation devices at the base of the structure to decouple the structure from ground and increases the fundamental period of structure. In the later technique the forces transmitted to the structure are reduced significantly.

M. K. Shrimali [Structural Engineering Conventions, SEC-2003] has done work to study seismic response of elevated liquid tank isolated, using base isolation technique under the effect of earthquake ground motion. In the modeling sloshing, impulsive and rigid masses are considered. Equivalent springs are used to connect sloshing and impulsive masses with the tank wall and mass of the tower structure is lumped equally at top and bottom. The stiffness and damping of the sloshing and impulsive masses are calculated as per single-degree-of-freedom system concept. Further, slender and broad tanks are taken to study the effectiveness of isolation system for different types of tank. Furthermore, comparative performance of the tanks isolated using various bearings is also investigated to study effectiveness of different types of isolation bearings to control response.

➢ A structure designed using the limit state design is strong enough to resist the worst combination of loads with a known factor of safety and it behaves satisfactorily at service loads. If optimization techniques are coupled with the limit state method, the thickness of tank wall and amount of reinforcement can be further reduced and the cost of the structure brought down considerably. In the current method of design,

- The permissible stresses in the reinforcement are reduced in both direct and bending tension.
- Tensile strength of concrete is ignored.
- A minimum reinforcement of about 0.3% is provided.

Such conventional design method is conservative and results in relatively thick walls with a substantial amount of reinforcement. On the other hand, if the design is based on strength criterion alone, it may lead to the excessive cracking of the concrete. Limit state method offers an alternative procedure which results in considerable savings. Prof. S. R. Adidam and Prof. A. V. Subramanyam [Vol.108, No.6, June 1982] carried of work on the subject of optimizing the design of reinforced concrete water tanks. In this study a solved example of a book is chosen and is solved using limit state method to get an idea of the saving in the cost that can be obtained. For cost comparison cost of concrete, reinforcement and formwork is chosen as the objective function in the optimization problem formulation.

➢ As we know that cracking in container will not serve the functional purpose while cracking in staging will no longer be safe for the water tower to carry on for the rest of its design life. Therefore the stress analysis in the container is as important as the stress analysis of the supporting structure. In the context of stress analysis, the assessment of the expected failure modes for the structure caused by seismic event is a very important consideration. Generally the structure is considered to fail functionally when the degree of distress in the structure due to seismic event achieves a certain level, beyond which it would be reasonable to assume that the appropriate and designated functioning of the structure could be interfered substantially. However the degree of distress is very critical and must reflect the location of the structural distress.

Kapilesh Bhargava, A. K. Ghosh, & S. Ramanujam [Structural Engineering Conventions, SEC-2003] has carried out work for the stress analysis of a water-storage structure. In stress analysis, static and seismic loading were considered and the evaluation of stresses at different locations for various combinations of static and seismic loading was carried out. Further most likely failure modes for the structure during seismic load were assessed.

2.4 Performance of Elevated Water Tanks in past earthquakes:

Many elevated water tanks suffered damage to their supporting structure in the Bhuj earthquake of January 26th, 2001 and at least three of them collapsed. The majority of these tanks are supported on cylindrical shaft type staging. Further these water tanks are located in the area of a radius of approximately 125 km from the epicenter.

Durgesh C. Rai [Proc. Indian Acad. Sci. 112, No.3, September 2003, Page 421 - 429] carried out work to study the performance of elevated tanks during 2001 Bhuj earthquake. According to his observations, the current designs of supporting structures of elevated water tanks are extremely vulnerable under lateral forces due to an earthquake. Further he found that most of the shaft type elevated tanks either met or exceeded the strength requirements of IS: 1893-1984. Further they were all found deficient when compared with requirements of the International Building Code. As per his observations IS: 1893-1984 is unjustifiably low for such structures which do not have the advantage of ductility and redundancy. He suggested increasing the level of forces by a factor of three to meet the requirements of International Building Code immediately.

3 DESIGN CONSIDERATIONS

This chapter deals with general requirements and codal provisions of elevated water towers. These general requirements include practical considerations, types of loads, stiffness and time period of staging. For example dead load, live load, wind load, earthquake load etc. The codal provisions includes different provisions made in IS codes for the water tanks. For example permissible stresses in concrete, permissible stresses in steel, minimum cover etc.

3.1 GENERAL CONSIDERATION:

To deal with any analysis and design problem the first and very important part is to study type and amount of loads that the structure has to carry during its design life. These loads will help us, understanding the behavior of the structure and its probable modes of failure under the effect of different loads.

- **Dead Load:**

To start with, the designer does not have exact value of the dead load of the structure. He can make an estimate either by reference to similar other existing structures or by empirical formulae. So dead load of the structure should be checked after completion of design. Dead loads include the self weight of the container, supporting system, staircase, railing and the water stored. Dead load should be calculated on the basis of unit weights taken in accordance with IS: 1911 – 1967. Unless more accurate results are necessary, the unit weight of concrete may be taken as 25 kN / m^2.

- **Imposed Load:**

The use of term live load has been modified to 'imposed load' to cover not only the physical contribution due to persons but also due to nature of occupancy, the furniture and other requirements which are part of the character of the occupancy. The live load in most cases is indefinite and liable to change with time. As far as water tank is concerned the amount of live load it has to carry is far lesser than the other loads. So it has little contribution as far as design is concerned. In case of water tank live load is only because of the maintenance staff, which is limited only on the staircase and inspection gallery portion. Imposed loads shall be calculated in accordance with IS: 875 (Part 2) – 1987.

- **Wind Load:**

Wind loads are basically horizontal forces. Forces produced in case of wind load are either static or dynamic in nature. It is a common practice to design the structure by considering static pressure due to wind, acting on various projected surfaces of the structure. Wind load depends upon topography of the area, height of structure, wind velocity, and most importantly surface area. It is a common practice to design the structures for the static wind loads. But for some special structures dynamic affect of the wind needs to be considered. In case of gust load, the dynamic affect of the wind should be considered. The major difficulty in assessing wind load arrives at the time of cyclones or tornados. It is very difficult to asses the magnitude of forces during cyclones or tornados. For analysis wind effect needs to be considered along with both axes of the building, taken one at a time and in reversible directions and wind loads are combined with normal dead and imposed loads. Wind loads shall be calculated in accordance with IS: 875 (Part 3) – 1987.

- **Seismic Load:**

Seismic forces are also horizontal forces. Basically this force is directly proportional to the mass or self weight of the structure. These forces are also estimated as equivalent static forces causing oscillation of the structure. The response of the structure to seismic force is a function of the nature of foundation soil, materials, form, size and mode of construction of structures, and characteristics (intensity, duration etc.) of the ground motion. The design lateral force shall be considered in each of the two orthogonal directions, but along one direction at a time. For calculation of seismic loads response spectrum method is used in which frequency of vibration and mode shapes have to be worked out. For water tanks when seismic loading is considered following two cases must be considered.

- Tank Full
- Tank Empty

Seismic loads should be calculated in accordance with IS: 1893 – 1984 or with proposed draft code. As many research papers have stimulated, the forces estimated for elevated tanks as per IS 1893: 1984 are 2.5 to 4.5 times lower than those prescribed by codes of some other countries. The reason behind this is prescription of same basic seismic force as that for the most ductile building frame for which the design force is the least.

- **Staging Stiffness and Time Period:**

In both single and two degree of freedom idealization stiffness of the staging needs to be obtained for calculating the time period. The design seismic force for the water tank depends upon its flexibility and hence on the time period. For shaft supported tanks, staging stiffness is calculated using equation $3EI/L^3$, which is not accurate as for derivation the load is applied at the top of staging instead of CG of container.

- **Hydrodynamic Pressure in Tanks:**

When a tank containing liquid vibrates, the liquid exerts impulsive and convective hydrodynamic pressure on the tank wall and the tank base in addition to the hydrostatic pressure. During lateral base excitation, tank wall is subjected to lateral hydrodynamic pressure and tank base is subjected to hydrodynamic pressure in vertical direction. At the time of lateral vibrations the liquid in the lower region of tank behaves like a mass that is rigidly connected to tank wall. This mass is termed as impulsive liquid mass which accelerates along with the wall and induces impulsive hydrodynamic pressure on tank wall. Liquid mass in the upper region of tank undergoes sloshing motion and this mass is termed as convective mass.

3.2 CODAL PROVISIONS FOR DESIGN OF WATER TANK:

- **Permissible stresses in concrete on Water Face (IS 3370-Part II):**

TABLE: 3.1: PERMISSIBLE STRESSES IN CONCRETE ON WATER FACE

	Permissible Stress in Concrete Grade N/mm^2			
TYPE OF STRESS	M 20	M 25	M 30	M 35
Direct Tension	1.2	1.3	1.5	1.6
Bending Tension	1.7	1.8	2.0	2.2
Direct Compression	5.0	6.0	8.0	9.0
Bending Compression	7.0	8.5	10.0	11.5

- **Minimum Recommended Clear Cover to the Reinforcement (IS 3370-Part II):**

TABLE: 3.2: MINIMUM RECOMMENDED CLEAR COVER TO THE REINFORCEMENT

CONDITION	**COVER (mm)**
Direct Tension	20mm or diameter of bar
Bending Tension	25 mm
Alternating wetting and drying condition	30 mm

- **Permissible stresses in Reinforcement (IS 3370-Part II):**

TABLE: 3.3: PERMISSIBLE STRESSES IN REINFORCEMENT

Types of Stress in steel Reinforcement	STRESS (N/mm^2)	
	Plain Mild Steel	HYSD BARS
1) Tensile stress in members under direct tension:	115	150
2) Tensile stress in members due to bending:		
a) On liquid retaining face of members:	115	150
b) On face away from liquid for members less than 225mm:	115	150
c) On face away from liquid for members 225 mm or more in thickness:	125	190
3) tensile stress in shear reinforcement:		
a) For members less than 225 mm thickness.	115	150
b) For members 225 mm or more in thickness.	125	175
4) Compressive stress in columns subjected to direct load:	125	175

- **Minimum Reinforcement:**

1. The high strength deformed bars in walls, floors and roof in each of the two directions at right angles should be 0.24% with 0.2% proof stress of 415 N/mm^2 of the gross concrete section in that direction for sections up to 100mm thick. For sections of thickness greater than 450mm, the minimum high strength deformed bars in each of the two directions should be 0.16% further for sections of thickness greater than 100mm and less than 450mm, the minimum high strength deformed bars in each of two directions may be linearly interpolated from 0.24% to 0.16%
2. For sections of thickness 225mm or greater, two layers of reinforcing steel shall be placed one near each face of the section to make up the minimum reinforcement.

   NOTE: The percentage shall be increased by 25% in case of mild steel reinforcement.

- IS: 11682-1985 clause 8.2.1 suggests minimum thickness of shaft for staging 150mm, when internal diameter exceeds 6m, the minimum thickness shall be given by
- Minimum $t = 150 + \frac{D-6000}{120}$ where, D is the internal diameter of concrete shell in mm.

- **Reinforcement requirements for hollow shaft staging:**

1. The minimum vertical reinforcement 0.25% for deformed bars, of the concrete section under consideration. The reinforcement shall be provided in two layers, one near each face to make up minimum reinforcement.
2. Minimum diameter of the longitudinal bars shall be 10 mm and maximum c/c distance of reinforcement shall not exceed twice the thickness of the shell nor 400 mm c/c in each layer.
3. The circumferential reinforcement shall not be less than 0.2%, of concrete area in vertical section under consideration subject to a minimum of 4 cm^2 per meter height. In case if the vertical reinforcement is provided in two

layers, the circumferential reinforcement shall also be provided in two layers and dividing minimum reinforcement equally in two layers.

4. The spacing of the reinforcement shall not be less than 300 mm or shell thickness whichever is less.
5. At both the top and bottom of the each opening, additional reinforcement shall be placed having an area at least equal to one half the area of the design circumferential reinforcement interrupted by the opening.
6. At both the sides of the each opening, additional reinforcement shall be placed having an area at least equal to one half the area of the design vertical reinforcement interrupted by the opening.
7. Diagonal reinforcement with total cross-sectional area in cm^2 of not less than half of shell thickness in cm shell be placed at each corner of the opening.

3.3 STUDY OF IS: 1893–1984:

As per IS:1893-1984, for the purpose of seismic analysis, elevated tanks shall be regarded as systems with a single degree of freedom with their mass concentrated at their centers of gravity. A satisfactory spring – mass analogue to characterize basic dynamics of elevated tanks was suggested by Housner (1963) after Chilean Earthquake of 1960. This approach has been found to be more realistic over the conventional approaches used by the design codes. The two mass models adequately represent the impulsive and convective modes of vibration as observed in many experimental studies.

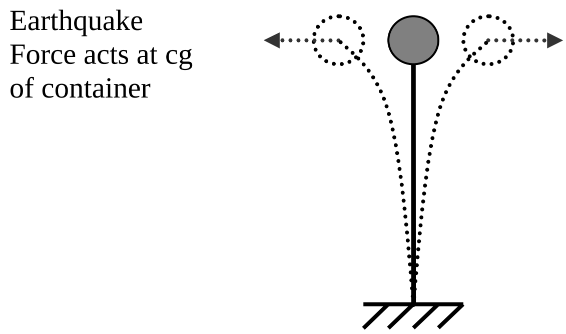


FIGURE 3.1: SINGLE DEGREE OF FREEDOM IDEALIZATION FOR ANALYSIS OF WATER TANKS

Figure 3.1 shows single degree of freedom idealization used in IS: 1893-1984. The design force for the tank highly depends upon the natural time period and hence the natural time period should be calculated with greater accuracy and it needs to be reasonably accurate.

As IS: 1893-1984 suggests a single degree of freedom idealization, accuracy of estimated natural time period is questionable, particularly when the tank is partially full due to the reason that sloshing mode of vibrations also contribute to the seismic response of the system. For some containers (large width to depth ratios), single mass model is certainly not an appropriate representation as most of the mass in the tank acts as a convective one and this will take to misleading behavior of the tank.

The damping in the system may be assumed as 2 percent of the critical for steel structures and 5 percent of the critical for concrete (including masonry) structures.

The free period T, of such structures shall be calculated from the following formula:

$$T = 2\pi \sqrt{\Delta/g}$$

Where Δ =The static horizontal deflection at the top of the tank under a static horizontal force equal to a weight W acting at the center of gravity of tank.

g = Acceleration due to gravity (m/ s^2)

The code requires that design shall be carried out for both conditions

a) Tank full, the weight of contents is to be added to the weight under empty condition.
b) Tank empty, the weight W used in the design shall consist of the dead load of tank and one third the weight of staging.

Using the time period T for appropriate damping, the spectral acceleration shall be read off from the average acceleration spectra given in Fig.2 of code as given below.

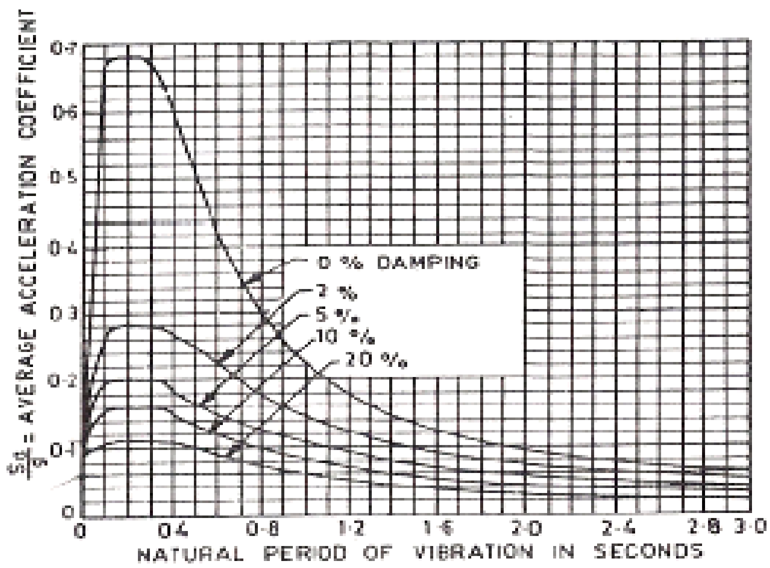


FIGURE 3.2 RESPONSE SPECTRA SUGGESTED BY IS: 1893-1984

Clause 3.4.2.3 of IS: 1893-1984 gives the expression for calculating the design horizontal seismic coefficient (α_h)

$$\alpha_h = \beta I \, F_0 \, (S_a / g)$$

Where,

β = Co-efficient depending upon the soil-foundation system as per table 3 of code

I = Importance factor as per table 4 of code;

F_0 = Seismic zone factor given in table 2 of code;

S_a / g = Average acceleration coefficient obtained from acceleration spectra given in figure 2 of IS: 1893-1984

3.4 STUDY OF PROPOSED DRAFT CODE IS: 1893 (PART–2):

IS: 1893 (Part 2) (Proposed Draft Code for Seismic Analysis of Liquid Retaining - Structure) contains provisions on liquid retaining tanks. This standard incorporates the following important provisions and changes for elevated water tanks:

- For elevated tanks, the single degree of freedom idealization of tank is replaced by a two-degree of freedom idealization and is used for analysis.

- Bracing beam flexibility is explicitly included in the calculation of lateral stiffness of tank staging.
- The effect of convective hydrodynamic pressure is included in the analysis.
- The distribution of impulsive and convective hydrodynamic pressure is represented graphically for convenience in analysis; a simplified hydrodynamic pressure distribution is also suggested for stress analysis of the tank wall.

3.4.1 TWO MASS IDEALIZATION:

A satisfactory spring mass analogue to characterize basic dynamics of elevated tanks was suggested by Housner (1963) after the chileane earthquake of 1960. This two mass model adequately represents the impulsive and convective modes of vibration. Figure 4.4 shows the proposed two mass model for the elevated water tanks.

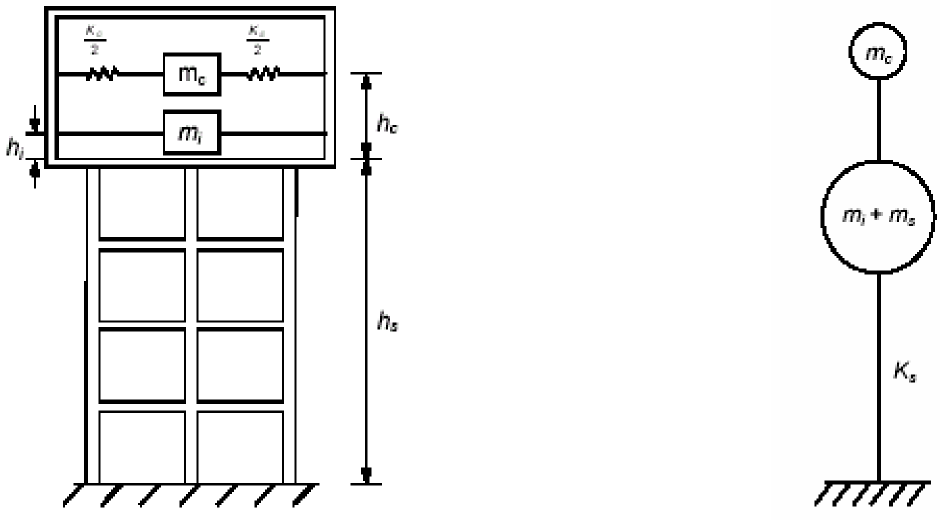


FIGURE 3.3 PROPOSED TWO MASS IDEALIZATIONS FOR ANALYSIS OF ELEVATED TANKS

Where,

m_i = Impulsive Mass

m_c = Convective Mass

m_s = Structural mass of container of elevated tank and one third of staging.

K_c = Spring stiffness of convective Mode.

K_s = Lateral stiffness of tank staging

h_c = Height at which resultant of convective pressure on wall is located from the bottom of tank wall

h_c* = Height at which resultant of convective pressure on wall and base is located.

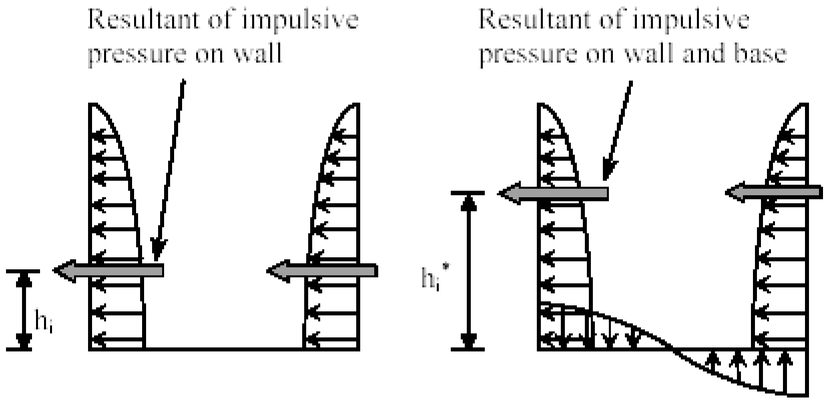
h_i =Height at which the resultant of impulsive hydrodynamic pressure on wall is located from the bottom of tank wall

h_i*=Height at which the resultant of impulsive pressure on wall and base is located from the bottom of tank wall.

3.4.2 CONVECTIVE PRESSURE AND IMPULSIVE PRESSURE:

The hydrodynamic pressure on the wall and base of the tank is comprised of convective pressure and impulsive pressure. Under lateral accelerations the fluids in the upper regions of the tank do not move with the tank wall thus generate seismic waves or sloshing motion of fluids (Convective behavior). On the contrary, fluids nearer the base of tank move with the tank structure and therefore add to the inertial mass of tank structure (Impulsive behavior). The portion of the tank fluids that act in impulsive mode largely depends on the aspect ratio (height/diameter) of the tank. For tanks of very low aspect ratio, a very little of tank fluids acts in the impulsive mode. Various experimental works suggest that convective mode period is considerably higher than the impulsive mode period.

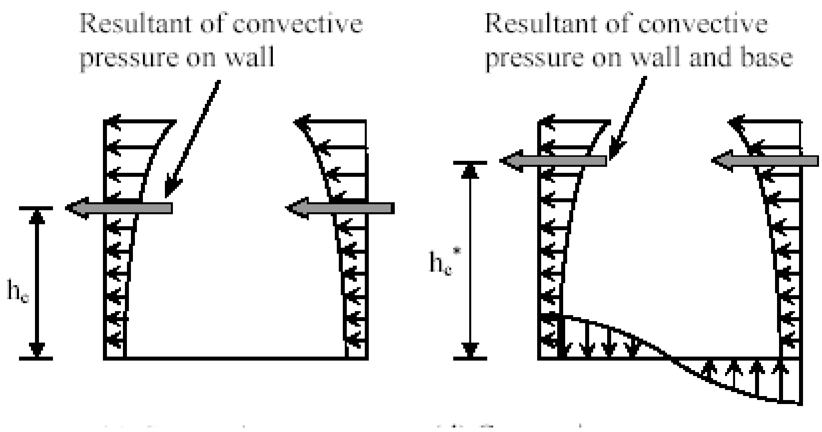
In the spring mass model of tank, h_i is the height at which the resultant of impulsive hydrodynamic pressure on wall is located from the bottom of tank wall. On the other hand, h_i* is the height at which the resultant of impulsive pressure on wall and base is located from the bottom of tank wall. Thus, if effect of base pressure is not considered, impulsive mass of liquid m_i will act at a height of h_i and if effect of base pressure is considered, m_i will act at h_i* Heights h_i and h_i*, are schematically described in Figures 4.5 (a) and (b).



(a) **(b)**

FIGURE 3.4 IMPULSIVE PRESSURE ON THE WALL AND BASE OF TANK

Similarly, h_c is the height at which resultant of convective pressure on wall is located from the bottom of tank wall, while, h_c* is the height at which resultant of convective pressure on wall and base is located. Heights h_c and h_c* are described in Figures 4.6 (a) and (b).



(a) **(b)**

FIGURE 3.5 CONVECTIVE PRESSURE ON THE WALL AND BASE OF TANK

3.4.3 TIME PERIOD:

IS: 1893-1984 is silent about calculation of lateral stiffness of the water tower structure. However, proposed draft code gives some idea regarding calculation of lateral stiffness. Clause 4.3.1.3 gives expression for calculating time period for elevated water tanks.

Time period of Impulsive mode, $T_i = 2 * \pi * \sqrt{((m_i + m_s)/K_s)}$

Where,

m_s = mass of container and one third mass of staging

K_s = lateral stiffness of staging

Lateral stiffness of the staging is the horizontal force required to be applied at the center of gravity of the tank to cause a corresponding unit horizontal displacement. Center of gravity of tank can be approximated as combined center of gravity of empty container and impulsive mass of water.

Time period of Convective mode,

$T_c = C_c * \sqrt{(D/g)}$ for Circular tanks

$T_c = C_c * \sqrt{[L / g]}$ for Rectangular tanks,

Where,

C_c = Coefficient of time period for convective mode. Value of C_c for circular tank can be obtained from figure 4.7 and value of Cc for rectangular tank can be obtained from figure 4.8

D = Inner diameter of tank, L = Length of rectangular tank

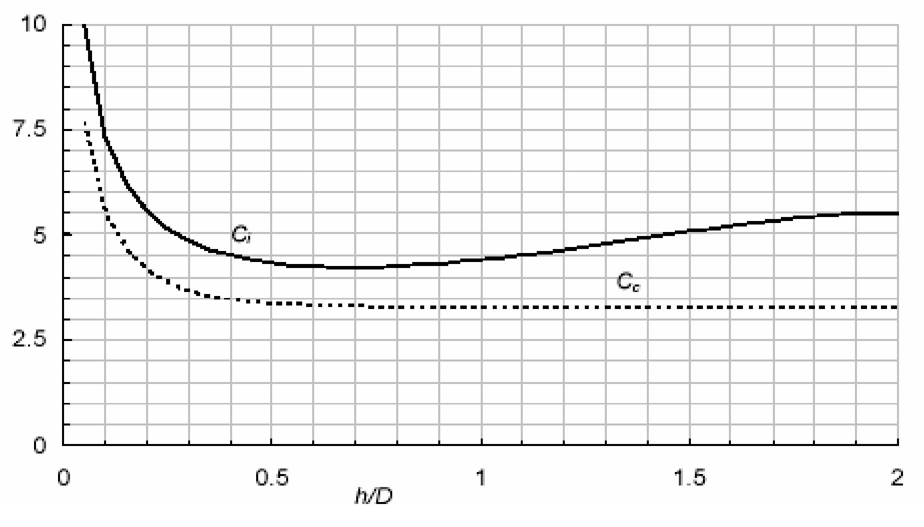


FIGURE 3.6 COEFFICIENT OF IMPULSIVE (C_I) AND CONVECTIVE (C_C) MODE TIME PERIOD FOR CIRCULAR TANK

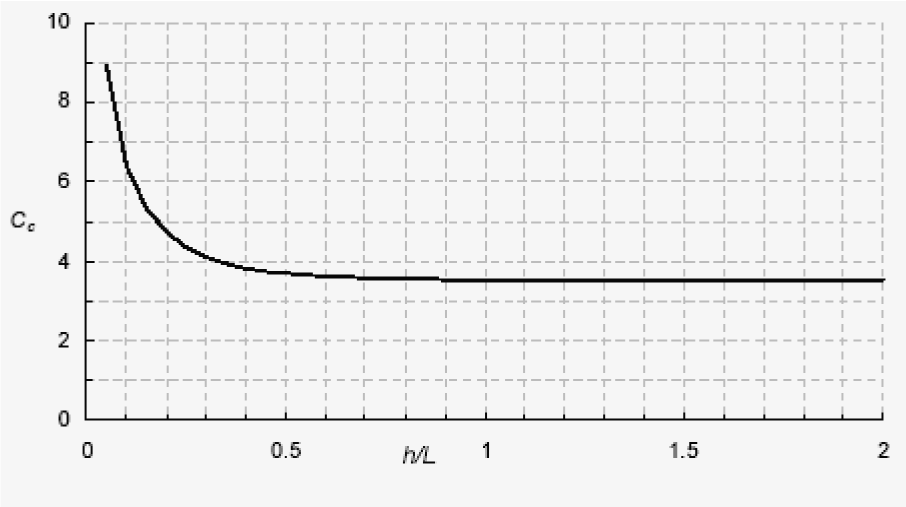


FIGURE 3.7 COEFFICIENT OF CONVECTIVE MODE TIME PERIOD (C_C) FOR RECTANGULAR TANK

3.4.4 DAMPING:

Damping in the convective mode for all types of liquids and for all types of tanks shall be taken as 0.5% of the critical. Damping in the impulsive mode shall be taken as 2% of the critical for steel tanks and 5% of the critical for concrete and masonry tanks. Multiplication factors to be used for damping other than 5 % as per Table 3 of IS 1893 (Part – I): 2002. Coefficient for 0.5 % damping works out to be 1.75.

3.4.5 DESIGN HORIZONTAL SEISMIC COEFFICIENT:

Clause 4.5.1 of Proposed Draft code gives expression to calculate the design horizontal seismic coefficient. Design horizontal seismic coefficient, A_h will be calculated separately for impulsive $(A_h)_i$ and convective $(A_h)_c$ modes.

Design horizontal seismic coefficient A_h shall be obtained by the following expression,

$$A_h = \left(\frac{Z}{2}\ \frac{I}{R}\right) \frac{S_a}{g}$$

Where,

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

I = Importance factor given in Table 1 of this

R = Response reduction factor given in Table 2 of draft code

Sa/g = Average response acceleration coefficient as given by Figure 2 and Table 3 of IS: 1893(Part 1): 2002.

The Response Spectra given in IS 1893: 1984 have been revised in its new edition IS 1893: Part- I- 2002, giving general provisions and criteria for buildings which is given below.

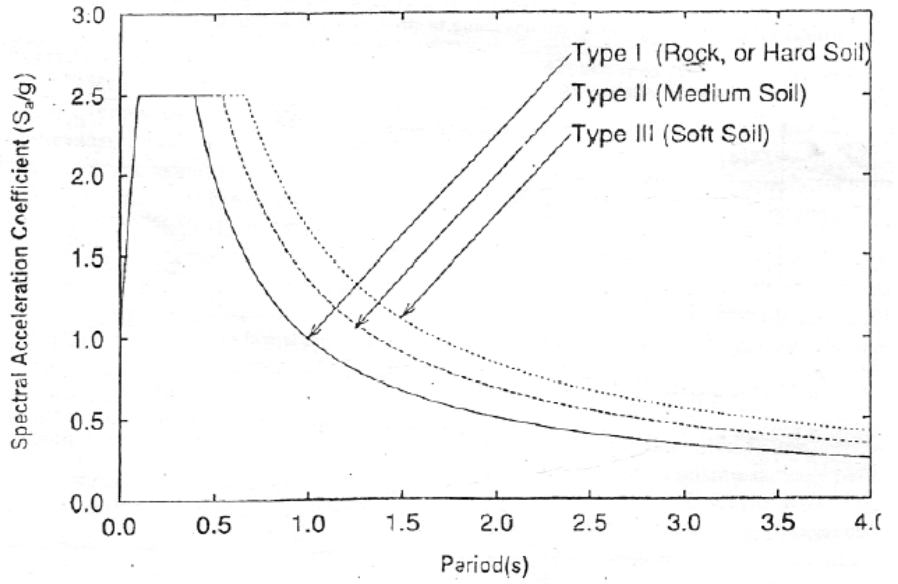


FIGURE 3.8 RESPONSE SPECTRA SUGGESTED BY IS: 1893-2002 (PART-I)

Importance factor (I) is meant to ensure a better seismic performance of important and critical tanks. Its value depends on functional need, consequences of failure, and post earthquake utility of the tank. Draft code classifies tanks into three categories and importance factor to each category is assigned. Highest value of I =1.75 is assigned to tanks used for storing hazardous materials. Since release of these materials can be harmful to human life, the highest value of I is assigned to these tanks. For tanks used in water distribution systems, value of I is kept as 1.5, which is same as value of I assigned to hospital, telephone exchange, and fire station buildings in IS 1893 (Part 1):2002. Less important tanks are assigned I = 1.25.

TABLE 3.4 IMPORTANCE FACTOR (I) FOR TANKS

Type of Liquid Stored in Tank	I
Tanks used for storing highly toxic chemicals, explosives and other highly inflammable liquids, accidental release of which would be dangerous to society	1.75
Tanks used for storing potable water, non- volatile material, low inflammable petrochemicals, etc. and intended for emergency services such as fire fighting services	1.5
All other tanks with low risk to life and with negligible consequences to environment, society and economy	1.25

Response reduction factor (R) represents ratio of maximum seismic force on a structure during specified ground motion if it were to remain elastic to the design seismic force. Thus, actual seismic forces are reduced by a factor R to obtain design forces. This reduction depends on over strength, redundancy, and ductility of structure. Generally, liquid containing tanks posses low over strength, redundancy, and ductility as compared to buildings. In buildings, non structural components substantially contribute to over strength; in tanks, such non structural components are not present. For tanks supported on reinforced concrete shaft, value of response reduction factor given in draft code is as below.

R = 1.3 for Reinforced concrete shaft with reinforcement in one curtain (both horizontal and vertical) at the center of shaft thickness

R = 1.5 for Reinforced concrete shaft with two curtains of reinforcement, each having horizontal and vertical reinforcement

Average acceleration coefficient (Sa/g) The value of Sa/g depends upon the soil condition. For time period is less than 0.1 second, the value of Sa/g shall be taken as 2.5 for 5% damping and be multiplied with appropriate factor, for other damping. Value of multiplying factor for 0.5% damping shall be taken as 1.75.

For time periods greater than three seconds, the value of Sa/g shall be taken as the value corresponding to three second. The values of various Sa/g related to the soil conditions are given below

For hard soil sites

$Sa/g = 2.5$ for $T < 0.4$

$= 1.0/T$ for $0.4 \le T \le 3.0$

$= 0.33$ for $T > 3.0$

For medium soil sites

$Sa/g = 2.5$ for $T < 0.55$

$=1.36/T$ for $0.55 \le T \le 3.0$

$= 0.45$ for $T > 3.0$

For soft soil sites

$Sa/g = 2.5$ for $T < 0.67$

$= 1.67/T$ for $0.67 \le T \le 3.0$

$= 0.55$ for $T > 3.0$

3.4.6 BASE SHEAR:

For tanks total Base Shear has two components namely base shears in impulsive and convective modes respectively. Clause 4.6.2 gives shear at the base of staging.

Base shear in impulsive mode, just above the base of staging (i.e. at the top of footing of staging) is given by

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

Base shear in convective mode is given by

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

Where m_s = Mass of container and one-third mass of staging

Total base shear V, shall be the absolute sum of the base shear in impulsive and convective modes which is given by

$$V = V_i + V_c$$

3.4.7 BASE MOMENT:

Base moment can also be divided in to components, overturning moment in impulsive mode and over turning moment in convective mode. Structural mass m_s which include mass of empty container and one-third mass of staging is considered to be acting at the center of gravity of empty container. Base of staging may be considered at the top of footing.

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

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

Overturning moment in convective mode

$$M^*_c = (A_h)_c\ [m_c\ (h_c^* + h_s)]g$$

Where,

h_s = Structural height of staging, measured from top of footing of staging to the bottom of

tank wall

h_{cg} = Height of center of gravity of empty container, measured from base of staging.

Total moment shall be the absolute sum of the moment in impulsive and convective modes which is given by

$$M = M_i + M_c$$

$$M^* = M^*_i + M^*_c$$

3.4.8 SLOSHING WAVE HEIGHT:

Free board to be provided in a tank should be based on maximum value of sloshing wave height. This is particularly important for tanks containing toxic liquids, where loss of liquid needs to be prevented. If sufficient free board is not provided roof structure should be designed to resist the uplift pressure due to sloshing of liquid.

Sloshing height for rectangular tank can be given by

$$d_{max} = \frac{(A_h)_c\, L}{2}$$

Sloshing height for circular tank can be given by

$$d_{max} = \frac{(A_h)_c\, D}{2}$$

Where $(A_h)_c$ = Design horizontal seismic coefficient for convective mode

4 ANALYSIS AND DESIGN

This chapter gives analysis and design of supporting structure (Circular Shaft) and foundation system as per present code for seismic analysis and design of elevated reinforced concrete water tanks IS: 1893-1984 and Proposed draft for IS 1893 (Part II) for the selected case study. The selected water tank is located at Ajwa, Vadodara. The water tank has capacity of 1800000 Liters and total height of 41m from the natural ground level. The height of the shaft is 30m from the natural ground level. The foundation is 3m below the natural ground level. The water tank is under construction and nearing completion stage.

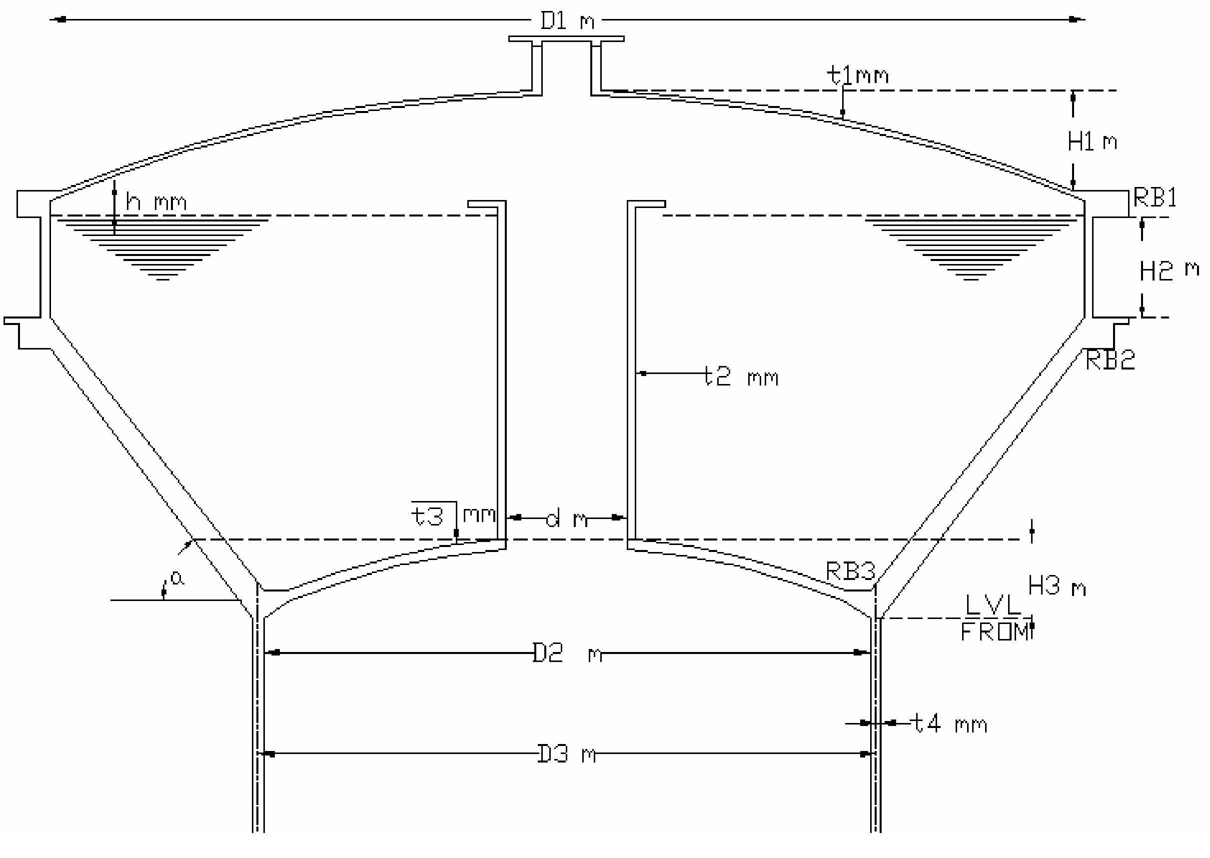


FIGURE: 4.1: DESIGN DATA

DATA:

1. Capacity of the water tank	:		:	1800 m^3
2. Inside Diameter of the tank	:	D_1	:	21 m
3. Inside Diameter of Staging	:	D_2	:	12.17 m
4. Inside Diameter of stair shaft	:	d	:	2.45 m
5. Thickness of top Dome	:	t_1	:	100 mm
6. Thickness of bottom Dome	:	t_3	:	200 mm
7. Thickness of stair shaft	:	t_2	:	160 mm
8. Thickness of staging shaft	:	t_4	:	215 mm
9. Rise of top Dome	:	H_1	:	2.1 m
10. Rise of bottom Dome	:	H_2	:	2.05 m
11. Height of Cylindrical wall	:	H_3	:	1.3 m
12. Diameter of spiral stair column	:	d_1	:	300 m
13. Depth of free board	:	h	:	300 m
14. Inclination of conical wall with horizontal	:	θ°	:	52 degrees
15. Height of Staging form G.L.	:	Hs	:	30 m
16. Depth of footing from G.L.	:	D_f	:	3 m
17. S.B.C at depth 3 m below G.L.	:	SBC	:	203 kN / m^2
18. Grade of Concrete below G.L.	:	M-25	:	25 N / mm^2
19. Grade of Concrete above G.L.	:	M-25	:	25 N / mm^2
20. Grade of Reinforcement	:	Fe-415	:	415 N / mm^2
21. Density of water	:	γ	:	10 kN / m^3
22. Density of Concrete	:	ρ	:	25 kN / m^3
23. Seismic Zone	:		:	III

4.1 WEIGHT CALCULATIONS:

TOP DOME:

Diameter, D_1 = 21.1 m

Rise, R_1 = 2.15 m

Dead load of top dome = 1100.337 kN

Total load on top of top dome = 22.80 kN

Live Load on dome = 273.14 kN

TOTAL WEIGHT = 1123.14 kN (Excluding Live Load)

TOP RING BEAM:

b = 0.65 m

d = 0.55 m

Diameter, D = 22.30 m

WEIGHT = 588.89 kN

CYLINDRICAL WALL:

Outer Diameter, D1 = 21.36 m

Inner Diameter, D2 = 21.00 m

Height, H = 2.05 m

Weight of cylindrical wall = 613.82 kN

Weight of 20 mm thick plaster = 66.03 kN

TOTAL WEIGHT = 679.85 kN

MIDDLE RING BEAM:

Weight of gallery and railing = 171.92 kN

Live load on gallery = 78.14 kN

b = 0.625 m

d = 0.625 m

Diameter, D = 22.25 m

WEIGHT INCLUDING RAILING = 815.375 kN

CONICAL WALL:

Outer Diameter at top, D1 = 21.00 m

Inner Diameter at top, D2 = 20.11 m

Outer Diameter at bottom, D3 = 13.44 m

Inner Diameter at bottom, D4 =12.17 m

Height, H = 5.20 m

V1 = п H/12*(D12+ D32+ (D1*D3)) = 1230.43 m3

V2 = п H/12*(D22+ D42+ (D2*D4)) = 1085.35 m3

VOLUME = V1 – V2 = 145.08 m3

WEIGHT = 3987.268 kN

Weight of 20 thick plaster = 131.10 kN

TOTAL WEIGHT = 3760 kN

SPIRAL STAIR SHAFT WALL:

Height of wall, H = 6.524 m

Outer Diameter, D1 = 2.77 m

Inner Diameter, D2 = 2.45 m

WEIGHT INCLUDING PLATFORM = 199.866 kN

SPIRAL STAIR:

Height of column below plinth = 3.25 m

Diameter of column below plinth = 400 mm

Height of column above plinth = 37.25 m

Diameter of column above plinth = 300 mm

Total weight of column = 80 kN

Rise of step = 200 mm (Assumed)

Approximate no. of total steps = 187 No

Weight of steps = 135 kN

Total weight of spiral stair = 215 kN

BOTTOM DOME:

Diameter, Db = 12.385 m

Rise, Rb = 1.2 m

Weight of bottom dome = 646.069 kN

SHAFT:

Outer Diameter of shaft, D1 = 12.6 m

Inner Diameter of shaft, D2 = 12.17 m

Height of shaft wall up to top of footing, H = 33.25 m

WEIGHT = 6953.7 kN

WATER:

Weight of water including free board and dead storage = 19000 kN

TOTAL LOAD OF EMPTY CONTAINER = 8040.45 kN

TOTAL LOAD OF FULL CONTAINER = 27040.45 kN

TOTAL LOAD OF EMPTY TANK UPTO TOP OF FOOTING = 14994.16 kN

TOTAL LOAD OF FULL TANK AT TOP OF FOOTING = 33994.16 kN

TOTAL LIVE LOAD = 352 kN

4.2 WIND LOAD CALCULATIONS:

As per IS 875 (Part 3): 1987

Physical Parameters:

Height (h) up to top of sky light 38.9 m

Total height 41 m

Diameter of shaft (outer) 12.6 m

Diameter of tank 21.36 m

Surface condition fairly rough tank

Height / Min. Lateral dimension

= 41 / 12.6

= 3.25

Since the height to minimum lateral dimension ratio is less than 5, it is not necessary to check the design for dynamic forces.

Wind Data:

Basic wind speed 44 m/s Ajwa, Vadodara.

Terrain Category: 1

Class of structure: B

Topography: Flat

Design Wind Pressure at Various Heights:

For calculating design wind pressure at various elevations of the tank, only k_2 will change and accordingly pz at ith level have been computed.

At 41.0 m Elevation:

Vb = 44 m/s

k1 = 1

k2 = 1.1575

k3 = 1

Vz = Vb * k1 * k2 * k3

Vz = 50.93 m/s

Design pressure,pz = 0.6 * Vz^2 N/m^2

pz = 1556.32 N/m^2

= 1.56 kN/m2 for 30 m to 41.0 m

Wind Load Calculation:

As per Table: 19, IS 875 (Part 3): 1987

Uplift wind pressure on roof with H = total height of tank portion and

D = Diameter of the tank portion

H / D = 9.65 / 21.36

= 0.45

Cpe = -0.65

Cpi = ± 0.2 (For less than 5 % openings)

Vertical load on roof, = P = 0.785 D2 (Cpi - Cpe) pz with eccentricity e = 0.1 D

P = 473794.46 N

P = 474 kN

Eccentricity = 0.1 D = 2.136 m

Bending Moment = 1013 kN –m

Overall Horizontal Force on Tank and Shaft per meter length:

Ratio of (H / 2D) = 0.226

F = C_f Ae pd

Force Coefficient for tank C_f = 0.7 For circular section with H/D < 0.5

Force Coefficient for shaft C_f = 1.2

TABLE: 4.1: LATERAL FORCE AND MOMENTS DUE TO WIND LOAD

Elevation m	k2	Vz m/s	pz kN/m2	Cf	Ae m2	F kN/m	Height m	Total F kN	L.A. m	Momet At Base kN.m
41.0 to 38.9	1.16	50.93	1.56	0.70	10.68	11.64	2.10	24.43	42.95	1049.42
38.9 to 36.6	1.16	50.93	1.56	0.70	21.36	23.27	2.30	53.52	40.75	2180.99
36.6 to 31.35	1.16	50.93	1.56	0.70	16.98	18.50	5.25	97.12	36.98	3591.37
31.35 to 30	1.16	50.93	1.56	1.20	12.60	23.53	1.35	31.77	33.68	1069.93
30 to 20	1.13	49.72	1.48	1.20	12.60	22.43	10.00	224.27	28.00	6279.47
20 to 15	1.10	48.40	1.41	1.20	12.60	21.25	5.00	106.26	20.50	2178.30
15 to 10	1.07	47.08	1.33	1.20	12.60	20.11	5.00	100.54	15.50	1558.40
10 to 0	1.03	45.32	1.23	1.20	12.60	18.63	10.00	186.33	8.00	1490.64

Total horizontal force due to wind	:	Vb	:	825	kN
Total moment due to horizontal force	:	Mb	:	19400	kN m
Uplift force due to wind	:	U	:	474	kN

4.3 ANALYSIS AND DESIGN OF SUPPORTING STRUCTURE AS PER IS: 1893,1984

4.3.1 SEISMIC LOAD CALCULATION AS PER IS: 1893, 1984:

Weight of empty container	=	8040 kN	(A)
Weight of staging (shaft)	=	6954 kN	(B)
Weight of water	=	19000 kN	(C)
Weight of empty container + 1/3 weight of staging	=	10358 kN	(D)

TANK FULL CONDITION:

Total Seismic Weight:

W = C + D

= 29358 kN

Deflection:

$\Delta = [(W * a^2) / (3 * E * I)] * [1 + (3 * b) / (2 * a)]$

Where,

a = 39.25 m

b = 3.25 m

E = 25000 N / mm^2

I = $160.44 * 10^{12}$ mm^4

Δ = 165.19 mm

Time Period:

$T = 2 * \Pi * (\Delta / g)^{1/2}$

= 0.815 Sec

$\alpha_h = \beta * I * F_0 * (S_a / g)$

Where,

β = 1

I = 1.5

F_0 = 0.2

Sa / g = 0.12

α_h = 0.036

Base Shear (Vb):

Vb = α_h * W

= 1056.88 kN

Base Moment (Mb):

Mb = Vb * a

= 41482.71 kN m

Say 41500 kN m

<u>TANK EMPTY CONDITION:</u>

Total Seismic Weight:

W = D

= 10358 kN

Deflection:

Δ = $[(W * a^2) / (3 * E * I)] * [1 + (3 * b) / (2 * a)]$

= 57.05 mm

Time Period:

T = $2 * \Pi * (\Delta / g)^{1/2}$

= 0.48 Sec

α_h = $\beta * I * F_0 * (S_a / g)$

Where,

β = 1

I = 1.5

F_0 = 0.2

Sa / g = 0.16

α_h = 0.051

Base Shear (Vb):

Vb = α_h * W

= 528.26 kN

Base Moment (Mb):

Mb = Vb * a

= 20734.13 kN m Say 20750 kN m

For this tank, since total base shear for tank full condition (1057 kN) is more than that for tank empty condition (528.26 kN) design will be governed by tank full condition.

Even though the IS: 11682 - 1985 has been silent about the construction tolerances in the shafts of elevated water tanks, as a designer it is a good practice to consider such parameter in design. Therefore additional moment due to certain minimum eccentricity has been included here in design.

Moment due to eccentricity

= 0.05 * 27040.46

= 1352.02 kN m

TABLE: 4.2: FINAL DESIGN FORCES AS PER IS: 1893, 1984

Particular	Tank Full	Due to Eccentricity	Design Forces
Total Base Shear (kN)	1056.88	-	1056.88
Total Base Moment (kN m)	41482.71	1352.02	42834.73

4.3.2 DESIGN OF SHAFT AS PER IS: 1893, 1984:

Mass of container = 8040 kN

Mass of staging = 6955 kN

Mass of water = 19000 kN

Total Vertical Load at top of footing,

Tank Full P_{full} = 33995 kN

Tank Empty P_{empty} = 14995 kN

Tank Full M_{full} = 42850 kN m

Tank Empty M_{empty} = 22102 kN m

Grade of concrete f_{ck} = 25 N/mm^2

Grade of steel f_y = 415 N/mm^2

Inside Diameter of shaft D2 = 12.17 m

Thickness of shaft t_4 = 0.215 m

Outside Diameter of shaft = 12.60 m

Permissible stress in shaft concrete:

(Clause 8.2.6..1, IS 11682: 1985)

Permissible stress = 0.40 σcv = 10 N/mm^2

Where σcv = 28 day ultimate cubic strength of concrete = 25 N/mm^2

Permissible stress in shaft reinforcement:

(Clause 8.2.6..2, IS 11682: 1985)

Permissible stress = 0.60 σsy = 249 N/mm^2

Where σsy = yield or proof stress in steel = 415 N/mm^2

Area of cross section, As = 8.37 m^2

Moment of Inertia of section, Is = 160.44 m^4

Mean radius of shaft r = 6.1925 m

Radius of gyration = 4.38

Direct compressive stress:

(f' ac) = P_{full} / As

= 4.062 N/mm^2

Vertical Reinforcement in Shaft:

Using Tor steel in two layers, p = 0.25 %

Area of reinforcement required Ast = 537.5 mm^2/m total on both faces

= 268.75 mm^2/m on each face

Provide 12 Dia 300 c/c bars on each face

Ast, pro = 376.8 mm^2/m on each face O.K.

Circumferential Reinforcement in Shaft:

Using Tor steel in two layers, p = 0.2 %

Area of reinforcement required Ast = 430 mm^2/m total on both faces

= 215 mm^2/m on each face

Provide 8 Dia 200 c/c bars on each face

Ast, pro = 251.2 mm^2/m on each face O.K.

Stresses are checked at opening in shaft:

As per Clause - 8.2.5 of IS 11682: 1985

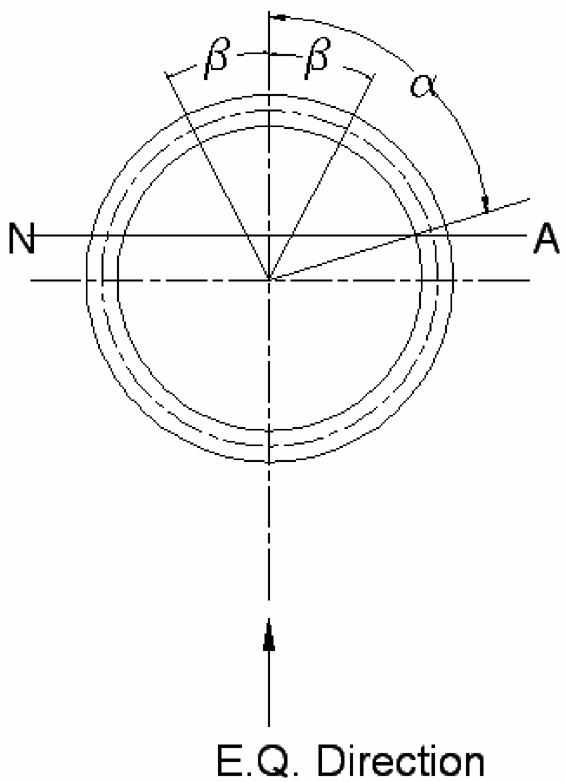


FIGURE: 4.2: SHAFT WITH OPENING

Sin β = 0.6 / 6.1925

= 0.0969

Therefore, β = 5.56 degree

Cos β = 0.9953

Size of opening in shaft, b = 1200 mm

r = 6.1925 m

P = 33995 kN

M = 42850 kN m

e = M / P

= 42852 / 33995

= 1.261 m

e / r = 1.261 / 6.1925

= 0.204

Therefore away from the opening the whole shaft is under compression.

As per clause 8.2.5.1 (a) permissible stress = 5.72 N/mm^2

Therefore OK.

Check for stresses at opening:

As per Clause - 8.2.5 of IS 11682: 1985

e / r ≤ 0.5013

Therefore, OK.

Therefore the whole section is under compression and maximum vertical compressive stress,

σcv = 6.32 N/mm^2

Area of shaft at opening

$Ao = (2 \pi r - b) * t_4$

= 8.11 m^2

Section modulus at opening

$Zo = (\pi * r^2 * t) - (b*r*t)$

= 24.304 m^3

Axial stress,

fa' = P /Ao

= 33995 / 8.11

= 4.19 N/mm^2

Bending Stress,

fb' = M /Zo

= 42852 / 24.304

= 1.76 N/mm^2

Max Stress:

fa' + fb' = 5.95 N/mm^2

Mini Stress:

fa' - fb' = 2.43 N/mm^2

4.3.3 ANALYSIS AND DESIGN OF ANNULAR RAFT FOUNDATION:

PROPORTIONING OF ANNULAR RAFT FOUNDATION:

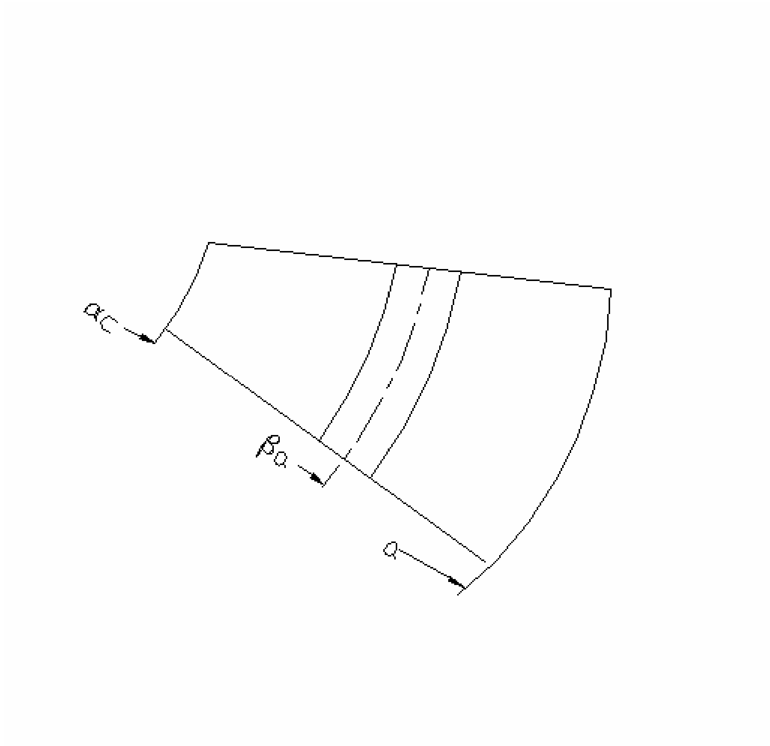


FIGURE: 4.3: PROPORTIONINIG OF RAFT

Assume outer diameter of raft =	17.150	m
Therefore, a =	8.575	m
c/c diameter of shaft =	12.385	m
Thickness of Shaft =	0.215	m
Therefore, βa =	6.193	m
β =	0.722	m

For α = 0.333

$$\beta = \frac{2\,(1-\alpha^3)}{3\,(1-\alpha^2)} = 0.722 \quad m$$

Therefore, αa = 2.856 m

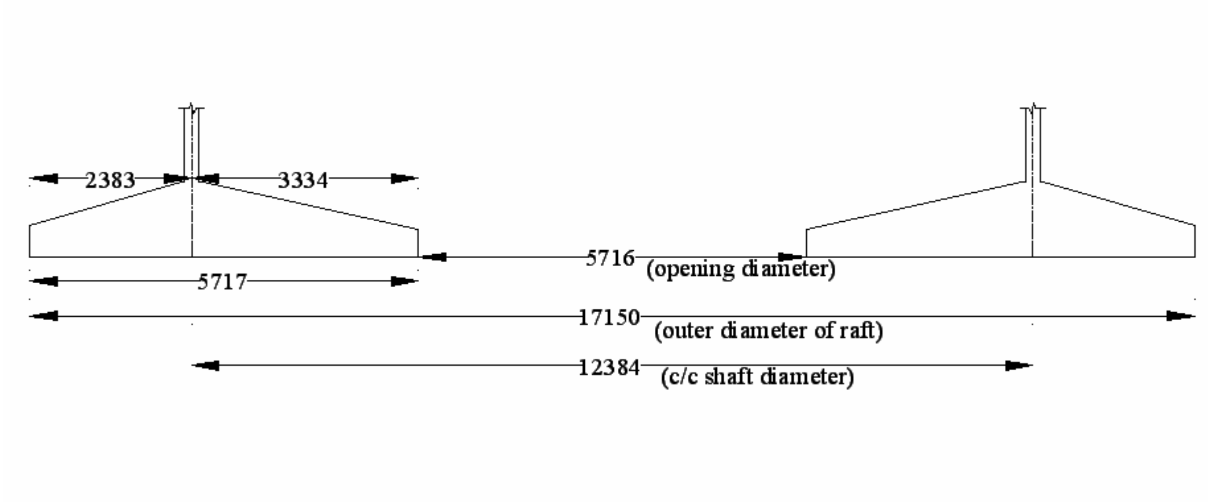


FIGURE: 4.4: CROSS SECTION OF ANNULAR RAFT AS PER IS: 1893 - 1984

Therefore, diameter of opening =	5.711	m
Area of raft footing, A_f =	205.38	m^2
Moment of Inertia of annular raft, I =	4194.19	m^4
section modulus, Z = I / outer radius =	489.12	m^3
Total load from superstructure, P =	33995.00	kN
Live load on top dome, P_1 =	352.00	kN
Total moment at base of raft due to, M =	43000.00	kN m
P / A = p =	165.52	kN/m^2
M / Z = q =	87.91	kN/m^2
Net Safe SBC =	203.00	kN/m^2

PRESSURE CACLUCLATION:

Dead Load + Live Load Case:

Total load for this case, P =	34347.00
Gross pressure at base of raft =	Pressure due to loads from super structure
=	P /A
=	167.23 kN/m^2
Permissible pressure for this case	
is =	Net SBC
Hence, permissible pressure =	203.00 kN/m^2 > 167.23 kN/m^2
Therefore	OK

Dead Load + Live Load Case:

Total load for this case, P =	33995.00 kN
Total Moment at base, M =	43000.00 kN m

For DL + EQ load case 37.5% increase in soil pressure is allowed

Hence, permissible pressure =	279.13 kN/m^2
Pressure at base of raft =	Pressure due to loads from super structure + EQ
=	(P /A) + (M / Z)
=	253.43 kN/m^2 < 279.13 kN/m^2
	Therefore OK.
Pressure at base of raft =	Pressure due to loads from super structure - EQ
=	(P /A) - (M / Z)
=	77.61 kN/m^2 > 0
	Therefore footing is in compression.

DATA:

Outside diameter of raft D1 =	17.150	m
Inside diameter of raft =	5.711	m
Mean Diameter of shaft =	12.385	m
Thickness of shaft =	0.215	m
Outside radius, a =	8.575	m
Mean Radius of shaft =	6.193	m
α =	0.333	m
β =	0.722	m

For the analysis and design of annular raft foundation,

Mr = Radial Moments in Raft

Mt = Tangential Moments in Raft

TABLE: 4.3: SUMMARY OF RADIAL AND TANGENTIAL MOMENTS AS PER IS: 1893 -1984

Radial Distance (m)	Diameter d_1 (m)	f d_1/D_1	Moment due to load		Moment due to moment		Combined Moment	
			Mr (kN.m)	Mt (kN.m)	Mr (kN.m)	Mt (kN.m)	Mr (kN.m)	Mt (kN.m)
8.575	17.150	1.000	0.0360	-58.9414	-0.0152	-39.5947	0.0512	98.5360
8.347	16.695	0.973	-2.6902	-61.1854	0.6886	-41.8643	3.3788	103.0497
8.120	16.240	0.946	-14.2951	-64.6765	-3.0143	-44.9668	17.3094	109.6433
7.892	15.785	0.920	-35.2663	-69.2226	-11.2327	-48.8116	46.4990	118.0342
7.665	15.330	0.893	-66.1554	-74.5935	-24.1062	-53.2926	90.2615	127.8861
7.437	14.875	0.867	-107.5894	-80.5129	-41.8104	-58.2840	149.3998	138.7969
7.210	14.420	0.840	-160.2840	-86.6484	-64.5652	-63.6352	224.8492	150.2837
6.982	13.965	0.814	-225.0600	-92.5993	-92.6447	-69.1645	317.7047	161.7638
6.755	13.510	0.787	-302.8637	-97.8811	-126.3886	-74.6503	429.2522	172.5314
6.527	13.055	0.761	-394.7918	-101.9066	-166.2188	-79.8202	561.0106	181.7268
6.300	12.600	0.734	-502.1230	-103.9606	-212.6591	-84.3367	714.7821	188.2973
6.192	**12.385**	**0.722**	**-559.0072**	**-104.3897**	**-237.0844**	**-86.1263**	**796.0916**	**190.5160**
6.085	12.170	0.709	-523.9893	-90.1926	-224.2066	-82.1115	748.1959	172.3041
5.762	11.524	0.672	-424.4808	-49.9644	-188.2381	-71.0922	612.7188	121.0566
5.439	10.878	0.634	-333.7234	-13.3520	-156.1534	-61.6098	489.8769	74.9618
5.116	10.232	0.596	-252.0578	19.6860	-127.6951	-53.6444	379.7528	73.3304
4.793	9.586	0.559	-179.9200	49.2283	-102.5885	-47.1985	282.5085	96.4267
4.470	8.940	0.521	-117.8800	75.4120	-80.5326	-42.3085	198.4126	117.7206
4.147	8.294	0.483	-66.6993	98.4689	-61.1834	-39.0642	127.8827	137.5331
3.824	7.648	0.446	-27.4220	118.7839	-44.1258	-37.6412	71.5478	156.4252
3.501	7.003	0.408	-1.5238	136.9992	-28.8211	-38.3603	30.3448	175.3595
3.178	6.357	0.370	8.8343	154.1995	-14.5075	-41.7973	23.3419	195.9967
2.855	5.711	0.333	0.3508	172.2634	0.0001	-49.0028	0.3509	221.2661

Analytical procedure is detailed in Appendix B.

DESIGN OF ANNULAR RAFT:

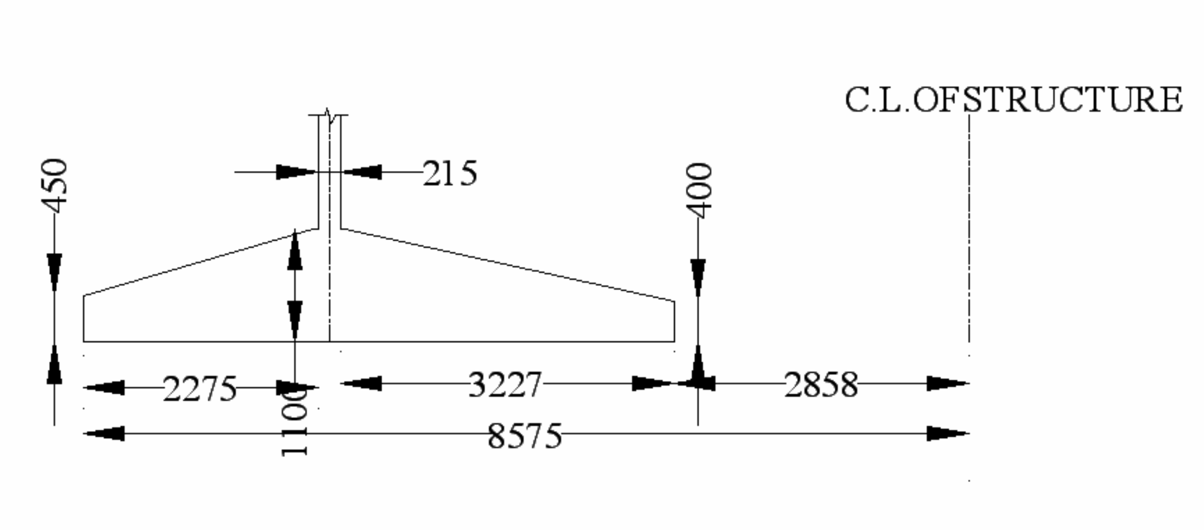


FIGURE: 4.5: HALF CROSS SECTION OF ANNULAR RAFT AS PER IS: 1893 - 1984

Concrete Grade (fck)	=	25	N / mm^2
Steel Grade (fy)	=	415	N / mm^2
Effective Cover	=	75	mm
Thickness at face of shat	=	1.1	m
Thickness at Outer Edge	=	0.45	m
Thickness at Inner Edge	=	0.4	m

TABLE: 4.4: SUMMARY OF RADIAL AND TANGENTIAL REINFORCEMENT AS PER IS: 1893 - 1984

Diameter (m)	Radius (m)	Effective Depth (m)	Mr (kN m)	Ast req Radial (mm^2/m)	Total Ast req (mm^2)	Mt (kN m)	Ast req Tangential (mm^2/m)	Total Ast req (mm^2)
17.150	8.575	0.375	0.051	450.00	24245.24	147.804	1425.30	76792.92
16.695	8.348	0.440	3.379	528.00	27693.01	154.575	1270.39	66630.61
16.240	8.120	0.505	17.309	606.00	30917.80	164.465	1177.70	60085.52
15.785	7.893	0.570	46.499	684.00	33919.59	177.051	1123.25	55702.00
15.330	7.665	0.635	90.262	762.00	36698.39	191.829	1092.43	52612.04
14.875	7.438	0.700	149.400	840.00	39254.20	208.195	1075.54	50261.11
14.420	7.210	0.765	224.849	1062.87	48150.10	225.425	1065.60	48273.50
13.965	6.983	0.830	317.705	1384.20	60727.92	242.646	1057.17	46380.70
13.510	6.755	0.895	429.252	1734.37	73611.70	258.797	1074.00	45583.69
13.055	6.528	0.960	561.011	2113.26	86671.97	272.590	1152.00	47247.54
12.600	6.300	1.025	714.782	2521.75	99821.09	282.446	1230.00	48688.40
12.385	**6.193**	-	-	-	-	-	-	-
12.170	6.085	1.025	748.196	2639.63	100921.57	258.456	1230.00	47026.81
11.524	5.762	0.955	612.719	2320.12	83997.81	181.585	1146.00	41489.92
10.878	5.439	0.885	489.877	2001.69	68407.70	112.443	1062.00	36293.91
10.232	5.116	0.815	379.753	1684.98	54165.45	109.996	978.00	31438.78
9.587	4.793	0.745	282.508	1371.28	41298.85	144.640	894.00	26924.54
8.941	4.470	0.675	198.413	1062.96	29856.32	176.581	946.00	26571.17
8.295	4.147	0.605	127.883	764.38	19918.80	206.300	1233.09	32132.88
7.649	3.824	0.535	71.548	642.00	15427.10	234.638	1585.97	38110.56
7.003	3.502	0.465	30.345	558.00	12276.38	263.039	2045.59	45004.48
6.357	3.179	0.395	23.342	474.00	9466.55	293.995	2691.50	53753.69
5.711	2.856	0.325	0.351	390.00	6997.60	331.899	3692.96	66261.19

4.4 ANALYSIS AND DESIGN AS PER PROPOSED DRAFT CODE FOR IS: 1893 (PART II)

4.4.1 SEISMIC LOAD CALCULATION AS PER PROPOSED DRAFT CODE FOR IS: 1893 (PART II):

Weight of empty container	=	8040 kN	(A)
Weight of staging (shaft)	=	6954 kN	(B)
Weight of water	=	19000 kN	(C)
Volume of water	=	1936.8 m^3	
Mass of water (m)	=	1936799.19 kg	
Weight of empty container + 1/3 weight of staging	=	10358 kN	(D)
Mass of empty container + Mass of 1/3 weight of staging (ms)	=	1055861.37 kg	

Inner diameter of tank = 21.00 m

<u>TANK FULL CONDITION:</u>

Total Seismic Weight:

W = C + D

= 29358 kN

Assume equivalent circular container of same volume and diameter equal to diameter of tank at top level to obtain parameters of spring mass model.

Let h be height of equivalent circular cylinder.

$(\Pi / 4) * D^2 * h = 1936.8\ m^3$

Therefore, h = 5.62 m

For h / D = 0.267

mi / m = 0.3

Therefore,

mi = 581039.76 kg

hi / h = 0.375

mc / m = 0.65

Therefore,

mc = 1258919.47 kg

hc / h = 0.51

Therefore,	Therefore,
hi = 2.11 m	hc = 2.87 m
hi* / h = 1.6	hc* / h = 1.6
Therefore,	Therefore,
hi* = 9.00	hc* = 9.00

Lateral Stiffness:

Ks = $(3 * E * I) / (L^3)$

= $3.28 * 10^8$ N / m

Time Period:

Time period impulsive mode:

Ti = 2 * π * [(mi + ms) / ks] ½

= 0.44 Sec

Time period in convective mode:

Tc = Cc * (D / g) ½

Where,

Cc = 3.7

Tc = 5.41 Sec

Design horizontal seismic coefficient:

For impulsive mode:

(Ah)i = $(Z / 2) * (I / R) * (Sa / g)_i$

Where,

Z = 0.16

I = 1.5

R = 1.8

$(Sa / g)_i$ = 2.5 (For time period of impulsive mode)

Ti = 0.44 Sec

(Ah)i = 0.166

For convective mode:

$(Ah)c$ = $(Z / 2) * (I / R) * (Sa / g)_c$

Where,

Z = 0.16

I = 1.5

R = 1.8

$(Sa / g)_c = 0.58$ (For time period of impulsive mode Ti = 5.41 Sec)

$(Ah)c$ = 0.04

For Sa / g multiplying factor of 1.75 is used to obtain values for 0.5% damping.

Base Shear:

For impulsive mode:

Vi = $(Ah)i * (mi + ms) * g$

= 2665.63 kN

For convective mode:

Vc = $(Ah)c * mc * g$

= 494 kN

Total base shear as per SRSS

V = $(Vi^2 + Vc^2)$ ½

= 2711.02 kN

Base Moment (Overturning Moment):

Impulsive Mode:

Mi* = $(Ah)i * [mi * (hi^* + hs) + (ms * hcg)] * g$

= 107560 kN m

Convective Mode:

Mc* = $(Ah)c * mc * (hc^* + hs) * g$

= 20921 kN m

Total overturning moment:

M* = $[(Mi^*)^2 + (Mc^*)^2]^{1/2}$

= 109576 kN m

Sloshing wave height:

dmax = (Ah)c * R * D / 2

= 0.756 m

<u>TANK EMPTY CONDITION:</u>

For empty tank condition, tank will be considered as a single degree of freedom system.

Total Seismic Weight:

W = D

= 10358 kN

Mass of empty container + 1/3 weight of staging = 1007100 kg

Time Period:

Ti = $2\,\pi * (ms / ks)^{1/2}$

= 0.356 Sec

Design horizontal seismic coefficient:

(Ah)i = (Z / 2) * (I /R) * (Sa / g)i

$(Sa / g)_I$ = 2.5 (For time period of impulsive mode Ti = 0.35 Sec)

(Ah)i = 0.17

Base Shear:

V = Vi = (Ah)i * ms * g

= 1761 kN

Base Moment (Overturning Moment):

Mi* = (Ah)i * ms * hcg * g

= 69113.76 kN m

For this tank, since total base shear for tank full condition (2711 kN) is more than that for tank empty condition (1761 kN) design will be governed by tank full condition.

Even though the IS: 11682 - 1985 has been silent about the construction tolerances in the shafts of elevated water tanks, as a designer it is a good practice to consider such parameter in design. Therefore additional moment due to certain minimum eccentricity has been included here in design.

Moment due to eccentricity

= 0.05 * 27040.46

= 1352.02 kN m

TABLE: 4.5: FINAL DESIGN FORCES AS PER PROPOSED DRAFT CODE IS: 1893 (PART – 2)

Particular	Tank Full	Due to Eccentricity	Design Forces
Total Base Shear (kN)	2711.00	-	2711.00
Total Base Moment (kN m)	109576	1352.02	110928

4.4.2 DESIGN OF SHAFT AS PER PROPOSED DRAFT CODE IS: 1893 (PART II):

Mass of container = 8040 kN

Mass of staging = 6955 kN

Mass of water = 19000 kN

Total Vertical Load at top of footing,

Tank Full P_{full} = 33995 kN

Tank Empty P_{empty} = 14995 kN

Tank Full M_{full} = 42852 kN m

Tank Empty M_{empty} = 22102 kN m

Grade of concrete f_{ck} = 25 N/mm^2

Grade of steel f_y = 415 N/mm^2

Inside Diameter of shaft D_2 = 12.17 m

Thickness of shaft t_4 = 215 mm

Outside Diameter of shaft = 12.60 m

Permissible stress in shaft concrete:

(Clause 8.2.6..1, IS 11682: 1985)

Permissible stress = 0.40 σcv = 10 N/mm^2

Where σcv = 28 day ultimate cubic strength of concrete = 25 N/mm^2

Permissible stress in shaft reinforcement:

(Clause 8.2.6..2, IS 11682: 1985)

Permissible stress = 0.60 σsy = 249 N/mm^2

Where σsy = yield or proof stress in steel = 415 N/mm^2

Area of cross section, As = 8.37 m^2

Moment of Inertia of section, Is = 160.44 m^4

Mean radius of shaft r = 6.1925 m

Radius of gyration = 4.38

Direct compressive stress:

(f' ac) = P_{full} / As

= 4.061 N/mm^2

Vertical Reinforcement in Shaft:

Using Tor steel in two layers, p = 0.25 %

Area of reinforcement required Ast = 537.5 mm^2/m total on both faces

= 268.75 mm^2/m on each face

Provide 12 Dia 300 c/c bars on each face

Ast, pro = 376.8 mm^2/m on each face O.K.

Circumferential Reinforcement in Shaft:

Using Tor steel in two layers, p = 0.2 %

Area of reinforcement required Ast = 430 mm^2/m total on both faces

= 215 mm^2/m on each face

Provide 8 Dia 200 c/c bars on each face

Ast, pro = 251.2 mm^2/m on each face O.K.

r = 6.1925 m

P = 33995 kN

M = 110928 kN m

e = M / P

= 110928 / 33995

= 3.263 m

e / r = 3.263 / 6.1925

= 0.5269

Therefore as per clause 8.2.5.1 (a) the whole section of shaft is not under compression.

Maximum vertical in concrete away from opening = 8.46 N/mm^2

Check for stresses at opening:

As per Clause - 8.2.5 of IS 11682: 1985

Sin β = 0.6 / 6.1925

= 0.0969

Therefore, β = 5.56 degree

Cos β = 0.9953

Size of opening in shaft, b = 1200 mm

As per Clause - 8.2.5 of IS 11682: 1985

e / r = 0.5013 > 0.5269

Therefore the whole section is not under compression and maximum vertical compressive stress.

Maximum vertical in concrete with one opening:

$\sigma cv = 11.53\ N/mm^2 > 10.00\ N/mm^2$

Therefore additional reinforcement shall be provided on near the opening.

Area of shaft at opening

$Ao = (2\ \pi\ r - b) * t_4$

$= 8.11\ m^2$

Section modulus at opening

$Zo = (\pi * r^2 * t) - (b*r*t)$

$= 24.304\ m^3$

Axial stress,

$fa' = P / Ao$

$= 33995 / 8.11$

$= 4.19\ N/mm^2$

Bending Stress,

$fb' = M / Zo$

$= 110928 / 24.304$

$= 4.56\ N/mm^2$

Max Stress:

$fa' + fb' = 8.75\ N/mm^2$

Mini Stress:

$fa' - fb' = -0.37\ N/mm^2$

4.4.3 ANALYSIS AND DESIGN OF ANNULAR RAFT FOUNDATION:

Here the foundation raft is designed as per Proposed Draft Code for the shaft, designed as per IS: 1893 – 1984.

PROPORTIONING OF ANNULAR RAFT FOUNDATION:

Assume outer diameter of raft =	26.500	m
Therefore, a =	13.250	m
c/c diameter of shaft =	12.385	m
Thickness of Shaft =	0.215	m
Therefore, βa =	6.193	m
β =	0.467	m
For α =	0.000	

$$\beta = \frac{2\,(1-\alpha 3)}{3\,(1-\alpha 2)} = 0.667 \text{ m}$$

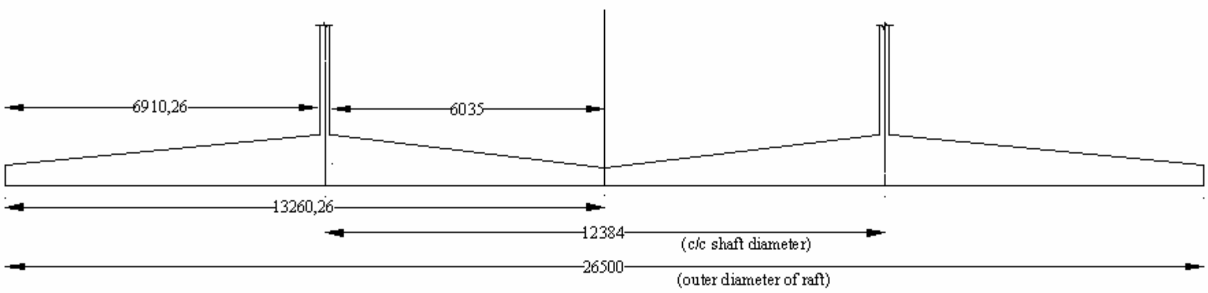


FIGURE: 4.6: CROSS SECTION OF RAFT AS PER PROPOSED DRAFT CODE IS: 1893 (PART II)

Therefore, αa =	0.000	m
Therefore, diameter of opening =	0.000	m
Area of raft footing, Af =	551.55	m^2

Moment of Inertia of annular raft, I = 24207.50 m^4

section modulus, Z = I / outer radius = 1826.98 m^3

Total load from superstructure, P = 33950.00 kN

Live load on top dome,P1 = 351.28 kN

Total moment at base of raft due to, M = 112306.00 kN m

P / A = p = 61.55 kN/m^2

M / Z = q = 61.47 kN/m^2

Net Safe SBC = 203.00 kN/m^2

PRESSURE CACLUCLATION:

Dead Load + Live Load Case:

Total load for this case, P = 34301.28

Gross pressure at base of raft = Pressure due to loads from super structure

= P /A

= 62.19 kN/m^2

Permissible pressure for this case is = Net SBC

Hence, permissible pressure = 203.00 kN/m^2 > 62.19 kN/m^2

Therefore OK

Dead Load + Earthquake Load Case:

Total load for this case, P = 33950.00 kN

Total Moment at base, M = 112306.00 kN m

For DL + EQ load case 37.5% increase in soil pressure is allowed

Hence, permissible pressure = 279.13 kN/m^2

Pressure at base of raft = Pressure due to loads from super structure + EQ

= (P /A) + (M / Z)

= 123.03 kN/m^2 < 279.13 kN/m^2

Therefore OK.

Pressure at base of raft = Pressure due to loads from super structure - EQ

= (P /A) - (M / Z)

= 0.08 kN/m^2 > 0

Therefore footing is in compression.

ANALYSIS OF ANNULAR RAFT:

DATA:

Outside diameter of raft D1 =	26.500	m
Inside diameter of raft =	0.000	m
Mean Diameter of shaft =	12.385	m
Thickness of shaft =	0.215	m
Outside radius, a =	13.250	m
Mean Radius of shaft =	6.193	m
α =	0.000	m
β =	0.467	m

TABLE: 4.6: SUMMARY OF RADIAL AND TANGENTIAL MOMENTS AS PER PROPOSED DRAFT CODE IS: 1893 (Part II)

Radial Distance (m)	Diameter d_1 (m)	f d_1/D_1	Moment due to load		Moment due to moment		Combined Moment	
			Mr (kN.m)	Mt (kN.m)	Mr (kN.m)	Mt (kN.m)	Mr (kN.m)	Mt (kN.m)
13.2500	26.5000	1.0000	-0.3804	-646.9937	-0.0756	-187.6768	0.4559	834.6705
12.5550	25.1100	0.9475	21.0283	-685.8095	14.1140	-209.1035	35.1422	894.9130
11.8600	23.7200	0.8951	16.0325	-734.3874	2.4324	-238.9898	18.4648	973.3772
11.1650	22.3300	0.8426	-17.8445	-792.5551	-35.9594	-277.0331	53.8040	1069.5882
10.4700	20.9400	0.7902	-83.7558	-859.9087	-102.5326	-322.7974	186.2884	1182.7062
9.7750	19.5500	0.7377	-185.7919	-935.6741	-199.6834	-375.5984	385.4753	1311.2726
9.0800	18.1600	0.6853	-329.3762	-1018.4789	-331.2002	-434.2949	660.5764	1452.7738
8.3850	16.7700	0.6328	-521.8810	-1105.9710	-503.0376	-496.9022	1024.9186	1602.8731
7.6900	15.3800	0.5804	-773.6239	-1194.1577	-724.6519	-559.8554	1498.2758	1754.0130
6.9950	13.9900	0.5279	-1099.5473	-1276.2229	-1011.4167	-616.5468	2110.9640	1892.7698
6.3000	12.6000	0.4755	-1522.1891	-1340.3117	-1389.2494	-654.2864	2911.4385	1994.5981
6.1925	12.3850	0.4674	-1598.2372	-1347.4420	-1458.3669	-657.0673	3056.6041	2004.5093
6.0850	12.1700	0.4592	-1582.2429	-1340.0796	-1429.0524	-644.3024	3011.2953	1984.3819
5.4765	10.9530	0.4133	-1496.9871	-1300.8349	-1266.9695	-573.3554	2763.9566	1874.1903
4.8680	9.7360	0.3674	-1420.7057	-1265.7212	-1110.9426	-504.4664	2531.6483	1770.1876
4.2595	8.5190	0.3215	-1353.3985	-1234.7385	-960.2990	-437.4066	2313.6976	1672.1451
3.6510	7.3020	0.2755	-1295.0657	-1207.8869	-814.3657	-371.9474	2109.4314	1579.8343
3.0425	6.0850	0.2296	-1245.7071	-1185.1663	-672.4698	-307.8602	1918.1769	1493.0264
2.4340	4.8680	0.1837	-1205.3228	-1166.5767	-533.9385	-244.9162	1739.2612	1411.4929
1.8255	3.6510	0.1378	-1173.9128	-1152.1181	-398.0987	-182.8869	1572.0115	1335.0049
1.2170	2.4340	0.0918	-1151.4771	-1141.7905	-264.2776	-121.5435	1415.7547	1263.3340
0.6085	1.2170	0.0459	-1138.0156	-1135.5940	-131.8024	-60.6574	1269.8180	1196.2514
0.0000	0.0000	0.0000	0.4473	-2267.5043	0.0000	0.0000	0.4473	2267.5043

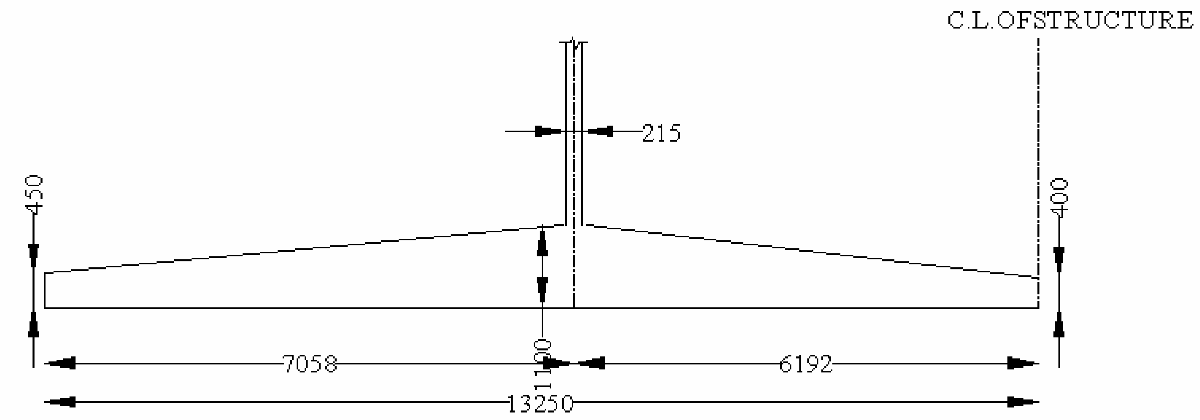


FIGURE: 4.7: HALF CROSS SECTION OF RAFT AS PER PROPOSED DRAFT CODE IS: 1893 (PART II)

Concrete Grade (fck):	25	N/mm^2
Steel Grade (fy):	415	N/mm^2
Effective Cover:	75	mm
Thickness at face of shat :	1.1	m
Thickness at Outer Edge:	0.45	m
Thickness at Inner Edge:	0.4	m

TABLE: 4.7: SUMMARY OF ADIAL AND TANGENTIAL REINFORCEMENT AS PER PROPOSED DRAFT CODE IS: 1893 (Part II)

Radius	Diameter	Effective Depth	Mr	Ast min	Ast	Ast req	Total Ast req	Mt	Ast min	Ast	Ast req	Total Ast req
(m)	(m)	(m)	(kN m)	(mm^2)	(mm^2)	(mm^2)	(mm^2)	(kN m)	(mm^2)	(mm^2)	(mm^2)	(mm^2)
13.250	26.500	0.375	0.456	450.00	4.40	450.00	18731.75	834.670	450.00	8048.89	8048.89	335043.82
12.555	25.110	0.440	35.142	528.00	288.82	528.00	20825.74	894.913	528.00	7354.96	7354.96	290099.39
11.860	23.720	0.505	18.465	606.00	132.22	606.00	22579.13	973.377	606.00	6970.15	6970.15	259702.70
11.165	22.330	0.570	53.804	684.00	341.34	684.00	23991.90	1069.588	684.00	6785.69	6785.69	238013.96
10.470	20.940	0.635	186.288	762.00	1060.88	1060.88	34894.81	1182.706	762.00	6735.27	6735.27	221539.81
9.775	19.550	0.700	385.475	840.00	1991.36	1991.36	61152.93	1311.273	840.00	6774.03	6774.03	208024.11
9.080	18.160	0.765	660.576	918.00	3122.58	3122.58	89073.69	1452.774	918.00	6867.34	6867.34	195895.46
8.385	16.770	0.830	1024.919	996.00	4465.43	4465.43	117629.52	1602.873	996.00	6983.50	6983.50	183961.14
7.690	15.380	0.895	1498.276	1074.00	6053.70	6053.70	146250.39	1754.013	1074.00	7086.99	7086.99	171213.53
6.995	13.990	0.960	2110.964	1152.00	7951.73	7951.73	174742.79	1892.770	1152.00	7129.82	7129.82	156680.97
6.300	12.600	1.025	2911.438	1230.00	10271.55	10271.55	203294.80	1994.598	1230.00	7036.94	7036.94	139275.28
6.193	12.385	-	-	-	-	-	-	-	-	-	-	-
6.085	12.170	1.025	3011.295	1230.00	10623.84	10623.84	203091.63	1984.382	1230.00	7000.89	7000.89	133833.23
5.477	10.953	0.955	2763.957	1146.00	10465.98	10465.98	180066.49	1874.190	1146.00	7096.80	7096.80	122099.92
4.868	9.736	0.885	2531.648	1062.00	10344.56	10344.56	158202.25	1770.188	1062.00	7233.16	7233.16	110618.70
4.260	8.519	0.815	2313.698	978.00	10266.00	10266.00	137375.59	1672.145	978.00	7419.39	7419.39	99283.47
3.651	7.302	0.745	2109.431	894.00	10239.09	10239.09	117441.86	1579.834	894.00	7668.45	7668.45	87956.73
3.043	6.085	0.675	1918.177	810.00	10276.30	10276.30	98223.96	1493.026	810.00	7998.63	7998.63	76453.31
2.434	4.868	0.605	1739.261	726.00	10395.89	10395.89	79493.57	1411.493	726.00	8436.75	8436.75	64512.79
1.826	3.651	0.535	1572.011	642.00	10625.61	10625.61	60937.65	1335.005	642.00	9023.63	9023.63	51750.30
1.217	2.434	0.465	1415.755	558.00	11010.00	11010.00	42094.72	1263.334	558.00	9824.66	9824.66	37562.78
0.609	1.217	0.395	1269.818	474.00	11625.09	11625.09	22223.22	1196.251	474.00	10951.60	10951.60	20935.72
0.000	0.000	0.325	0.447	390.00	4.98	390.00	0.00	2267.504	390.00	25229.98	25229.98	0.00

4.5 SUMMARY:

➔ For the selected case study the increase in base shear and base moment is 260% in tank full condition and 330% in tank empty condition as per proposed draft code IS: 1893 (Part II), when compared with IS: 1893 – 1984. Available results show that the forces are unjustifiably high in proposed draft code IS: 1893 (Part II). The comparison of analysis results as per both IS: 1893 – 1984 and Proposed draft code IS: 1893 (Part II) are presented in TABLE: 4.8.

TABLE: 4.8: COMPARISON OF SEISMIC FORCES AS PER IS: 1893, 1984 AND PROPOSED DRAFT CODE IS: 1893 -1984

PARTICULAR	IS 1893:1984		IS:1893 (Part-2) Draft Code	
	Tank Full	Tank Empty	Tank Full	Tank Empty
Time Period (sec)	0.815	0.48	T = 0.44 (IM)* = 5.41 (CM)*	T = 0.36 (IM)
Importance Factor I	1.5	1.5	1.5	1.5
Response Reduction Factor	-	-	1.8	1.8
Total Base Shear (kN)	1057	529	2711	1761
Total Base Moment (kN m)	41500	20735	109576	69114

(*IM = Impulsive mode, CM = Convective mode)

➔ The selected water tank of case study is situated in seismic zone III. If the same tank would have been in zone V then the seismic forces on the tank are 2.25 time higher then present forces. With such forces the design of supporting shaft and foundation raft is impractical to implement and may indirectly ban shaft supported elevated water tanks.

➔ Response Reduction Value (R) suggested for OMRF is 3 while suggested T value for shaft supported water tank is 1.8 which gives and impression that shaft supported elevated water tanks are much weaker then OMRF. Therefore suggested R value in the draft code seems to be impractical and the issue is still under discussion by designers.

5 OPENINGS

This chapter outlines the nature of stress distribution in the shaft around the region of different openings such as those for door, inlet and outlet pipe, ventilators etc located at different location in plan as well as elevation. For the purpose of finding the stresses in the region of opening the selected water tank is located at Ajwa, Vadodara. The water tank has a capacity of 1800000 Liters and total height of 41m from the natural ground level. The height of the shaft is 30m from the natural ground level. The water tank is under construction and nearing completion. Details and design of the same have been discussed in the previous chapter.

5.1 NEED OF OPENINGS:

Shaft is a hollow column, which will be of either cylindrical or hyperbolic shape. In normal circumstances supporting shaft is circular in plan. As its thickness is very small compared to its diameter it behaves like a membrane structure. Opening size requirements for each and every opening differ according to the purpose of the opening. However opening size should be as small as possible so as to cause minimum stress variations in the shaft. If this could be achieved then the constant shell thickness can be used over the height of the openings to assist the flow of stresses around the openings. In many cases more than two or three openings are required at the same level. For example the openings required for inlet and outlet pipes of water, in many cases which are required to be placed at more or less same level and some times very near to the foundation raft too. Such openings are common in most of the shafts of water tanks.

5.2 MODELING:

The hollow circular shaft has been modeled using 4 nodded shell elements. The software used for modeling this hollow circular shaft is STAAD-Pro. For the purpose of modeling the aspect ratio is kept as near to one as possible. So the shell elements used in modeling are nearly square. The shaft has uniform thickness throughout the height.

Since the purpose of the study is to determine the stress concentration and stress release in the region of different openings of the shaft only, the present model consist of the shaft of the water tank only. Size of each element has been kept 0.53m X 0.54m, 0.53m in circumferential direction while 0.54 in the direction of height. The sizes of the elements have been derived to achieve the actual sizes of various openings such as door, inlet pipe and outlet pipe and to model nearly the same height of the shaft as at the actual site.

Supporting structure of the water tank (shaft or trestle) is subjected to vertical loads due to self weight, weight of container, water and occasional live load on the tank. Apart of such permanent loads supporting structure is subjected to occasional live loads also. These vertical loads produce axial compressive stresses in the supporting structure. In addition to these vertical loads supporting structure is also subjected to lateral forces due to wind and earthquake. Both these forces have different magnitudes and modes of action, consequently these are to be dealt with separately. Both these forces produce axial forces, tension or compression, as well as shear force in the supporting structure. Magnitude and sign of axial forces produced in the supporting structure depends upon the position of particular member with respect to axis of bending, very similar to distribution of bending stresses. Similarly magnitude of shear force on any particular member or element depends upon its position from neutral axis of a section. This is very similar to distribution of shear stress across a cross section.

The gravity loads, loads of the intze container above the shaft and weight of water, are applied on the shaft directly as a nodal force distributed equally around the shaft periphery.

The lateral force, governing force (seismic force in this case), acts on the center of gravity of the container. Since the container portion is not modeled here the lateral fore is transferred at the level of top of the shaft. Therefore in addition to lateral forces moments, caused due to lever arm of the distance between the centre of gravity of the container and the top of the shaft, will also act on the top of the shaft and such moments are transferred as axial forces, either compressive or tensile, on the shaft. The same concept is explained in FIGURE -

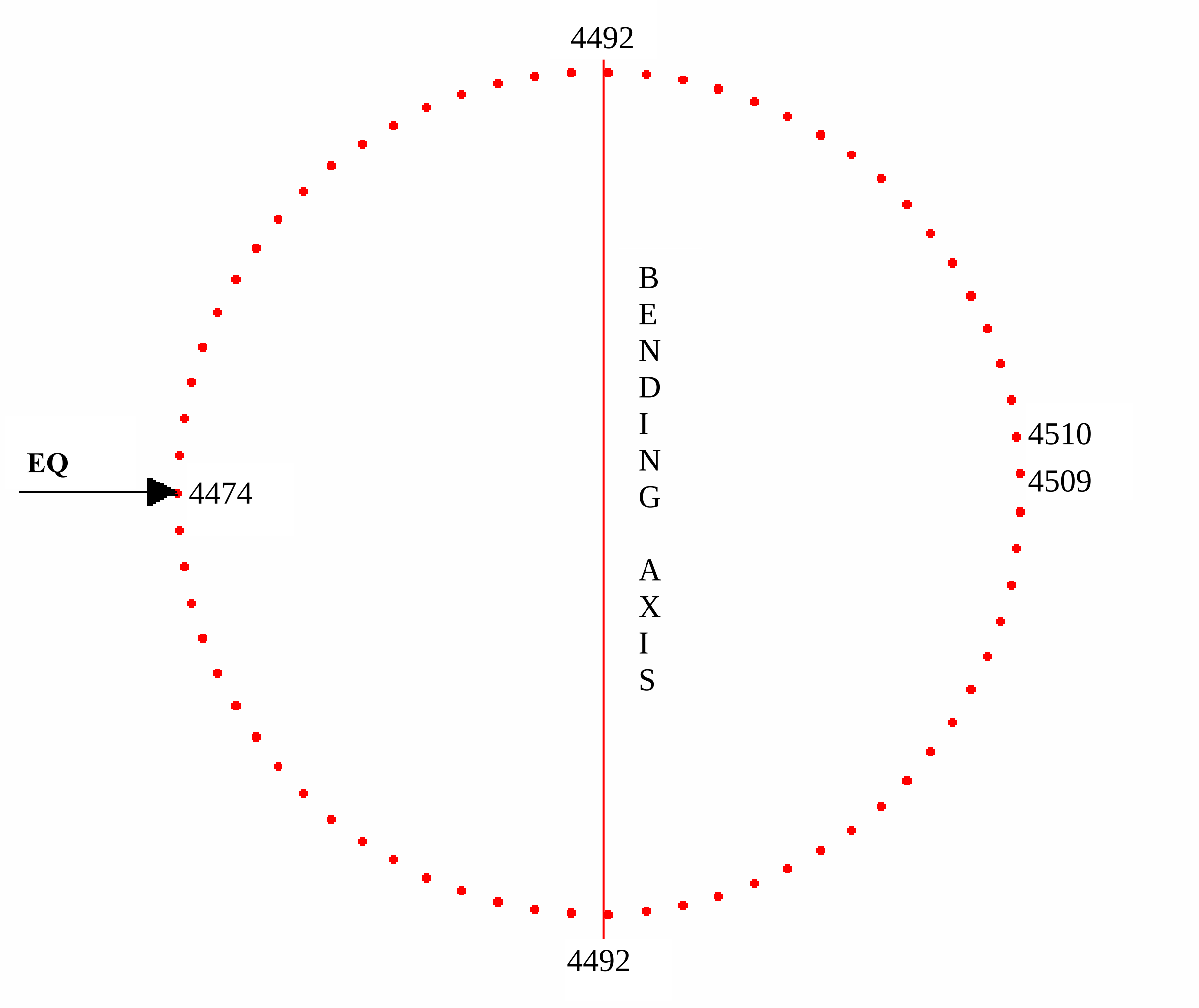


FIGURE 5.1: PLAN AT TOP LEVEL OF SHAFT

Number of nodes	=	71	No
Radius of shaft	=	6.085	m
Vertical load of Container and Water	=	27041	kN
Vertical load at each node	=	380	kN
Total lateral force	=	1057	kN
Distance of centre of gravity from top of the shaft	=	6.4	m
Moment on top of the shaft	=	6065	kN m

TABLE: 5.1: DISTRIBUTION OF SHEAR FORCE ON TOP OF SHAFT

Node No.	θ (Degrees)	θ (Radians)	cosθ	$\cos^2\theta$	Cs	H (kN)
4474	90.000	1.5708	0.0000	0.0000	0.0000	0.00
4475	95.070	1.6593	-0.0884	0.0078	0.0002	0.23
4476	100.141	1.7478	-0.1761	0.0310	0.0009	0.92
4477	105.211	1.8363	-0.2624	0.0688	0.0019	2.05
4478	110.282	1.9248	-0.3466	0.1202	0.0034	3.57
4479	115.352	2.0133	-0.4282	0.1833	0.0052	5.45
4480	120.423	2.1018	-0.5064	0.2564	0.0072	7.63
4481	125.493	2.1903	-0.5806	0.3371	0.0095	10.03
4482	130.563	2.2788	-0.6503	0.4229	0.0119	12.58
4483	135.634	2.3673	-0.7149	0.5111	0.0144	15.20
4484	140.704	2.4558	-0.7739	0.5989	0.0169	17.82
4485	145.775	2.5442	-0.8268	0.6837	0.0193	20.34
4486	150.845	2.6327	-0.8733	0.7627	0.0215	22.69
4487	155.915	2.7212	-0.9129	0.8335	0.0235	24.79
4488	160.986	2.8097	-0.9454	0.8939	0.0252	26.59
4489	166.056	2.8982	-0.9705	0.9419	0.0265	28.02
4490	171.127	2.9867	-0.9880	0.9762	0.0275	29.04
4491	176.197	3.0752	-0.9978	0.9956	0.0280	29.62
4492	181.268	3.1637	-0.9998	0.9995	0.0282	29.73
4493	186.338	3.2522	-0.9939	0.9878	0.0278	29.38
4494	191.408	3.3407	-0.9802	0.9609	0.0271	28.58
4495	196.479	3.4292	-0.9589	0.9195	0.0259	27.35
4496	201.549	3.5177	-0.9301	0.8651	0.0244	25.73
4497	206.620	3.6062	-0.8940	0.7992	0.0225	23.77
4498	211.690	3.6947	-0.8509	0.7240	0.0204	21.54
4499	216.761	3.7832	-0.8011	0.6418	0.0181	19.09
4500	221.831	3.8717	-0.7451	0.5552	0.0156	16.52
4501	226.901	3.9602	-0.6833	0.4668	0.0132	13.89
4502	231.972	4.0487	-0.6160	0.3795	0.0107	11.29
4503	237.042	4.1372	-0.5440	0.2960	0.0083	8.80
4504	242.113	4.2257	-0.4677	0.2188	0.0062	6.51

TABLE: 5.1: CONTINUED

Node No.	θ (Degrees)	θ (Radians)	cosθ	$\cos^2\theta$	Cs	H (kN)
4505	247.183	4.3142	-0.3878	0.1504	0.0042	4.47
4506	252.254	4.4027	-0.3048	0.0929	0.0026	2.76
4507	257.324	4.4912	-0.2194	0.0482	0.0014	1.43
4508	262.394	4.5796	-0.1324	0.0175	0.0005	0.52
4509	267.465	4.6681	-0.0442	0.0020	0.0001	0.06
4510	272.535	4.7566	0.0442	0.0020	0.0001	0.06
4511	277.606	4.8451	0.1324	0.0175	0.0005	0.52
4512	282.676	4.9336	0.2194	0.0482	0.0014	1.43
4513	287.746	5.0221	0.3048	0.0929	0.0026	2.76
4514	292.817	5.1106	0.3878	0.1504	0.0042	4.47
4515	297.887	5.1991	0.4677	0.2188	0.0062	6.51
4516	302.958	5.2876	0.5440	0.2960	0.0083	8.80
4517	308.028	5.3761	0.6160	0.3795	0.0107	11.29
4518	313.099	5.4646	0.6833	0.4668	0.0132	13.89
4519	318.169	5.5531	0.7451	0.5552	0.0156	16.52
4520	323.239	5.6416	0.8011	0.6418	0.0181	19.09
4521	328.310	5.7301	0.8509	0.7240	0.0204	21.54
4522	333.380	5.8186	0.8940	0.7992	0.0225	23.77
4523	338.451	5.9071	0.9301	0.8651	0.0244	25.73
4524	343.521	5.9956	0.9589	0.9195	0.0259	27.35
4525	348.592	6.0841	0.9802	0.9609	0.0271	28.58
4526	353.662	6.1726	0.9939	0.9878	0.0278	29.38
4527	358.732	6.2611	0.9998	0.9995	0.0282	29.73
4528	3.803	0.0664	0.9978	0.9956	0.0280	29.62
4529	8.873	0.1549	0.9880	0.9762	0.0275	29.04
4530	13.944	0.2434	0.9705	0.9419	0.0265	28.02
4531	19.014	0.3319	0.9454	0.8939	0.0252	26.59
4532	24.085	0.4204	0.9129	0.8335	0.0235	24.79
4533	29.155	0.5088	0.8733	0.7627	0.0215	22.69
4534	34.225	0.5973	0.8268	0.6837	0.0193	20.34
4535	39.296	0.6858	0.7739	0.5989	0.0169	17.82

TABLE: 5.1: CONTINUED

Node No.	θ (Degrees)	θ (Radians)	cosθ	$\cos^2\theta$	Cs	H (kN)
4536	44.366	0.7743	0.7149	0.5111	0.0144	15.20
4537	49.437	0.8628	0.6503	0.4229	0.0119	12.58
4538	54.507	0.9513	0.5806	0.3371	0.0095	10.03
4539	59.577	1.0398	0.5064	0.2564	0.0072	7.63
4540	64.648	1.1283	0.4282	0.1833	0.0052	5.45
4541	69.718	1.2168	0.3466	0.1202	0.0034	3.57
4542	74.789	1.3053	0.2624	0.0688	0.0019	2.05
4543	79.859	1.3938	0.1761	0.0310	0.0009	0.92
4544	84.930	1.4823	0.0884	0.0078	0.0002	0.23
				35.5000	**1.0000**	**1056.00**

Cs = Shear Force at Node / Total Shear

H = Cs * Total Shear

TABLE: 5.2: DISTRIBUTION OF AXIAL FORCE DUE TO MOMENTS ON TOP OF SHAFT

Node No.	θ (Degrees)	θ (Radians)	sinθ	$Sin^2θ$	C_a	F_a (kN)	F_b (kN)	$F_a + F_b$ (kN)
4474	90.000	1.571	1.000	1.000	0.028	380.14	-31.29	348.85
4475	95.070	1.659	0.996	0.992	0.028	380.14	-31.16	348.98
4476	100.141	1.748	0.984	0.969	0.027	380.14	-30.80	349.34
4477	105.211	1.836	0.965	0.931	0.026	380.14	-30.19	349.95
4478	110.282	1.925	0.938	0.880	0.025	380.14	-29.35	350.79
4479	115.352	2.013	0.904	0.817	0.023	380.14	-28.27	351.87
4480	120.423	2.102	0.862	0.744	0.021	380.14	-26.98	353.16
4481	125.493	2.190	0.814	0.663	0.019	380.14	-25.47	354.67
4482	130.563	2.279	0.760	0.577	0.016	380.14	-23.77	356.37
4483	135.634	2.367	0.699	0.489	0.014	380.14	-21.88	358.26
4484	140.704	2.456	0.633	0.401	0.011	380.14	-19.81	360.33
4485	145.775	2.544	0.562	0.316	0.009	380.14	-17.60	362.54
4486	150.845	2.633	0.487	0.237	0.007	380.14	-15.24	364.90
4487	155.915	2.721	0.408	0.167	0.005	380.14	-12.77	367.37
4488	160.986	2.810	0.326	0.106	0.003	380.14	-10.19	369.95
4489	166.056	2.898	0.241	0.058	0.002	380.14	-7.54	372.60
4490	171.127	2.987	0.154	0.024	0.001	380.14	-4.83	375.31
4491	176.197	3.075	0.066	0.004	0.000	380.14	-2.08	378.07
4492	181.268	3.164	-0.022	0.000	0.000	380.14	0.69	380.83
4493	186.338	3.252	-0.110	0.012	0.000	380.14	3.45	383.59
4494	191.408	3.341	-0.198	0.039	0.001	380.14	6.19	386.33
4495	196.479	3.429	-0.284	0.080	0.002	380.14	8.87	389.02
4496	201.549	3.518	-0.367	0.135	0.004	380.14	11.49	391.63
4497	206.620	3.606	-0.448	0.201	0.006	380.14	14.02	394.16
4498	211.690	3.695	-0.525	0.276	0.008	380.14	16.44	396.58
4499	216.761	3.783	-0.598	0.358	0.010	380.14	18.72	398.86
4500	221.831	3.872	-0.667	0.445	0.013	380.14	20.87	401.01
4501	226.901	3.960	-0.730	0.533	0.015	380.14	22.84	402.99
4502	231.972	4.049	-0.788	0.620	0.017	380.14	24.64	404.79
4503	237.042	4.137	-0.839	0.704	0.020	380.14	26.25	406.39
4504	242.113	4.226	-0.884	0.781	0.022	380.14	27.65	407.79

TABLE: 5.2: CONTINUED

Node No.	θ (Degrees)	θ (Radians)	sinθ	$Sin^2θ$	C_a	F_a (kN)	F_b (kN)	$F_a + F_b$ (kN)
4505	247.183	4.314	-0.922	0.850	0.024	380.14	28.84	408.98
4506	252.254	4.403	-0.952	0.907	0.026	380.14	29.80	409.94
4507	257.324	4.491	-0.976	0.952	0.027	380.14	30.52	410.66
4508	262.394	4.580	-0.991	0.982	0.028	380.14	31.01	411.15
4509	267.465	4.668	-0.999	0.998	0.028	380.14	31.26	411.40
4510	272.535	4.757	-0.999	0.998	0.028	380.14	31.26	411.40
4511	277.606	4.845	-0.991	0.982	0.028	380.14	31.01	411.15
4512	282.676	4.934	-0.976	0.952	0.027	380.14	30.52	410.66
4513	287.746	5.022	-0.952	0.907	0.026	380.14	29.80	409.94
4514	292.817	5.111	-0.922	0.850	0.024	380.14	28.84	408.98
4515	297.887	5.199	-0.884	0.781	0.022	380.14	27.65	407.79
4516	302.958	5.288	-0.839	0.704	0.020	380.14	26.25	406.39
4517	308.028	5.376	-0.788	0.620	0.017	380.14	24.64	404.79
4518	313.099	5.465	-0.730	0.533	0.015	380.14	22.84	402.99
4519	318.169	5.553	-0.667	0.445	0.013	380.14	20.87	401.01
4520	323.239	5.642	-0.598	0.358	0.010	380.14	18.72	398.86
4521	328.310	5.730	-0.525	0.276	0.008	380.14	16.44	396.58
4522	333.380	5.819	-0.448	0.201	0.006	380.14	14.02	394.16
4523	338.451	5.907	-0.367	0.135	0.004	380.14	11.49	391.63
4524	343.521	5.996	-0.284	0.080	0.002	380.14	8.87	389.02
4525	348.592	6.084	-0.198	0.039	0.001	380.14	6.19	386.33
4526	353.662	6.173	-0.110	0.012	0.000	380.14	3.45	383.59
4527	358.732	6.261	-0.022	0.000	0.000	380.14	0.69	380.83
4528	3.803	0.066	0.066	0.004	0.000	380.14	-2.08	378.07
4529	8.873	0.155	0.154	0.024	0.001	380.14	-4.83	375.31
4530	13.944	0.243	0.241	0.058	0.002	380.14	-7.54	372.60
4531	19.014	0.332	0.326	0.106	0.003	380.14	-10.19	369.95
4532	24.085	0.420	0.408	0.167	0.005	380.14	-12.77	367.37
4533	29.155	0.509	0.487	0.237	0.007	380.14	-15.24	364.90
4534	34.225	0.597	0.562	0.316	0.009	380.14	-17.60	362.54
4535	39.296	0.686	0.633	0.401	0.011	380.14	-19.81	360.33

TABLE: 5.2: CONTINUED

Node No.	θ (Degrees)	θ (Radians)	sinθ	$Sin^2θ$	C_a	F_a (kN)	F_b (kN)	$F_a + F_b$ (kN)
4536	44.366	0.774	0.699	0.489	0.014	380.14	-21.88	358.26
4537	49.437	0.863	0.760	0.577	0.016	380.14	-23.77	356.37
4538	54.507	0.951	0.814	0.663	0.019	380.14	-25.47	354.67
4539	59.577	1.040	0.862	0.744	0.021	380.14	-26.98	353.16
4540	64.648	1.128	0.904	0.817	0.023	380.14	-28.27	351.87
4541	69.718	1.217	0.938	0.880	0.025	380.14	-29.35	350.79
4542	74.789	1.305	0.965	0.931	0.026	380.14	-30.19	349.95
4543	79.859	1.394	0.984	0.969	0.027	380.14	-30.80	349.34
4544	84.930	1.482	0.996	0.992	0.028	380.14	-31.16	348.98
				35.500	1.000	26990.00	0.00	26990.00

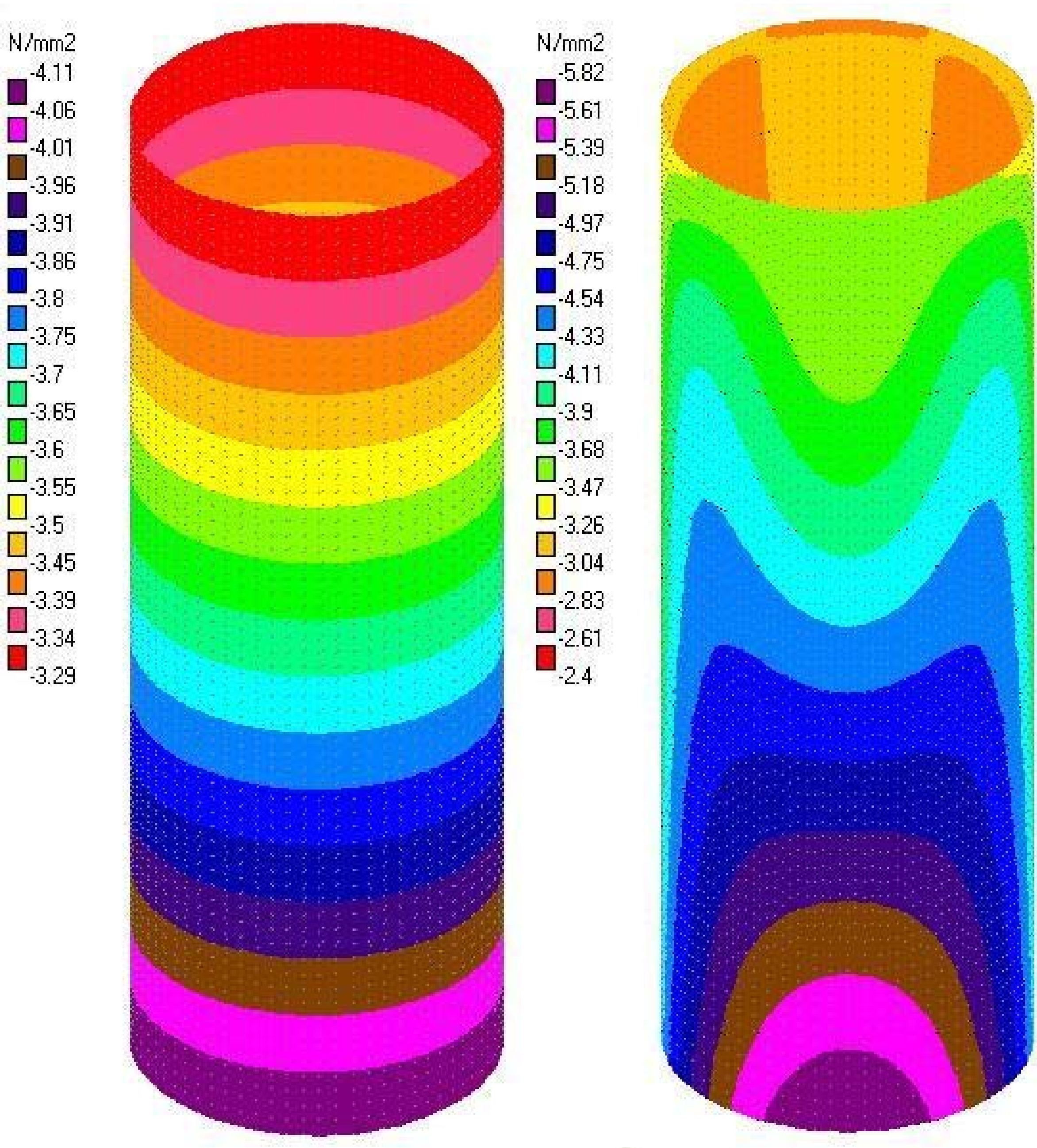
Ca = Axial Force at Node / Total Axial Force

F_b = Ca * Total Axial Force

5.3 STRESSES IN SHAFT WITHOUT OPENING:

Stresses in the shaft without any opening and for the load of self weight of shaft plus vertical load of container including water are shown in FIGURE: 5.2 (a). The stresses are checked with the stresses established by manual calculations. The variation of stresses is 0.02%, which is fairly acceptable. FIGURE: 5.3 (a) represent stress levels around the probable door opening region for same type of loading.

Similarly stresses for the case of self weight of shaft plus vertical load of container including water and lateral load (earthquake load) are shown in FIGURE: 5.2 (b). The stresses are similar to the stresses established by manual calculations. The variation of stresses is 0.01%, which is fairly acceptable. FIGURE: 5.3 (b) represent stress levels around the probable door opening region for same type of loading.



(a) **(b)**

(a) SELF WEIGHT + VERTICAL LOAD

(b) SELF WEIGHT + VERTICAL LOAD + EARTHQUAKE LOAD

FIGURE: 5.2: STRESSES IN SHFT WITHOUT OPENING

3.95	3.95	3.95	3.95	3.95	3.95	3.95	3.95	3.95	3.95
3.97	3.97	3.97	3.97	3.97	3.97	3.97	3.97	3.97	3.97
3.98	3.98	3.98	3.98	3.98	3.98	3.98	3.98	3.98	3.98
3.99	3.99	3.99	3.99	3.99	3.99	3.99	3.99	3.99	3.99
4.01	4.01	4.01	4.01	4.01	4.01	4.01	4.01	4.01	4.01
4.02	4.02	4.02	4.02	4.02	4.02	4.02	4.02	4.02	4.02
4.03	4.03	4.03	4.03	4.03	4.03	4.03	4.03	4.03	4.03
4.05	4.05	4.05	4.05	4.05	4.05	4.05	4.05	4.05	4.05
4.06	4.06	4.06	4.06	4.06	4.06	4.06	4.06	4.06	4.06
4.07	4.07	4.07	4.07	4.07	4.07	4.07	4.07	4.07	4.07
4.09	4.09	4.09	4.09	4.09	4.09	4.09	4.09	4.09	4.09
4.10	4.10	4.10	4.10	4.10	4.10	4.10	4.10	4.10	4.10
4.11	4.11	4.11	4.11	4.11	4.11	4.11	4.11	4.11	4.11

5.23	5.27	5.30	5.31	5.32	5.32	5.31	5.30	5.27	5.23
5.27	5.31	5.34	5.36	5.36	5.36	5.36	5.34	5.31	5.27
5.31	5.35	5.38	5.40	5.41	5.41	5.40	5.38	5.35	5.31
5.34	5.39	5.42	5.44	5.45	5.45	5.44	5.42	5.39	5.34
5.38	5.42	5.46	5.48	5.49	5.49	5.48	5.46	5.42	5.38
5.42	5.46	5.50	5.52	5.53	5.53	5.52	5.50	5.46	5.42
5.45	5.50	5.54	5.56	5.58	5.58	5.56	5.54	5.50	5.45
5.49	5.54	5.58	5.60	5.62	5.62	5.60	5.58	5.54	5.49
5.52	5.58	5.62	5.65	5.66	5.66	5.65	5.62	5.58	5.52
5.56	5.62	5.66	5.69	5.70	5.70	5.69	5.66	5.62	5.56
5.60	5.65	5.70	5.73	5.74	5.74	5.73	5.70	5.65	5.60
5.63	5.69	5.74	5.77	5.78	5.78	5.77	5.74	5.69	5.63
5.67	5.73	5.77	5.81	5.82	5.82	5.81	5.77	5.73	5.67

All values are in N/mm^2

(a) **(b)**

(a) SELF WEIGHT + VERTICAL LOAD

(b) SELF WEIGHT + VERTICAL LOAD + EARTHQUAKE LOAD

FIGURE 5.3: STRESSES IN THE SHAFT AROUND THE PROBABLE DOOR OPENING

FIGURE: 5.3 (a) and (b) represent stresses in the elements which would have been there in the shaft without door opening for the load cases mentioned in the respective figures. In FIGURE: 5.3 (a) and (b) the portion with doted line represents the probable door opening region, where in the model the door opening is provided for further study of nature of stress distribution.

5.4 EFFECTS OF STRESS CONCENTRATION AROUND DOOR OPENING:

As discussed earlier various openings are required in shaft type of staging for various purposes.

- Access opening (Door)
- Inlet and Outlet pipe for water
- Ventilations

Out of above mentioned three types of openings, the opening for the door is provided for excess in shaft when staircase is provided inside the shaft and for maintenance of pipes when placed inside the shaft. The door has notable size and is located at plinth level. Around this portion stress levels are high compared to that at higher level of the shaft. Therefore this portion should be given due importance regarding detailing of reinforcement. Below such openings there is stress release in shaft which should also be treated properly. Some times such portion below opening will alter loads transferred to foundation raft affecting the foundation design.

Inlet and outlet openings have variable sizes according to requirements for water distribution system and availability from source for feeding the container. Generally the pipes are provided inside the shaft to provide protection from external atmosphere and to avoid the risk of damage to the pipes. To bring the pipes outside the shaft openings are required. The sizes of such openings may not be large enough to get attraction from the designer. But the locations of such openings are below the ground level and many times very near to the foundation of the shaft. This is the region where max stress concentration is found even without openings. Even though the sizes of the openings are small, because of its location very high stress concentration is found. A part of stress concentration another issue with such openings is of stress release. If these openings are very near to the foundation practically zero stress can be observed very near to the foundation because of stress release below such openings.

For the purpose of air circulation and light number of ventilations are provided in the shaft at various elevations. Such openings also cause stress concentration around the region. Since such openings are located at higher elevations and the sizes of such openings are quite small, such openings have localized effect and can be taken care of.

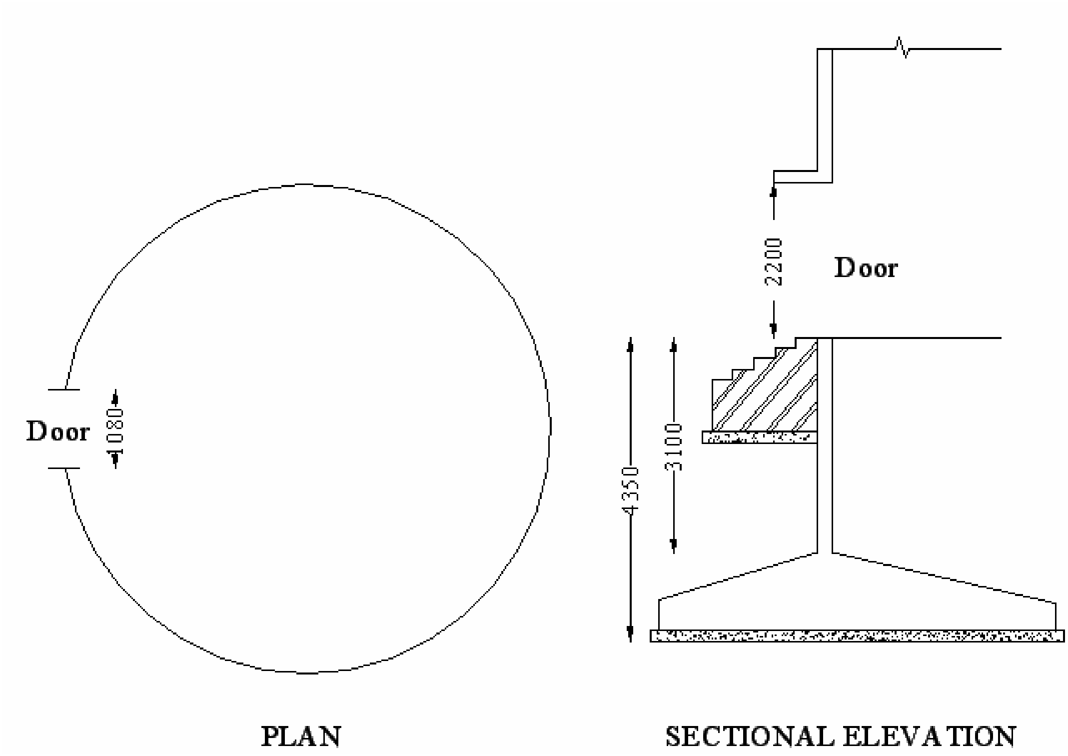
In the shaft of the water tanks the general sizes of the openings are around 1.2m to 1.5m. Considering these sizes and to suit with the model different sizes of the door have been modeled and stress concentration along the sides of the openings and stress release in top and bottom region of the opening has been studied. For the purpose of finding nature of stress distribution three sizes of door openings have been selected. Selected sizes of the openings are,

- 1.08m X 2.12m
- 1.62m X 2.12m
- 2.16m X 2.12m

➢ DOOR SIZE 1.08m X 2.12m:

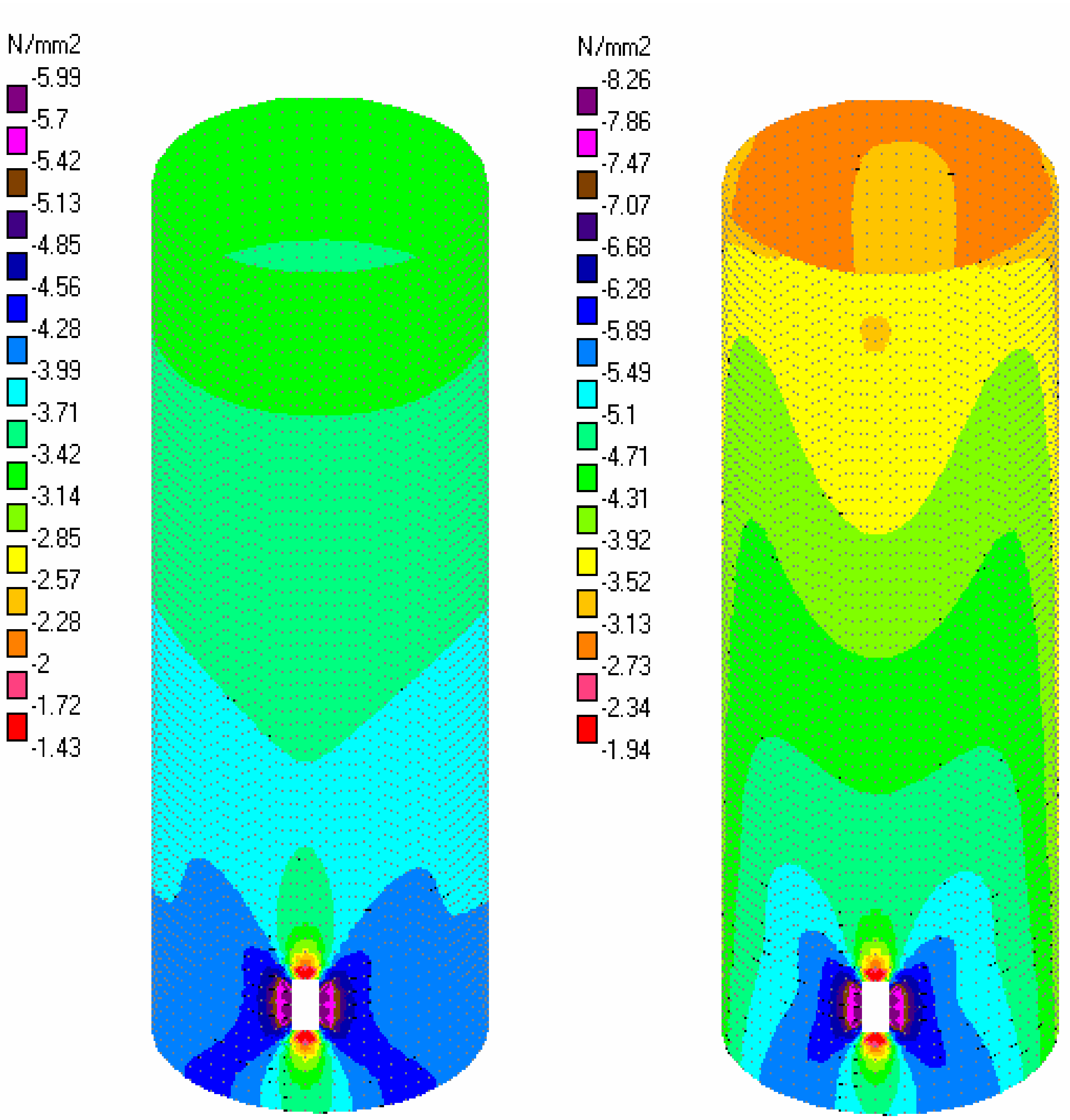
The size of door opening provided in the selected water tank of case study is 1.10m X 2.20m (Horizontal X Vertical). For ease in modeling the door of size 1.08m X 2.12m is provided. FIGURE: 5.4 show location of opening in shaft.

As discussed earlier the shaft model with element size of 0.54m X 0.53m (Horizontal X Vertical) is used and the effect of the door opening on the nature of stress distribution is studied.



All dimensions are in mm

FIGURE: 5.4: DETAILS OF DOOR OPENING IN SHAFT



(a) **(b)**

(a) SELF WEIGHT + VERTICAL LOAD

(b) SELF WEIGHT + VERTICAL LOAD + EARTHQUAKE LOAD

FIGURE: 5.5: STRESSES IN SHFT WITH 1.08m x 2.12m DOOR OPENING

In FIGURE: 5.5 (a) the variation of stresses around door opening can be seen for the load case when only the self weight of shaft plus vertical load of container including water is applied on the shaft. The maximum stress for this load case has been found in range of 5.99 N/mm^2 around both the sides of the door opening. FIGURE: 5.6 (b) shows the stresses in various elements around door opening for the same load case.

In FIGURE: 5.5 (b) the variation of stresses around door opening can be seen for the load case when the self weight of shaft plus vertical load of container including of water and earthquake load in governing direction is applied on the shaft. The maximum stress in this case has been found in range of 8.26 N/mm^2 around both the sides of the door opening. FIGURE: 5.7 (b) shows the stresses in various elements around door opening for the same load case.

3.91	3.91	3.91	3.91	3.91	3.91	3.91	3.91	3.91	3.91	3.91	3.91	3.91	3.91
3.93	3.93	3.93	3.93	3.93	3.93	3.93	3.93	3.93	3.93	3.93	3.93	3.93	3.93
3.94	3.94	3.94	3.94	3.94	3.94	3.94	3.94	3.94	3.94	3.94	3.94	3.94	3.94
3.95	3.95	3.95	3.95	3.95	3.95	3.95	3.95	3.95	3.95	3.95	3.95	3.95	3.95
3.97	3.97	3.97	3.97	3.97	3.97	3.97	3.97	3.97	3.97	3.97	3.97	3.97	3.97
3.98	3.98	3.98	3.98	3.98	3.98	3.98	3.98	3.98	3.98	3.98	3.98	3.98	3.98
3.99	3.99	3.99	3.99	3.99	3.99	3.99	3.99	3.99	3.99	3.99	3.99	3.99	3.99
4.01	4.01	4.01	4.01	4.01	4.01	4.01	4.01	4.01	4.01	4.01	4.01	4.01	4.01
4.02	4.02	4.02	4.02	4.02	4.02	4.02	4.02	4.02	4.02	4.02	4.02	4.02	4.02
4.03	4.03	4.03	4.03	4.03	4.03	4.03	4.03	4.03	4.03	4.03	4.03	4.03	4.03
4.05	4.05	4.05	4.05	4.05	4.05	4.05	4.05	4.05	4.05	4.05	4.05	4.05	4.05
4.06	4.06	4.06	4.06	4.06	4.06	4.06	4.06	4.06	4.06	4.06	4.06	4.06	4.06
4.07	4.07	4.07	4.07	4.07	4.07	4.07	4.07	4.07	4.07	4.07	4.07	4.07	4.07
4.09	4.09	4.09	4.09	4.09	4.09	4.09	4.09	4.09	4.09	4.09	4.09	4.09	4.09
4.10	4.10	4.10	4.10	4.10	4.10	4.10	4.10	4.10	4.10	4.10	4.10	4.10	4.10
4.11	4.11	4.11	4.11	4.11	4.11	4.11	4.11	4.11	4.11	4.11	4.11	4.11	4.11

4.05	4.04	3.99	3.91	3.71	3.68	3.58	3.58	3.68	3.71	3.91	3.99	4.04	4.05
4.08	4.08	4.05	3.97	3.82	3.63	3.48	3.48	3.63	3.82	3.97	4.05	4.08	4.08
4.10	4.13	4.13	4.06	3.89	3.60	3.33	3.33	3.60	3.89	4.06	4.13	4.13	4.10
4.12	4.16	4.23	4.20	4.00	3.51	2.93	2.93	3.51	4.00	4.20	4.23	4.16	4.12
4.11	4.18	4.34	4.28	3.67	3.67	2.43	2.43	3.67	3.67	4.28	4.34	4.18	4.11
4.10	4.17	4.41	4.56	4.49	4.49	1.41	1.41	4.49	4.49	4.56	4.41	4.17	4.10
4.09	4.15	4.25	4.41	4.74	5.88			5.88	4.74	4.41	4.25	4.15	4.09
4.09	4.14	4.24	4.46	4.94	5.76			5.76	4.94	4.46	4.24	4.14	4.09
4.11	4.15	4.25	4.47	4.95	5.77			5.77	4.95	4.47	4.25	4.15	4.11
4.15	4.20	4.28	4.44	4.78	5.91			5.91	4.78	4.44	4.28	4.20	4.15
4.20	4.25	4.33	4.45	4.60	4.53	1.42	1.42	4.53	4.60	4.45	4.33	4.25	4.20
4.25	4.30	4.35	4.40	4.33	3.72	2.47	2.47	3.72	4.33	4.40	4.35	4.30	4.25
4.29	4.32	4.32	4.26	4.05	3.58	3.01	3.01	3.58	4.05	4.26	4.32	4.32	4.29
4.33	4.32	4.27	4.15	3.92	3.59	3.29	3.29	3.59	3.92	4.15	4.27	4.32	4.33
4.33	4.31	4.23	4.08	3.87	3.63	3.45	3.45	3.63	3.87	4.08	4.23	4.31	4.33
4.33	4.29	4.20	4.05	3.87	3.67	3.54	3.54	3.67	3.87	4.05	4.20	4.29	4.33

All values are in N/mm^2

(a) **(b)**

(a) SHAFT WITHOUT OPENING

(b) SHAFT WITH DOOR OPENING

FIGURE 5.6: STRESSES IN THE SHAFT WITH 1.08m X 2.12m DOOR OPENING FOR SELF WEIGHT OF SHAFT PLUS VERTICAL LOAD

FIGURE: 5.6 (a) shows stresses in various elements of shaft without opening and around the probable door opening region for self weight of shaft plus vertical load of container including water. While FIGURE: 5.6 (b) shows stresses in various elements of shaft around door opening region with door opening of size 1.08m X 2.12m for the same load case.

By comparing the stresses in various elements around opening shown in FIGURE: 5.6 (a) with the stresses in FIGURE: 5.6 (b) one can get idea of increase in stresses around door opening. The percentage increase of stresses on the face of the opening is observed around 47.37%. Noticeable stress concentration is found in the region of ¾ width of the opening. Moving away from the opening stresses are reducing and at a distance twice the width of opening from face of opening the stresses are found similar to the stresses which would have been their without any opening.

5.04	5.09	5.12	5.15	5.17	5.19	5.19	5.19	5.19	5.17	5.15	5.12	5.09	5.04
5.07	5.12	5.16	5.19	5.21	5.23	5.24	5.24	5.23	5.21	5.19	5.16	5.12	5.07
5.10	5.16	5.20	5.23	5.26	5.27	5.28	5.28	5.27	5.26	5.23	5.20	5.16	5.10
5.13	5.19	5.23	5.27	5.30	5.31	5.32	5.32	5.31	5.30	5.27	5.23	5.19	5.13
5.17	5.22	5.27	5.31	5.34	5.36	5.36	5.36	5.36	5.34	5.31	5.27	5.22	5.17
5.20	5.26	5.31	5.35	5.38	5.40	5.41	5.41	5.40	5.38	5.35	5.31	5.26	5.20
5.23	5.29	5.34	5.39	5.42	5.44	5.45	5.45	5.44	5.42	5.39	5.34	5.29	5.23
5.26	5.38	5.38	5.42	5.46	5.48	5.49	5.49	5.48	5.46	5.42	5.38	5.38	5.26
5.29	5.36	5.42	5.46	5.50	5.52	5.53	5.53	5.52	5.50	5.46	5.42	5.36	5.29
5.32	5.39	5.45	5.50	5.54	5.56	5.58	5.58	5.56	5.54	5.50	5.45	5.39	5.32
5.35	5.43	5.49	5.54	5.58	5.60	5.62	5.62	5.60	5.58	5.54	5.49	5.43	5.35
5.38	5.46	5.52	5.58	5.62	5.65	5.66	5.66	5.65	5.62	5.58	5.52	5.46	5.38
5.41	5.49	5.56	5.62	5.66	5.69	5.70	5.70	5.69	5.66	5.62	5.56	5.49	5.41
5.45	5.63	5.60	5.65	5.70	5.73	5.74	5.74	5.73	5.70	5.65	5.60	5.63	5.45
5.48	5.56	5.63	5.69	5.74	5.77	5.78	5.78	5.77	5.74	5.69	5.63	5.56	5.48
5.51	5.59	5.67	5.73	5.77	5.81	5.82	5.82	5.81	5.77	5.73	5.67	5.59	5.51

(a)

5.22	5.25	5.23	5.15	5.00	4.85	4.73	4.73	4.85	5.00	5.15	5.23	5.25	5.22
5.28	5.34	5.33	5.25	5.07	4.83	4.63	4.63	4.83	5.07	5.25	5.33	5.34	5.28
5.33	5.42	5.46	5.40	5.18	4.80	4.44	4.44	4.80	5.18	5.40	5.46	5.42	5.33
5.36	5.48	5.63	5.61	5.36	4.72	3.93	3.93	4.72	5.36	5.61	5.63	5.48	5.36
5.37	5.52	5.70	5.83	5.76	4.98	3.26	3.26	4.98	5.76	5.83	5.70	5.52	5.37
5.37	5.52	5.72	5.94	6.18	6.09	1.91	1.91	6.09	6.18	5.94	5.72	5.52	5.37
5.37	5.51	5.71	5.97	6.46	8.00			8.00	6.46	5.97	5.71	5.51	5.37
5.38	5.51	5.70	6.06	6.75	7.88			7.88	6.75	6.06	5.70	5.51	5.38
5.42	5.55	5.73	6.09	6.79	7.95			7.95	6.79	6.09	5.73	5.55	5.42
5.48	5.63	5.80	6.06	6.56	8.19			8.19	6.56	6.06	5.80	5.63	5.48
5.56	5.72	5.88	6.10	6.36	6.30	1.99	1.99	6.30	6.36	6.10	5.88	5.72	5.56
5.65	5.79	5.93	6.05	6.00	5.20	3.47	3.47	5.20	6.00	6.05	5.93	5.79	5.65
5.72	5.84	5.91	5.89	5.64	5.02	4.24	4.24	5.02	5.64	5.89	5.91	5.84	5.72
5.77	5.86	5.86	5.75	5.49	5.05	4.66	4.66	5.05	5.49	5.75	5.86	5.86	5.77
5.80	5.86	5.82	5.68	5.44	5.13	4.89	4.89	5.13	5.44	5.68	5.82	5.86	5.80
5.82	5.84	5.79	5.65	5.44	5.21	5.04	5.04	5.21	5.44	5.65	5.79	5.84	5.82

All values are in N/mm^2

(b)

(a) SHAFT WITHOUT OPENING

(b) SHAFT WITH DOOR OPENING

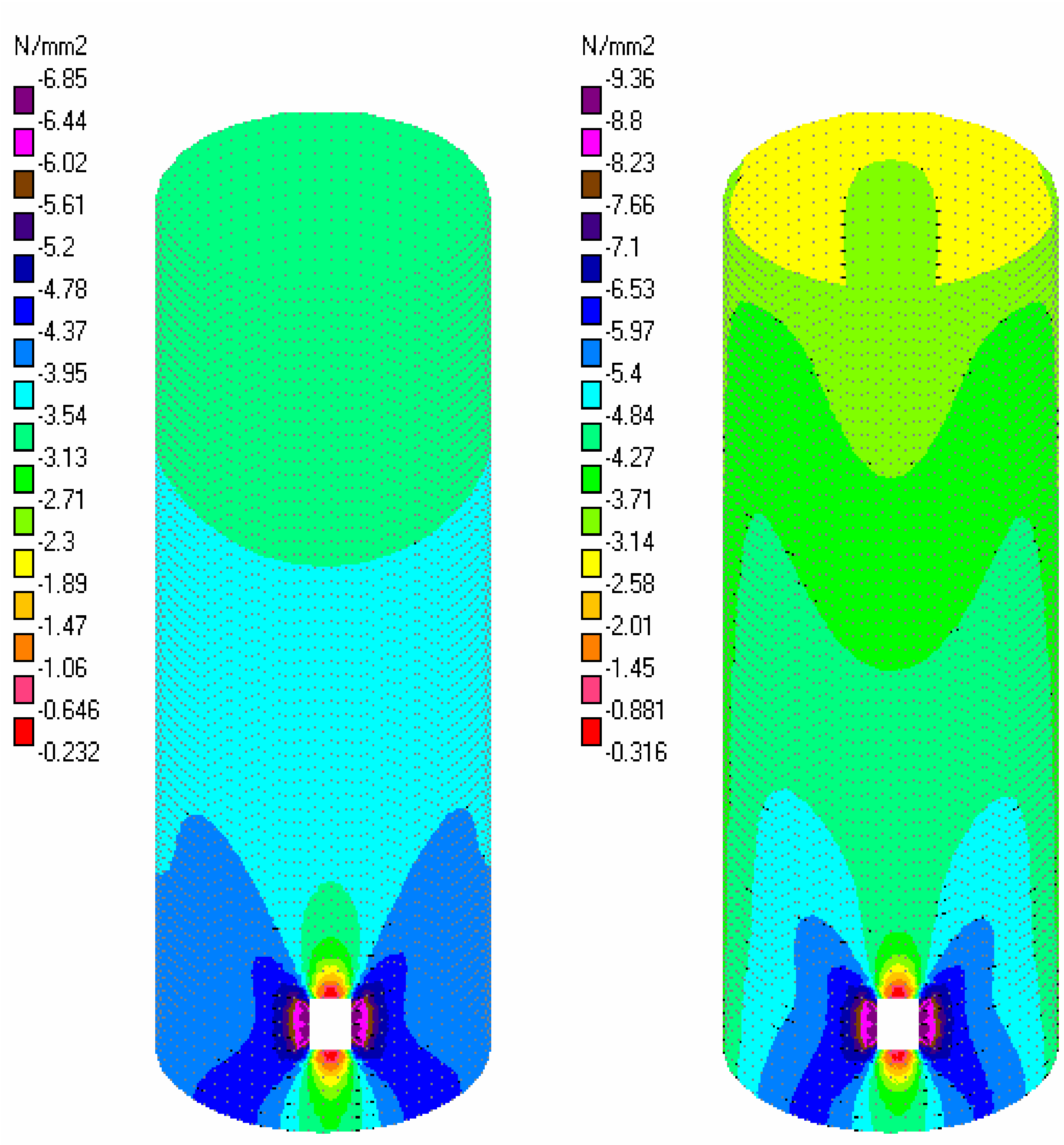
FIGURE 5.7: STRESSES IN THE SHAFT WITH 1.08m X 2.12m DOOR OPENING FOR SELF WEIGHT OF SHAFT, VERTICAL LOAD PLUS EARTHQUAKE LOAD

FIGURE: 5.7 (a) shows stresses in various elements of shaft without opening and around the probable door opening region for self weight of shaft plus vertical load of container including water and earthquake load in governing direction. While FIGURE: 5.7 (b) shows stresses in various elements of shaft around door opening region with door opening of size 1.08m X 2.12m for the same load case.

By comparing the stresses in various elements around opening shown in FIGURE: 5.7 (a) with the stresses in FIGURE: 5.7 (b) one can get idea of increase in stresses around door opening. The percentage increase of stresses on the face of the opening is observed around 47.06%. Noticeable stress concentration is found in the region of ¾ width of the opening. Moving away from the opening stresses are reducing and at a distance twice the width of opening from face of opening the stresses are found similar to the stresses which would have been their without any opening.

- **DOOR SIZE 1.62m X 2.12m:**

For further study and to extend the knowledge of effects of door opening on nature of stress distribution on shaft, the width of the door opening is increased. Size of door opening provided in the model is 1.62m X 2.12m. For the specified size of the door opening nature of stress distribution is studied. FIGURE: 5.8 (a) and (b) symbolize nature of stress distribution around the region of door opening for loading as mentioned in respective figures.



(a) **(b)**

(a) SELF WEIGHT + VERTICAL LOAD

(b) SELF WEIGHT + VERTICAL LOAD + EARTHQUAKE LOAD

FIGURE: 5.8: STRESSES IN SHFT WITH 1.62m x 2.12m DOOR OPENING

In FIGURE: 5.8 (a) the variation of stresses around door opening can be seen for the load case when only the self weight of shaft plus vertical load of container including water is applied on the shaft. The maximum stress for this load case has been found in range of 6.85 N/mm^2 around both the sides of the door opening. FIGURE: 5.9 (b) shows the stresses in various elements around door opening for the same load case.

In FIGURE: 5.8 (b) the variation of stresses around door opening can be seen for the load case when the self weight of shaft plus vertical load of container including of water and earthquake load in governing direction is applied on the shaft. The maximum stress in this case has been found in range of 9.36 N/mm^2 around both the sides of the door opening. FIGURE: 5.10 (b) shows the stresses in various elements around door opening for the same load case.

3.91	3.91	3.91	3.91	3.91	3.91	3.91	3.91	3.91	3.91	3.91	3.91	3.91	3.91	3.91
3.93	3.93	3.93	3.93	3.93	3.93	3.93	3.93	3.93	3.93	3.93	3.93	3.93	3.93	3.93
3.94	3.94	3.94	3.94	3.94	3.94	3.94	3.94	3.94	3.94	3.94	3.94	3.94	3.94	3.94
3.95	3.95	3.95	3.95	3.95	3.95	3.95	3.95	3.95	3.95	3.95	3.95	3.95	3.95	3.95
3.97	3.97	3.97	3.97	3.97	3.97	3.97	3.97	3.97	3.97	3.97	3.97	3.97	3.97	3.97
3.98	3.98	3.98	3.98	3.98	3.98	3.98	3.98	3.98	3.98	3.98	3.98	3.98	3.98	3.98
3.99	3.99	3.99	3.99	3.99	3.99	3.99	3.99	3.99	3.99	3.99	3.99	3.99	3.99	3.99
4.01	4.01	4.01	4.01	4.01	4.01	4.01	4.01	4.01	4.01	4.01	4.01	4.01	4.01	4.01
4.02	4.02	4.02	4.02	4.02	4.02	4.02	4.02	4.02	4.02	4.02	4.02	4.02	4.02	4.02
4.03	4.03	4.03	4.03	4.03	4.03	4.03	4.03	4.03	4.03	4.03	4.03	4.03	4.03	4.03
4.05	4.05	4.05	4.05	4.05	4.05	4.05	4.05	4.05	4.05	4.05	4.05	4.05	4.05	4.05
4.06	4.06	4.06	4.06	4.06	4.06	4.06	4.06	4.06	4.06	4.06	4.06	4.06	4.06	4.06
4.07	4.07	4.07	4.07	4.07	4.07	4.07	4.07	4.07	4.07	4.07	4.07	4.07	4.07	4.07
4.09	4.09	4.09	4.09	4.09	4.09	4.09	4.09	4.09	4.09	4.09	4.09	4.09	4.09	4.09
4.10	4.10	4.10	4.10	4.10	4.10	4.10	4.10	4.10	4.10	4.10	4.10	4.10	4.10	4.10
4.11	4.11	4.11	4.11	4.11	4.11	4.11	4.11	4.11	4.11	4.11	4.11	4.11	4.11	4.11

4.15	4.15	4.09	3.98	3.79	3.57	3.37	3.29	3.37	3.57	3.79	3.98	4.09	4.15	4.15
4.19	4.21	4.19	4.08	3.86	3.55	3.25	3.12	3.25	3.55	3.86	4.08	4.19	4.21	4.19
4.21	4.27	4.29	4.22	3.99	3.56	3.06	2.83	3.06	3.56	3.99	4.22	4.29	4.27	4.21
4.22	4.31	4.39	4.42	4.24	3.66	2.79	2.34	2.79	3.66	4.24	4.42	4.39	4.31	4.22
4.21	4.31	4.45	4.61	4.65	4.05	2.32	1.52	2.32	4.05	4.65	4.61	4.45	4.31	4.21
4.18	4.29	4.45	4.69	5.00	5.14	1.65	0.24	1.65	5.14	5.00	4.69	4.45	4.29	4.18
4.16	4.26	4.43	4.70	5.22	6.85				6.85	5.22	4.70	4.43	4.26	4.16
4.16	4.24	4.41	4.77	5.50	6.61				6.61	5.50	4.77	4.41	4.24	4.16
4.19	4.27	4.43	4.78	5.49	6.58				6.58	5.49	4.78	4.43	4.27	4.19
4.24	4.32	4.78	4.73	5.22	6.75				6.75	5.22	4.73	4.78	4.32	4.24
4.31	4.40	4.54	4.73	4.98	5.05	1.59	0.23	1.59	5.05	4.98	4.73	4.54	4.40	4.31
4.39	4.47	4.57	4.67	4.64	3.99	2.25	1.46	2.25	3.99	4.64	4.67	4.57	4.47	4.39
4.45	4.51	4.55	4.50	4.25	3.60	2.69	2.32	2.69	3.60	4.25	4.50	4.55	4.51	4.45
4.50	4.52	4.48	4.34	4.02	3.50	2.94	2.69	2.94	3.50	4.02	4.34	4.48	4.52	4.50
4.52	4.51	4.42	4.22	3.90	3.48	3.10	2.94	3.10	3.48	3.90	4.22	4.42	4.51	4.52
4.53	4.49	4.36	4.15	3.84	3.50	3.21	3.09	3.21	3.50	3.84	4.15	4.36	4.49	4.53

All values are in N/mm^2

(a) **(b)**

(a) SHAFT WITHOUT OPENING

(b) SHAFT WITH DOOR OPENING

FIGURE 5.9: STRESSES IN THE SHAFT WITH 1.62m x 2.12m DOOR OPENING FOR SELF WEIGHT OF SHAFT PLUS VERTICAL LOAD

FIGURE: 5.9 (a) shows stresses in various elements of shaft without opening and around the probable door opening region for self weight of shaft plus vertical load of container including water. While FIGURE: 5.9 (b) shows stresses in various elements of shaft around door opening region with door opening of size 1.62m X 2.12m for the same load case.

By comparing the stresses in various elements around opening shown in FIGURE: 5.9 (a) with the stresses in FIGURE: 5.9 (b) one can get idea of increase in stresses around door opening. The percentage increase of stresses on the face of the opening is observed around 71.68%. Noticeable stress concentration is found in the region of ¾ width of the opening. Moving away from the opening stresses are reducing and at a distance twice the width of opening from face of opening the stresses are found similar to the stresses which would have been their without any opening.

5.04	5.09	5.12	5.15	5.17	5.19	5.19	5.19	5.19	5.17	5.15	5.12	5.09	5.04
5.07	5.12	5.16	5.19	5.21	5.23	5.24	5.24	5.23	5.21	5.19	5.16	5.12	5.07
5.10	5.16	5.20	5.23	5.26	5.27	5.28	5.28	5.27	5.26	5.23	5.20	5.16	5.10
5.13	5.19	5.23	5.27	5.30	5.31	5.32	5.32	5.31	5.30	5.27	5.23	5.19	5.13
5.17	5.22	5.27	5.31	5.34	5.36	5.36	5.36	5.36	5.34	5.31	5.27	5.22	5.17
5.20	5.26	5.31	5.35	5.38	5.40	5.41	5.41	5.40	5.38	5.35	5.31	5.26	5.20
5.23	5.29	5.34	5.39	5.42	5.44	5.45	5.45	5.44	5.42	5.39	5.34	5.29	5.23
5.26	5.38	5.38	5.42	5.46	5.48	5.49	5.49	5.48	5.46	5.42	5.38	5.38	5.26
5.29	5.36	5.42	5.46	5.50	5.52	5.53	5.53	5.52	5.50	5.46	5.42	5.36	5.29
5.32	5.39	5.45	5.50	5.54	5.56	5.58	5.58	5.56	5.54	5.50	5.45	5.39	5.32
5.35	5.43	5.49	5.54	5.58	5.60	5.62	5.62	5.60	5.58	5.54	5.49	5.43	5.35
5.38	5.46	5.52	5.58	5.62	5.65	5.66	5.66	5.65	5.62	5.58	5.52	5.46	5.38
5.41	5.49	5.56	5.62	5.66	5.69	5.70	5.70	5.69	5.66	5.62	5.56	5.49	5.41
5.45	5.63	5.60	5.65	5.70	5.73	5.74	5.74	5.73	5.70	5.65	5.60	5.63	5.45
5.48	5.56	5.63	5.69	5.74	5.77	5.78	5.78	5.77	5.74	5.69	5.63	5.56	5.48
5.51	5.59	5.67	5.73	5.77	5.81	5.82	5.82	5.81	5.77	5.73	5.67	5.59	5.51

5.37	5.41	5.37	5.24	5.00	4.71	4.45	4.34	4.45	4.71	5.00	5.24	5.37	5.41	5.37
5.43	5.51	5.51	5.40	5.13	4.72	4.31	4.13	4.31	4.72	5.13	5.40	5.51	5.51	5.43
5.47	5.60	5.68	5.62	5.33	4.76	4.08	3.76	4.08	4.76	5.33	5.62	5.68	5.60	5.47
5.50	5.67	5.83	5.89	5.69	4.91	3.73	3.11	3.73	4.91	5.69	5.89	5.83	5.67	5.50
5.49	5.69	5.92	6.17	6.26	5.47	3.13	2.03	3.13	5.47	6.26	6.17	5.92	5.69	5.49
5.47	5.68	5.94	6.31	6.75	6.96	2.23	0.31	2.23	6.96	6.75	6.31	5.94	5.68	5.47
5.46	5.65	5.93	6.35	7.10	9.32				9.32	7.10	6.35	5.93	5.65	5.46
5.47	5.65	5.93	5.47	7.49	9.04				9.04	7.49	5.47	5.93	5.65	5.47
5.52	5.70	5.97	6.50	7.51	9.05				9.05	7.51	6.50	5.97	5.70	5.52
5.60	5.79	6.06	6.45	7.16	9.32				9.32	7.16	6.45	6.06	5.79	5.60
5.72	5.93	6.16	6.48	6.87	7.00	2.21	0.32	2.21	7.00	6.87	6.48	6.16	5.93	5.72
5.83	6.02	6.22	6.42	6.42	5.51	3.15	2.06	3.15	5.51	6.42	6.42	6.22	6.02	5.83
5.93	6.09	6.21	6.20	5.90	5.03	3.79	3.16	3.79	5.03	5.90	6.20	6.21	6.09	5.93
6.00	6.12	6.14	5.99	5.60	4.92	4.16	3.82	4.16	4.92	5.60	5.99	6.14	6.12	6.00
6.06	6.12	6.07	5.85	5.45	4.91	4.40	4.19	4.40	4.91	5.45	5.85	6.07	6.12	6.06
6.09	6.11	6.01	5.77	5.40	4.96	4.57	4.42	4.57	4.96	5.40	5.77	6.01	6.11	6.09

All values are in N/mm^2

(a) **(b)**

(a) SHAFT WITHOUT OPENING

(b) SHAFT WITH DOOR OPENING

FIGURE 5.10: STRESSES IN THE SHAFT WITH 1.62m x 2.12m DOOR OPENIN

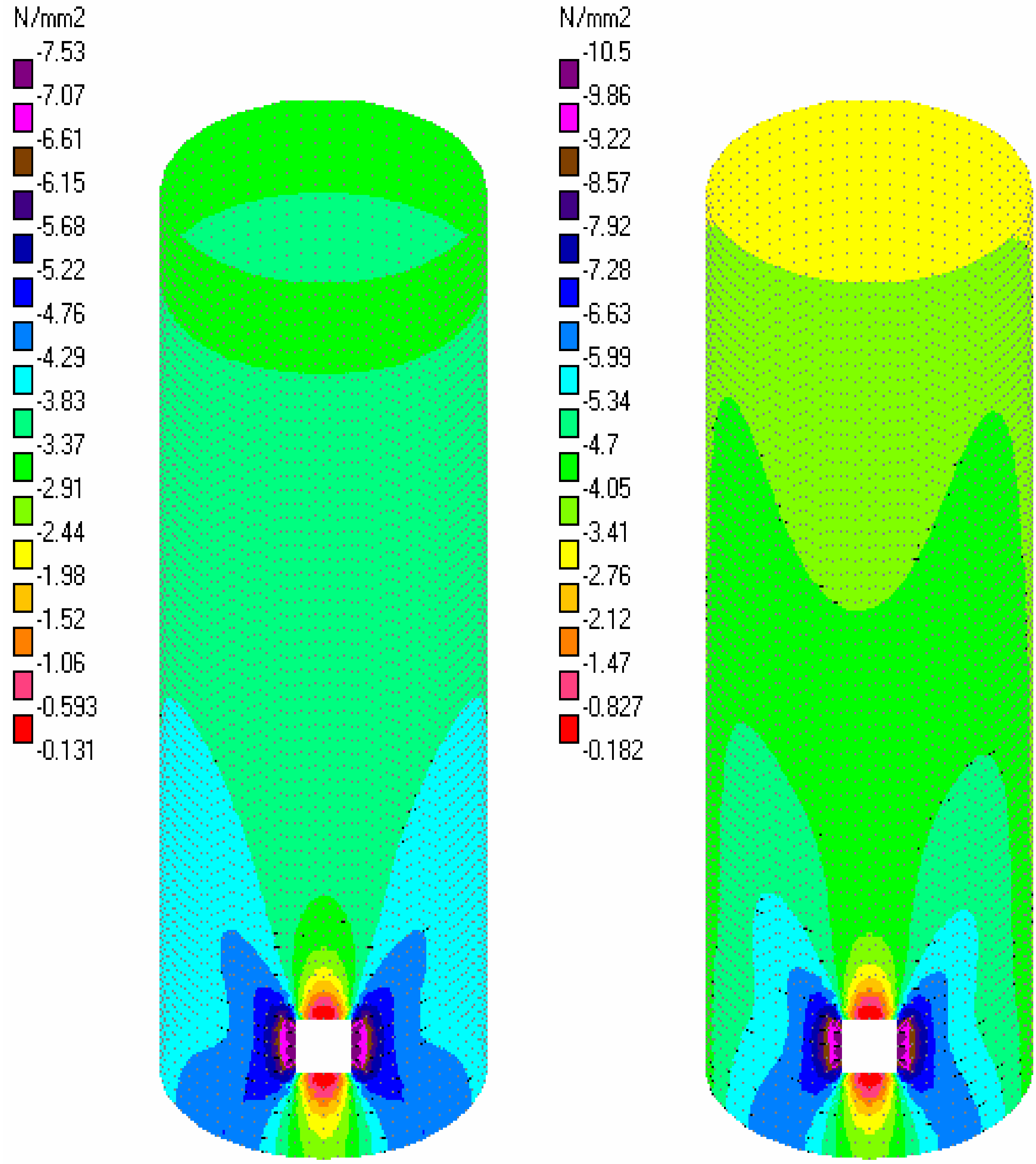
FOR SELF WEIGHT OF SHAFT, VERTICAL LOAD PLUS EARTHQUAKE LOAD

FIGURE: 5.10 (a) shows stresses in various elements of shaft without opening and around the probable door opening region for self weight of shaft plus vertical load of container including water and earthquake load in governing direction. While FIGURE: 5.10 (b) shows stresses in various elements of shaft around door opening region with door opening of size 1.62m X 2.12m for the same load case.

By comparing the stresses in various elements around opening shown in FIGURE: 5.10 (a) with the stresses in FIGURE: 5.10 (b) one can get idea of increase in stresses around door opening. The percentage increase of stresses on the face of the opening is observed around 71.32%. Noticeable stress concentration is found in the region of ¾ width of the opening. Moving away from the opening stresses are reducing and at a distance twice the width of opening from face of opening the stresses are found similar to the stresses which would have been their without any opening.

- **DOOR SIZE 2.16m X 2.12m:**

Generally the size of the door opening is restricted to the maximum width of 1.50m for unaffected flow of stresses in the shaft so that same thickness can be maintaining through out the height of shaft. But for further study and to enhance the knowledge of nature of stress distress distribution on the effects of door opening of shaft the width of the opening is increased. Size of door opening provided in the model is 2.16m X 2.12m. For the specified size of the door opening the effects of the door opening on nature of stress distribution is studied. FIGURE: 5.11 (a) and (b) represents the nature of stress distribution around the region of door opening for the load cases as mentioned in the respective figures.



(a) **(b)**

(a) SELF WEIGHT + VERTICAL LOAD

(b) SELF WEIGHT + VERTICAL LOAD + EARTHQUAKE LOAD

FIGURE: 5.11: STRESSES IN SHFT WITH 2.16m x 2.12m DOOR OPENING

In FIGURE: 5.11 (a) the variation of stresses around door opening can be seen for the load case when only the self weight of shaft plus vertical load of container including water is applied on the shaft. The maximum stress for this load case has been found in range of 7.53 N/mm^2 around both the sides of the door opening. FIGURE: 5.12 (b) shows the stresses in various elements around door opening for the same load case.

In FIGURE: 5.11 (b) the variation of stresses around door opening can be seen for the load case when the self weight of shaft plus vertical load of container including of water and earthquake load in governing direction is applied on the shaft. The maximum stress in this case has been found in range of 10.50 N/mm^2 around both the sides of the door opening. FIGURE: 5.13 (b) shows the stresses in various elements around door opening for the same load case.

3.89	3.89	3.89	3.89	3.89	3.89	3.89	3.89	3.89	3.89	3.89	3.89	3.89	3.89	3.89	3.89
3.90	3.90	3.90	3.90	3.90	3.90	3.90	3.90	3.90	3.90	3.90	3.90	3.90	3.90	3.90	3.90
3.91	3.91	3.91	3.91	3.91	3.91	3.91	3.91	3.91	3.91	3.91	3.91	3.91	3.91	3.91	3.91
3.93	3.93	3.93	3.93	3.93	3.93	3.93	3.93	3.93	3.93	3.93	3.93	3.93	3.93	3.93	3.93
3.94	3.94	3.94	3.94	3.94	3.94	3.94	3.94	3.94	3.94	3.94	3.94	3.94	3.94	3.94	3.94
3.95	3.95	3.95	3.95	3.95	3.95	3.95	3.95	3.95	3.95	3.95	3.95	3.95	3.95	3.95	3.95
3.97	3.97	3.97	3.97	3.97	3.97	3.97	3.97	3.97	3.97	3.97	3.97	3.97	3.97	3.97	3.97
3.98	3.98	3.98	3.98	3.98	3.98	3.98	3.98	3.98	3.98	3.98	3.98	3.98	3.98	3.98	3.98
3.99	3.99	3.99	3.99	3.99	3.99	3.99	3.99	3.99	3.99	3.99	3.99	3.99	3.99	3.99	3.99
4.01	4.01	4.01	4.01	4.01	4.01	4.01	4.01	4.01	4.01	4.01	4.01	4.01	4.01	4.01	4.01
4.02	4.02	4.02	4.02	4.02	4.02	4.02	4.02	4.02	4.02	4.02	4.02	4.02	4.02	4.02	4.02
4.03	4.03	4.03	4.03	4.03	4.03	4.03	4.03	4.03	4.03	4.03	4.03	4.03	4.03	4.03	4.03
4.05	4.05	4.05	4.05	4.05	4.05	4.05	4.05	4.05	4.05	4.05	4.05	4.05	4.05	4.05	4.05
4.06	4.06	4.06	4.06	4.06	4.06	4.06	4.06	4.06	4.06	4.06	4.06	4.06	4.06	4.06	4.06
4.07	4.07	4.07	4.07	4.07	4.07	4.07	4.07	4.07	4.07	4.07	4.07	4.07	4.07	4.07	4.07
4.09	4.09	4.09	4.09	4.09	4.09	4.09	4.09	4.09	4.09	4.09	4.09	4.09	4.09	4.09	4.09
4.10	4.10	4.10	4.10	4.10	4.10	4.10	4.10	4.10	4.10	4.10	4.10	4.10	4.10	4.10	4.10
4.11	4.11	4.11	4.11	4.11	4.11	4.11	4.11	4.11	4.11	4.11	4.11	4.11	4.11	4.11	4.11

4.18	4.14	4.05	3.92	3.74	3.54	3.37	3.27	3.27	3.37	3.54	3.74	3.92	4.05	4.14	4.18
4.23	4.21	4.14	3.99	3.78	3.53	3.30	3.16	3.16	3.30	3.53	3.78	3.99	4.14	4.21	4.23
4.28	4.29	4.24	4.10	3.85	3.53	3.21	3.00	3.00	3.21	3.53	3.85	4.10	4.24	4.29	4.28
4.31	4.36	4.35	4.24	3.97	3.56	3.09	2.77	2.77	3.09	3.56	3.97	4.24	4.35	4.36	4.31
4.33	4.42	4.48	4.43	4.18	3.64	2.93	2.43	2.43	2.93	3.64	4.18	4.43	4.48	4.42	4.33
4.33	4.45	4.58	4.66	4.51	3.87	2.74	1.89	1.89	2.74	3.87	4.51	4.66	4.58	4.45	4.33
4.30	4.44	4.63	4.86	4.99	4.43	2.47	1.09	1.09	2.47	4.43	4.99	4.86	4.63	4.44	4.30
4.27	4.41	4.62	4.94	5.37	5.69	1.95	0.14	0.14	1.95	5.69	5.37	4.94	4.62	4.41	4.27
4.24	4.37	4.59	4.96	5.65	7.70					7.70	5.65	4.96	4.59	4.37	4.24
4.24	4.35	4.58	5.06	5.99	7.37					7.37	5.99	5.06	4.58	4.35	4.24
4.27	4.39	4.61	5.07	5.97	7.30					7.30	5.97	5.07	4.61	4.39	4.27
4.34	4.46	4.67	5.00	7.49	7.30					7.30	7.49	5.00	4.67	4.46	4.34
4.44	4.56	4.74	5.00	5.34	5.52	1.82	0.13	0.13	1.82	5.52	5.34	5.00	4.74	4.56	4.44
4.54	4.66	4.79	4.94	4.95	4.29	2.33	0.99	0.99	2.33	4.29	4.95	4.94	4.79	4.66	4.54
4.63	4.72	4.79	4.76	4.50	3.75	2.56	1.71	1.71	2.56	3.75	4.50	4.76	4.79	4.72	4.63
4.70	4.75	4.73	4.57	4.19	3.52	2.72	2.17	2.17	2.72	3.52	4.19	4.57	4.73	4.75	4.70
4.75	4.75	4.62	4.42	4.00	3.43	2.84	2.45	2.45	2.84	3.43	4.00	4.42	4.62	4.75	4.75
4.77	4.73	4.59	4.32	3.91	3.40	2.92	2.62	2.62	2.92	3.40	3.91	4.32	4.59	4.73	4.77

All values are in N/mm^2

(a) **(b)**

(a) SHAFT WITHOUT OPENING

(b) SHAFT WITH DOOR OPENING

FIGURE 5.12: STRESSES IN THE SHAFT WITH 2.16m x 2.12m DOOR OPENING FOR SELF WEIGHT OF SHAFT PLUS VERTICAL LOAD

FIGURE: 5.12 (a) shows stresses in various elements of shaft without opening and around the probable door opening region for self weight of shaft plus vertical load of container including water. While FIGURE: 5.12 (b) shows stresses in various elements of shaft around door opening region with door opening of size 2.16m X 2.12m for the same load case.

By comparing the stresses in various elements around opening shown in FIGURE: 5.12 (a) with the stresses in FIGURE: 5.12 (b) one can get idea of increase in stresses around door opening. The percentage increase of stresses on the face of the opening is observed around 92.98%. Noticeable stress concentration is found in the region of ¾ width of the opening. Moving away from the opening stresses are reducing and at a distance twice the width of opening from face of opening the stresses are found similar to the stresses which would have been their without any opening.

4.93	4.98	5.02	5.05	5.07	5.09	5.10	5.11	5.11	5.10	5.09	5.07	5.05	5.02	4.98	4.93
4.96	5.01	5.05	5.09	5.11	5.13	5.14	5.15	5.15	5.14	5.13	5.11	5.09	5.05	5.01	4.96
4.99	5.04	5.09	5.12	5.15	5.17	5.19	5.19	5.19	5.19	5.17	5.15	5.12	5.09	5.04	4.99
5.01	5.07	5.12	5.16	5.19	5.21	5.23	5.24	5.24	5.23	5.21	5.19	5.16	5.12	5.07	5.01
5.04	5.10	5.16	5.20	5.23	5.26	5.27	5.28	5.28	5.27	5.26	5.23	5.20	5.16	5.10	5.04
5.07	5.13	5.19	5.23	5.27	5.30	5.31	5.32	5.32	5.31	5.30	5.27	5.23	5.19	5.13	5.07
5.10	5.17	5.22	5.27	5.31	5.34	5.36	5.36	5.36	5.36	5.34	5.31	5.27	5.22	5.17	5.10
5.13	5.20	5.26	5.31	5.35	5.38	5.40	5.41	5.41	5.40	5.38	5.35	5.31	5.26	5.20	5.13
5.16	5.23	5.29	5.34	5.39	5.42	5.44	5.45	5.45	5.44	5.42	5.39	5.34	5.29	5.23	5.16
5.18	5.26	5.38	5.38	5.42	5.46	5.48	5.49	5.49	5.48	5.46	5.42	5.38	5.38	5.26	5.18
5.21	5.29	5.36	5.42	5.46	5.50	5.52	5.53	5.53	5.52	5.50	5.46	5.42	5.36	5.29	5.21
5.24	5.32	5.39	5.45	5.50	5.54	5.56	5.58	5.58	5.56	5.54	5.50	5.45	5.39	5.32	5.24
5.27	5.35	5.43	5.49	5.54	5.58	5.60	5.62	5.62	5.60	5.58	5.54	5.49	5.43	5.35	5.27
5.30	5.38	5.46	5.52	5.58	5.62	5.65	5.66	5.66	5.65	5.62	5.58	5.52	5.46	5.38	5.30
5.33	5.41	5.49	5.56	5.62	5.66	5.69	5.70	5.70	5.69	5.66	5.62	5.56	5.49	5.41	5.33
5.35	5.45	5.63	5.60	5.65	5.70	5.73	5.74	5.74	5.73	5.70	5.65	5.60	5.63	5.45	5.35
5.38	5.48	5.56	5.63	5.69	5.74	5.77	5.78	5.78	5.77	5.74	5.69	5.63	5.56	5.48	5.38
5.41	5.51	5.59	5.67	5.73	5.77	5.81	5.82	5.82	5.81	5.77	5.73	5.67	5.59	5.51	5.41

5.37	5.36	5.28	5.11	4.89	4.63	4.40	4.26	4.26	4.40	4.63	4.89	5.11	5.28	5.36	5.37
5.46	5.48	5.41	5.24	4.97	4.64	4.33	4.13	4.13	4.33	4.64	4.97	5.24	5.41	5.48	5.46
5.53	5.59	5.56	5.40	5.09	4.67	4.23	3.95	3.95	4.23	4.67	5.09	5.40	5.56	5.59	5.53
5.60	5.71	5.74	5.61	5.28	4.73	4.10	3.66	3.66	4.10	4.73	5.28	5.61	5.74	5.71	5.60
5.63	5.81	5.92	5.89	5.57	4.87	3.92	3.22	3.22	3.92	4.87	5.57	5.89	5.92	5.81	5.63
5.64	5.87	6.09	6.22	6.06	5.21	3.68	2.52	2.52	3.68	5.21	6.06	6.22	6.09	5.87	5.64
5.62	5.87	6.18	6.52	6.72	5.99	3.34	1.46	1.46	3.34	5.99	6.72	6.52	6.18	5.87	5.62
5.59	5.84	6.18	6.65	7.27	7.73	2.65	0.19	0.19	2.65	7.73	7.27	6.65	6.18	5.84	5.59
5.56	5.80	6.16	6.71	7.68	10.51					10.51	7.68	6.71	6.16	5.80	5.56
5.57	5.80	6.17	6.86	8.17	10.08					10.08	8.17	6.86	6.17	5.80	5.57
5.64	5.86	6.22	6.91	8.17	10.03					10.03	8.17	6.91	6.22	5.86	5.64
5.75	5.98	6.33	6.84	7.73	10.32					10.32	7.73	6.84	6.33	5.98	5.75
5.89	6.14	6.45	6.86	7.36	7.62	2.52	0.19	0.19	2.52	7.62	7.36	6.86	6.45	6.14	5.89
6.05	6.28	6.54	6.79	6.85	5.96	3.25	1.39	1.39	3.25	5.96	6.85	6.79	6.54	6.28	6.05
6.18	6.39	6.55	6.56	6.24	5.23	3.60	2.43	2.43	3.60	5.23	6.24	6.56	6.55	6.39	6.18
6.29	6.44	6.48	6.32	5.83	4.94	3.85	3.10	3.10	3.85	4.94	5.83	6.32	6.48	6.44	6.29
6.37	6.46	6.40	6.13	5.60	4.84	4.03	3.50	3.50	4.03	4.84	5.60	6.13	6.40	6.46	6.37
6.41	6.45	6.32	6.00	5.48	4.82	4.18	3.77	3.77	4.18	4.82	5.48	6.00	6.32	6.45	6.41

All values are in N/mm^2

(a) **(b)**

(a) SELF WEIGHT + VERTICAL LOAD

(b) SELF WEIGHT + VERTICAL LOAD + EARTHQUAKE LOAD

FIGURE 5.13: STRESSES IN SHFT WITH 2.16m x 2.12m DOOR OPENING

FOR SELF WEIGHT OF SHAFT, VERTICAL LOAD PLUS EARTHQUAKE LOAD

FIGURE: 5.13 (a) shows stresses in various elements of shaft without opening and around the probable door opening region for self weight of shaft plus vertical load of container including water and earthquake load in governing direction. While FIGURE: 5.13 (b) shows stresses in various elements of shaft around door opening region with door opening of size 2.16m X 2.12m for the same load case.

By comparing the stresses in various elements around opening shown in FIGURE: 5.13 (a) with the stresses in FIGURE: 5.13 (b) one can get idea of increase in stresses around door opening. The percentage increase of stresses on the face of the opening is observed around 93.91%. Noticeable stress concentration is found in the region of ¾ width of the opening. Moving away from the opening stresses are reducing and at a distance twice the width of opening from face of opening the stresses are found similar to the stresses which would have been their without any opening.

5.5 EFFECT OF STRESS RELEASE IN SHAFT DUE TO DOOR OPENING ON FOUNDATION:

Up till now we have discussed the subject of stress concentration around the door opening region. Since the door opening is located at plinth level, which is near to the foundation, the other subject matter of discussion is the effects of stress release on foundation of the shaft. The effect of stress release is significant for the permanent loads, self weight of shaft, vertical load of container and water, as these loads are assumed to have effect on the foundation as uniformly distributed load all around the perimeter of the shaft. Therefore the soil reaction is also uniform. Annular raft is preferably positioned such that under the effect of permanent loads the centre of gravity of uniform soil reaction coincides with the shaft location. But now with the effect of released stresses below the door opening the permanent loads are not of uniform nature.

FIGURE: 5.14 (a) represent the stress distribution in shaft at foundation and probable opening region for self weight of shaft plus vertical load of container including water, without considering any opening in the shaft. The elements with doted lines represent the probable region of door opening in the shaft. FIGURE:

5.14 (b) represent the stress distribution in shaft at foundation and opening region for the same type of loading.

Comparing the stresses in FIGURE: 5.14 (a) and (b), it is very clear that because of free surface at opening level the stresses are transferring in alternate direction and therefore stresses at top and bottom region of the opening have tendency to arrive at minimum level. And as we move away from the free surface stresses are again reaching to the normal level.

3.95	3.95	3.95	3.95	3.95	3.95	3.95	3.95	3.95	3.95
3.97	3.97	3.97	3.97	3.97	3.97	3.97	3.97	3.97	3.97
3.98	3.98	3.98	3.98	3.98	3.98	3.98	3.98	3.98	3.98
3.99	3.99	3.99	3.99	3.99	3.99	3.99	3.99	3.99	3.99
4.01	4.01	4.01	4.01	4.01	4.01	4.01	4.01	4.01	4.01
4.02	4.02	4.02	4.02	4.02	4.02	4.02	4.02	4.02	4.02
4.03	4.03	4.03	4.03	4.03	4.03	4.03	4.03	4.03	4.03
4.05	4.05	4.05	4.05	4.05	4.05	4.05	4.05	4.05	4.05
4.06	4.06	4.06	4.06	4.06	4.06	4.06	4.06	4.06	4.06
4.07	4.07	4.07	4.07	4.07	4.07	4.07	4.07	4.07	4.07
4.09	4.09	4.09	4.09	4.09	4.09	4.09	4.09	4.09	4.09
4.10	4.10	4.10	4.10	4.10	4.10	4.10	4.10	4.10	4.10
4.11	4.11	4.11	4.11	4.11	4.11	4.11	4.11	4.11	4.11

4.23	4.20	4.00	3.51	2.93	2.93	3.51	4.00	4.20	4.23
4.34	4.28	3.67	3.67	2.43	2.43	3.67	3.67	4.28	4.34
4.41	4.56	4.49	4.49	1.41	1.41	4.49	4.49	4.56	4.41
4.25	4.41	4.74	5.88			5.88	4.74	4.41	4.25
4.24	4.46	4.94	5.76			5.76	4.94	4.46	4.24
4.25	4.47	4.95	5.77			5.77	4.95	4.47	4.25
4.28	4.44	4.78	5.91			5.91	4.78	4.44	4.28
4.33	4.45	4.60	4.53	1.42	1.42	4.53	4.60	4.45	4.33
4.35	4.40	4.33	3.72	2.47	2.47	3.72	4.33	4.40	4.35
4.32	4.26	4.05	3.58	3.01	3.01	3.58	4.05	4.26	4.32
4.27	4.15	3.92	3.59	3.29	3.29	3.59	3.92	4.15	4.27
4.23	4.08	3.87	3.63	3.45	3.45	3.63	3.87	4.08	4.23
4.20	4.05	3.87	3.67	3.54	3.54	3.67	3.87	4.05	4.20

All values are in N/mm^2

(a) **(b)**

(a) SHAFT WITHOUT OPENING

(b) SHAFT WITH DOOR OPENING

FIGURE: 5.14: STRESSES IN SHFT AT FOUNDATION LEVEL FOR SELF WEIGHT OF SHAFT, WEIGHT OF CONTAINER INCLUDING WATER

In FIGURE: 5.14 (a) and (b) the comparison of stresses at foundation level give an idea about the effect of stress release on foundation of the shaft. In FIGURE: 5.14 (a) (shaft without opening) the stresses are constant at foundation level, while the stresses in FIGURE: 5.14 (b) (shaft with door opening) are varying by maximum of 20.00%. Here sufficient clearance is available between the door and the foundation level. Therefore the variation of stresses vanishes considerably before reaching to the foundation level and does not have significant effect on foundation. But if the door size would have been bigger or the clearance available between the door and the foundation is less then in that case such opening has considerable effect on design of foundation raft.

5.6 REMEDY TO REDUCE STRESS CONCENTRATION AROUND OPENING:

It is quite clear from above discussion that because of door opening the stress concentration is found at least up to the region of ¾ the width of the opening. Further the stress release has also been found in the region of above and below the opening. The studies of load transfer represent the load path around opening for shaft similar to the load path for wind in the wind tunnels. The probable load path for the load around opening region has been presented in FIGURE: 5.15.

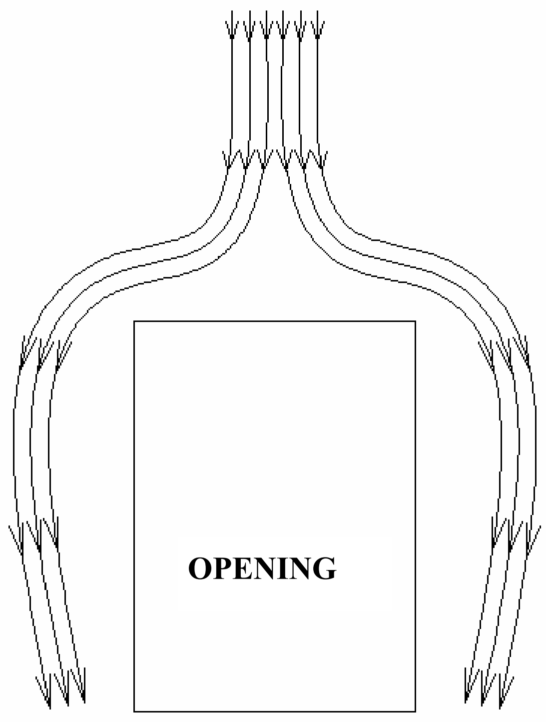
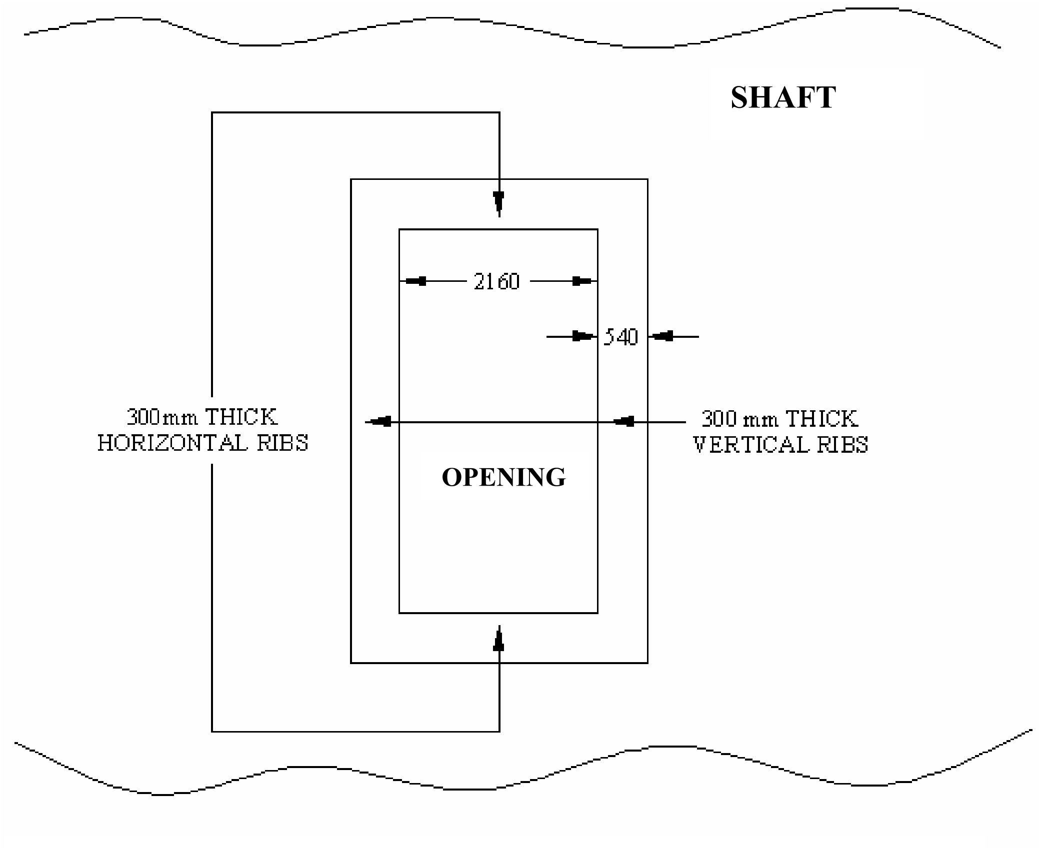


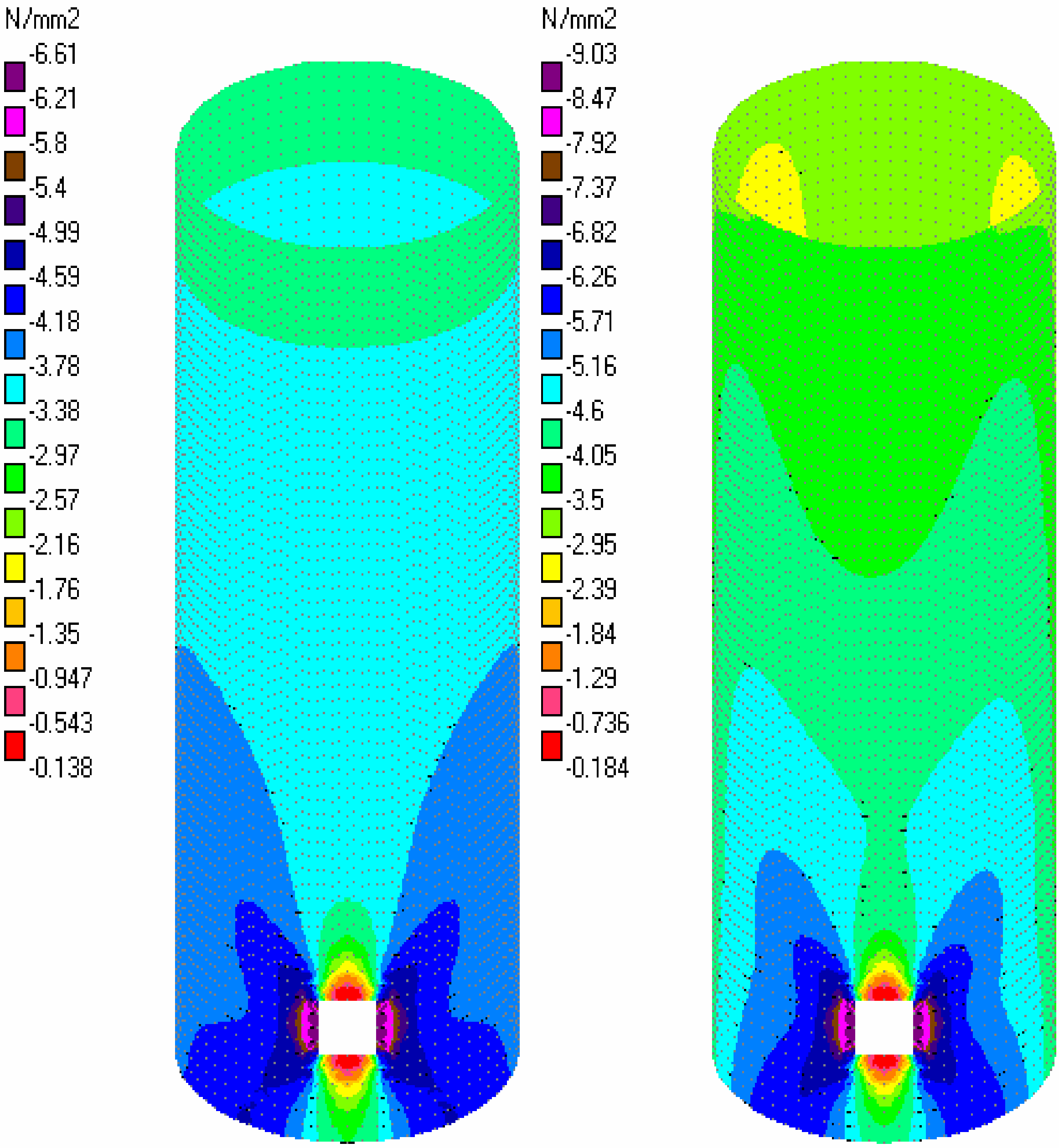
FIGURE: 5.15: LOAD PATH AROUND OPENING

Stresses are concentrated on sides of the opening, especially in the region of ¾ the width of the opening. These high stresses can be dealt with by providing additional reinforcement or by providing additional area around openings in form of vertical and horizontal ribs. Here the later option of increasing the cross section area of shaft has been adopted for the opening size of 2.16m X 2.12m. Vertical and horizontal ribs around the region of opening have been provided by increasing the cross section area for the region of ¼ the width of opening which is shown in FIGURE: 5.16. Increased thickness of these ribs is 300mm. Results are presented in FIGURE: 5.17.



All dimensions are in mm

FIGURE: 5.16: VERTICAL AND HORIZONTAL RIBS AROUND DOOR OPENING



(a) **(b)**

(a) SELF WEIGHT + VERTICAL LOAD WITHOUT OPENING

(b) SELF WEIGHT + VERTICAL LOAD WITH OPENING

FIGURE: 5.17: STRESSES IN SHFT AROUND OPENING AFTER STIFFENING

Comparing the stress values of FIGURE: 5.17 (a) with the FIGURE: 5.11 (a) witch represent the stresses in shaft for same size of opening and same kind of loading that is self weight of shaft plus vertical load of container including water. The maximum stress level around opening after stiffening has found in the region of 6.61 N/mm^2 which was earlier in the range of 7.53 N/mm^2. Therefore stress reduction after stiffening has been found in the range of 16.00%.

Comparing the stress values of FIGURE: 5.17 (b) with the FIGURE: 5.11 (b) witch represent the stresses in shaft for same size of opening and same kind of loading that is self weight of shaft, vertical load of container and water along with lateral load in the governing direction. The maximum stress level around opening after stiffening has found in the region of 9.03 N/mm^2 which was earlier in the range of 10.50 N/mm^2. Therefore stress reduction after stiffening has been found in the range of 16.00%.

5.7 STRESS CONCENTRATION AROUND INLET AND OUTLET OPENINGS:

Elevated water tanks are one of the most important components of any efficient water distribution system. The basic purpose of elevated water tank is to secure continuous water supply with sufficient flow to wide area by gravity. The inlet and outlet pipes of the elevated water tanks are the most important components to serve the purpose efficiently. To make off the water supply requirements the diameter of the inlet and outlet pipes are adjusted from site to site. Therefore the size of the opening for such purpose may vary from place to place. But the location of such openings, in elevation, remains just about the same for a good number of the structures that is below the natural ground level and some times very near to the foundation of the shaft. As discussed earlier, in such cases also the stress concentration increases on the sides of the openings. Add to the stress concentration on the sides of such openings, the fact of stress release above and below is the point of concern for us as a designer. When such openings are located near to the foundation stress release underneath such openings causes practically zero pressure on the foundation. But at the same time the foundation is subjected to upward soil pressure. Therefore such portion of the shaft and footing is subjected to uplift pressure from soil underneath.

The other effect of such stress release is that, the foundation of shaft which is designed for uniformly distributed load neglecting the effect of opening is now subjected to different pattern of loading.

4.03	4.03	4.03	4.03	4.03	4.03	4.03	4.03	4.03	4.03	4.03	4.03	4.03
4.05	4.05	4.05	4.05	4.05	4.05	4.05	4.05	4.05	4.05	4.05	4.05	4.05
4.06	4.06	4.06	4.06	4.06	4.06	4.06	4.06	4.06	4.06	4.06	4.06	4.06
4.07	4.07	4.07	4.07	4.07	4.07	4.07	4.07	4.07	4.07	4.07	4.07	4.07
4.09	4.09	4.09	4.09	4.09	4.09	4.09	4.09	4.09	4.09	4.09	4.09	4.09
4.10	4.10	4.10	4.10	4.10	4.10	4.10	4.10	4.10	4.10	4.10	4.10	4.10
4.11	4.11	4.11	4.11	4.11	4.11	4.11	4.11	4.11	4.11	4.11	4.11	4.11

5.35	5.42	5.47	5.52	5.55	5.57	5.57	5.57	5.55	5.52	5.47	5.42	5.35
5.38	5.45	5.51	5.56	5.59	5.61	5.61	5.61	5.59	5.56	5.51	5.45	5.38
5.42	5.49	5.55	5.59	5.63	5.69	5.70	5.69	5.63	5.59	5.55	5.49	5.42
5.45	5.52	5.59	5.63	5.67	5.69	5.70	5.69	5.67	5.63	5.59	5.52	5.45
5.49	5.56	5.62	5.67	5.71	5.74	5.73	5.74	5.71	5.67	5.62	5.56	5.49
5.52	5.60	5.66	5.72	5.75	5.78	5.79	5.78	5.75	5.72	5.66	5.60	5.52
5.55	5.63	5.70	5.75	5.79	5.81	5.82	5.81	5.79	5.75	5.70	5.63	5.55

All stresses are in N/mm^2

(a) **(b)**

(a) SELF WEIGHT + VERTICAL LOAD

(b) SELF WEIGHT + VERTICAL LOAD + EARTHQUAKE LOAD

FIGURE: 5.18 STRESSES IN SHFT AROUND PROBABLE INET/OUTLET PIPE OPENING

FIGURE: 5.18 (a) & (b) represents the level of stresses in the shaft at foundation level for the respective load cases as shown in figure. The portion shown with doted lines represents the probable region of openings for inlet / outlet openings. FIGURE: 5.19 show location of openings in shaft for inlet and outlet pipes for the selected water tank of case study.

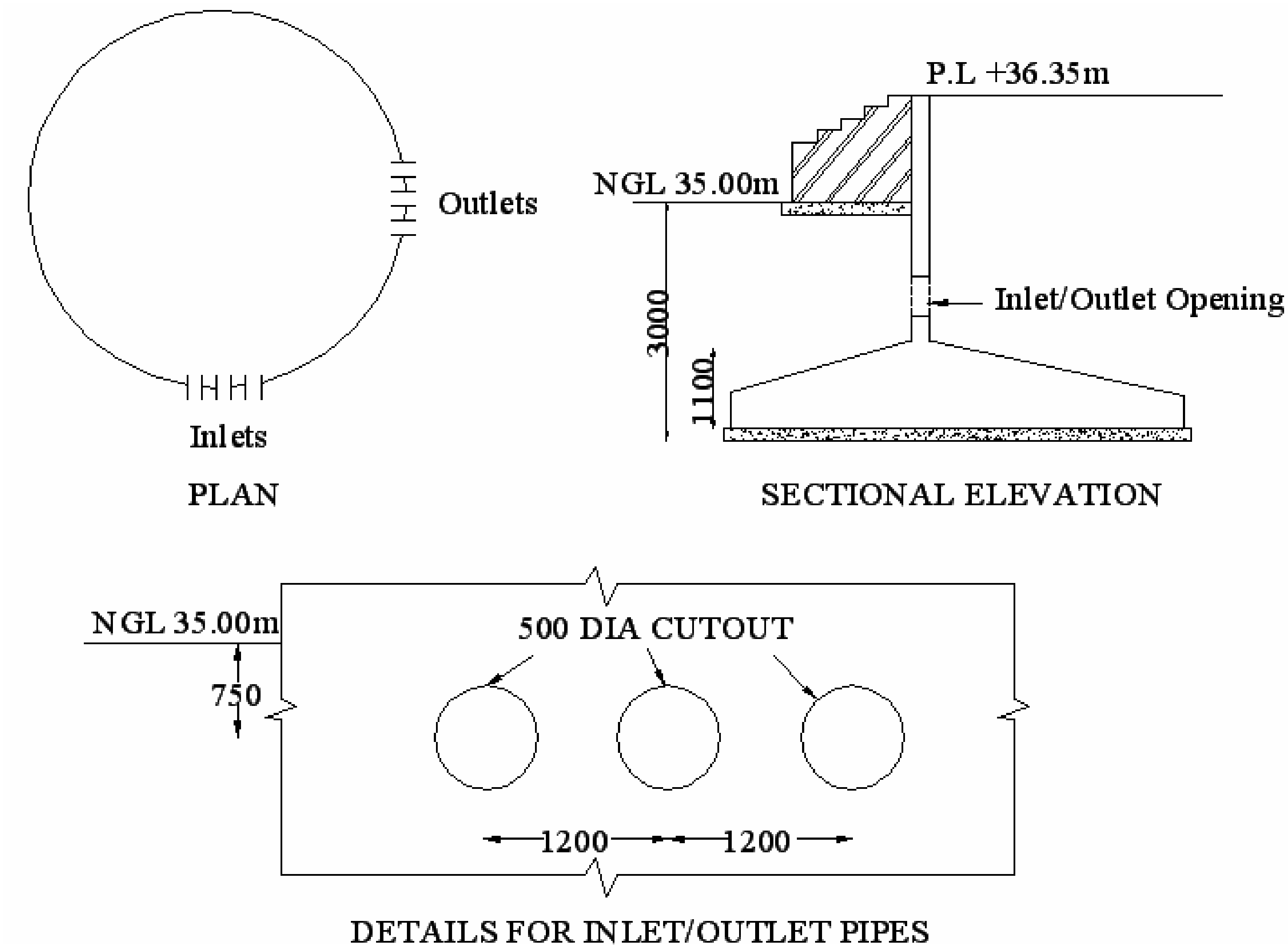
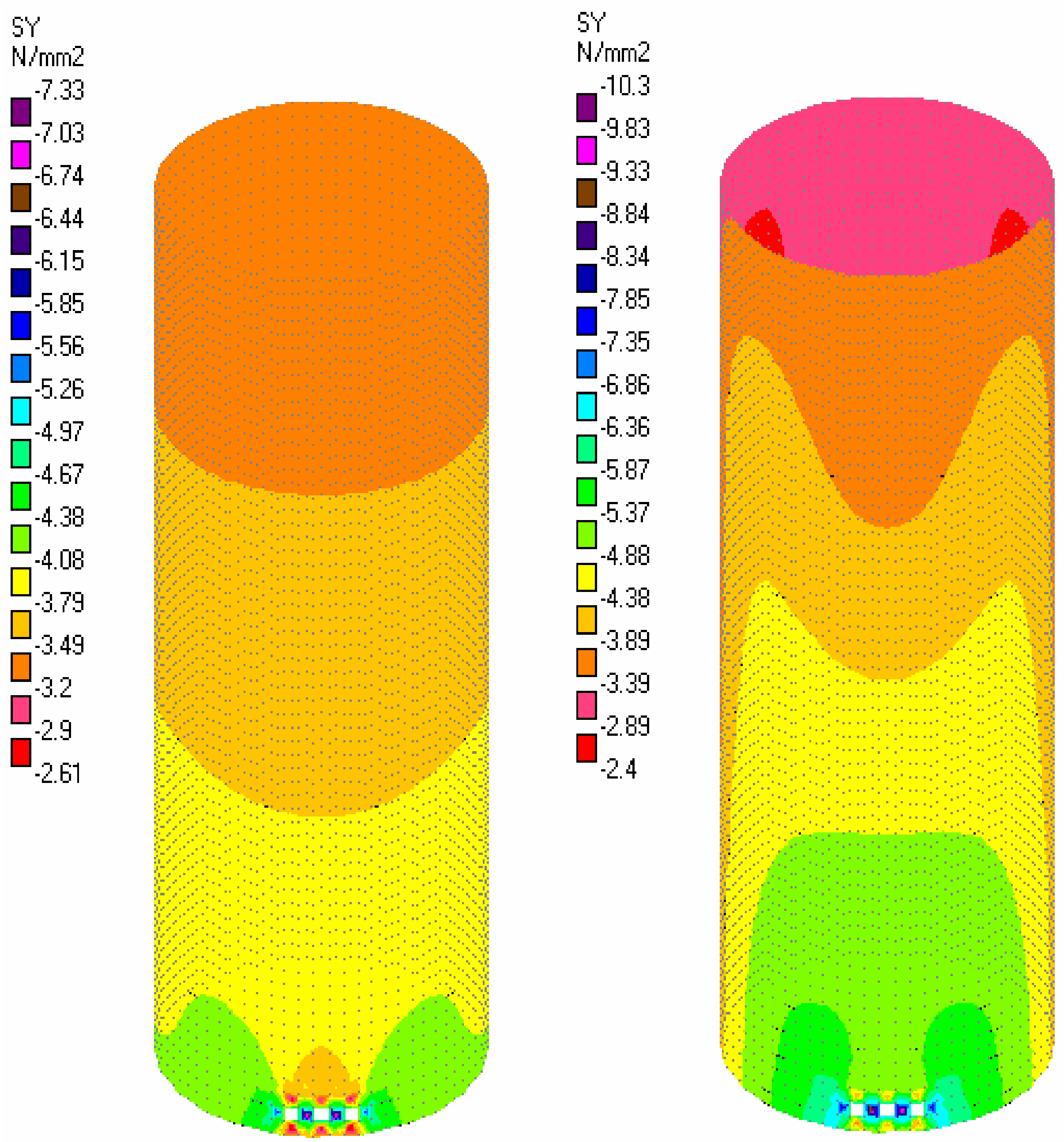


FIGURE: 5.19 DETAILS OF OPENINGS FOR INLET & OUTLET PIPES IN SHAFT



(a) **(b)**

(a) SELF WEIGHT + VERTICAL LOAD

(b) SELF WEIGHT + VERTICAL LOAD + EARTHQUAKE LOAD

FIGURE: 5.20: STRESSES IN SHFT AROUND INET/OUTLET OPENING

In FIGURE: 5.20 (a) the variation of stresses around inlet/outlet opening can be seen for the load case when only the self weight of shaft plus vertical load of container including water are applied on the shaft. The maximum stress in this case has been found in range of 7.33 N/mm^2 around the sides of the opening and between two adjacent openings. The detail study of stresses around the door opening is given in FIGURE: 5.21 (b).

In FIGURE: 5.20 (b) the variation of stresses around door opening can be seen for the load case when the self weight of shaft plus vertical load of container including water and load due to earthquake in the governing direction are applied on the shaft. The maximum stress in this case has been found in range of 10.30 N/mm^2 around the sides of the opening and between two adjacent openings. The detail study of stresses around the door opening is given in FIGURE: 5.22 (b).

4.03	4.03	4.03	4.03	4.03	4.03	4.03	4.03	4.03	4.03	4.03	4.03	4.03
4.05	4.05	4.05	4.05	4.05	4.05	4.05	4.05	4.05	4.05	4.05	4.05	4.05
4.06	4.06	4.06	4.06	4.06	4.06	4.06	4.06	4.06	4.06	4.06	4.06	4.06
4.07	4.07	4.07	4.07	4.07	4.07	4.07	4.07	4.07	4.07	4.07	4.07	4.07
4.09	4.09	4.09	4.09	4.09	4.09	4.09	4.09	4.09	4.09	4.09	4.09	4.09
4.10	4.10	4.10	4.10	4.10	4.10	4.10	4.10	4.10	4.10	4.10	4.10	4.10
4.11	4.11	4.11	4.11	4.11	4.11	4.11	4.11	4.11	4.11	4.11	4.11	4.11

4.17	4.13	4.06	3.96	3.86	3.79	3.77	3.79	3.86	3.96	4.06	4.13	4.17
4.21	4.18	4.11	3.98	3.83	3.75	3.72	3.75	3.83	3.98	4.11	4.18	4.21
4.25	4.24	4.20	4.00	3.79	3.70	3.68	3.70	3.79	4.00	4.20	4.24	4.25
4.27	4.33	4.31	4.12	3.53	3.83	3.47	3.83	3.53	4.12	4.31	4.33	4.27
4.28	4.35	4.51	4.45	2.80	4.38	2.94	4.38	2.80	4.45	4.51	4.35	4.28
4.30	4.38	4.54	5.82		7.30		7.30		5.82	4.54	4.38	4.30
4.35	4.48	4.71	4.41	2.60	4.44	2.82	4.44	2.60	4.41	4.71	4.48	4.35

All stresses are in N/mm^2

(a)

(b)

(a) WITHOUT OPENING

(b) WITH INLET AND OUTLET PIPE OPENINGS

FIGURE: 5.21: STRESSES IN SHFT AROUND INET/OUTLET OPENING

FOR SELF WEIGHT OF SHAFT PLUS VERTICAL LOAD

FIGURE: 5.21 (a) shows stresses in various elements of shaft without opening and around probable inlet/outlet pipe opening region for self weight of shaft plus vertical load of container including water. FIGURE: 5.21 (b) shows stresses in various elements of shaft with inlet/outlet pipe openings for the same load case.

By comparing the stresses in various elements around openings shown in FIGURE: 5.21 (a) with the stresses in FIGURE: 5.21 (b) one can get idea of increase in stresses around the opening region.

5.35	5.42	5.47	5.52	5.55	5.57	5.57	5.57	5.55	5.52	5.47	5.42	5.35
5.38	5.45	5.51	5.56	5.59	5.61	5.61	5.61	5.59	5.56	5.51	5.45	5.38
5.42	5.49	5.55	5.59	5.63	5.69	5.70	5.69	5.63	5.59	5.55	5.49	5.42
5.45	5.52	5.59	5.63	5.67	5.69	5.70	5.69	5.67	5.63	5.59	5.52	5.45
5.49	5.56	5.62	5.67	5.71	5.74	5.73	5.74	5.71	5.67	5.62	5.56	5.49
5.52	5.60	5.66	5.72	5.75	5.78	5.79	5.78	5.75	5.72	5.66	5.60	5.52
5.55	5.63	5.70	5.75	5.79	5.81	5.82	5.81	5.79	5.75	5.70	5.63	5.55

5.53	5.55	5.51	5.41	5.31	5.23	5.20	5.23	5.31	5.41	5.51	5.55	5.53
5.60	5.64	5.60	5.46	5.29	5.20	5.17	5.20	5.29	5.46	5.60	5.64	5.60
5.68	5.75	5.74	5.51	5.25	5.16	5.13	5.16	5.25	5.51	5.74	5.75	5.68
5.73	5.89	5.92	5.70	4.91	5.36	4.86	5.36	4.91	5.70	5.92	5.89	5.73
5.76	5.94	6.22	6.17	3.92	6.16	4.13	6.16	3.92	6.17	6.22	5.94	5.76
5.80	5.99	6.28	8.14		10.28		10.28		8.14	6.28	5.99	5.80
5.88	6.14	6.55	6.19	3.67	6.26	4.00	6.26	3.67	6.19	6.55	6.14	5.88

All stresses are in N/mm^2

(a) **(b)**

(a) WITHOUT OPENING

(b) WITH INLET AND OUTLET PIPE OPENINGS

FIGURE: 5.22: STRESSES IN SHFT AROUND INET/OUTLET OPENINGS

FOR SELF WEIGHT OF SHAFT, VERTICAL LOAD PLUS EARTHQUAKE LOAD

FIGURE: 5.22 (a) shows stresses in various elements of shaft without opening and around the probable inlet/outlet opening region for self weight of shaft plus vertical load of container including water and earthquake load in governing direction. While FIGURE: 5.22 (b) shows stresses in various elements of shaft with openings for inlet/outlet pipes for the same load case.

By comparing the stresses in various elements around openings shown in FIGURE: 5.22 (a) with the stresses in FIGURE: 5.22 (b) one can get idea of increase in stresses around the opening region.

Since the size of opening is very small the concentration of stresses vanishes as we move away from the opening. Moving equal to the width of the opening from the face of the opening the stress concentration reduces to 11.00% which can be considered as nominal stress concentration. Comparisons of stresses in both the cases, without opening and with opening, suggest that in the region of ¾ the width of the opening has significant effect of stress concentration which is similar to the earlier cases of opening for the door.

5.8 STRESS RELEASE ABOVE AND BELOW THE INLET AND OUTLET OPENINGS:

In the previous section the effects of inlet/outlet opening in view of the stress concentration on sides and between two adjacent openings have been discussed. In certain cases inlet and outlet pipes are provided very near to each other as well as they are almost at same level, further if the diameters are large and foundation raft is very close to bottom of openings, then in such cases because of stress release below pipe openings almost no load is transferred in that much length of raft.

This study is more significant for the permanent loads (self weight of shaft plus vertical load of container including water). Generally positioning of annular raft is done in such a way that under the effect of permanent loads the centre of gravity of the uniform soil reaction coincides with the shaft location. Now with the presence of openings the type of loading below the opening region, which generally assumed to be uniformly distributed load, changes significantly.

Therefore the basic assumption that the permanent loads will cause a uniform upward soil reaction goes wrong up to certain extent.

To study the effects of such openings on the foundation FIGURE: 5.23 represent the variation of stresses at foundation level.

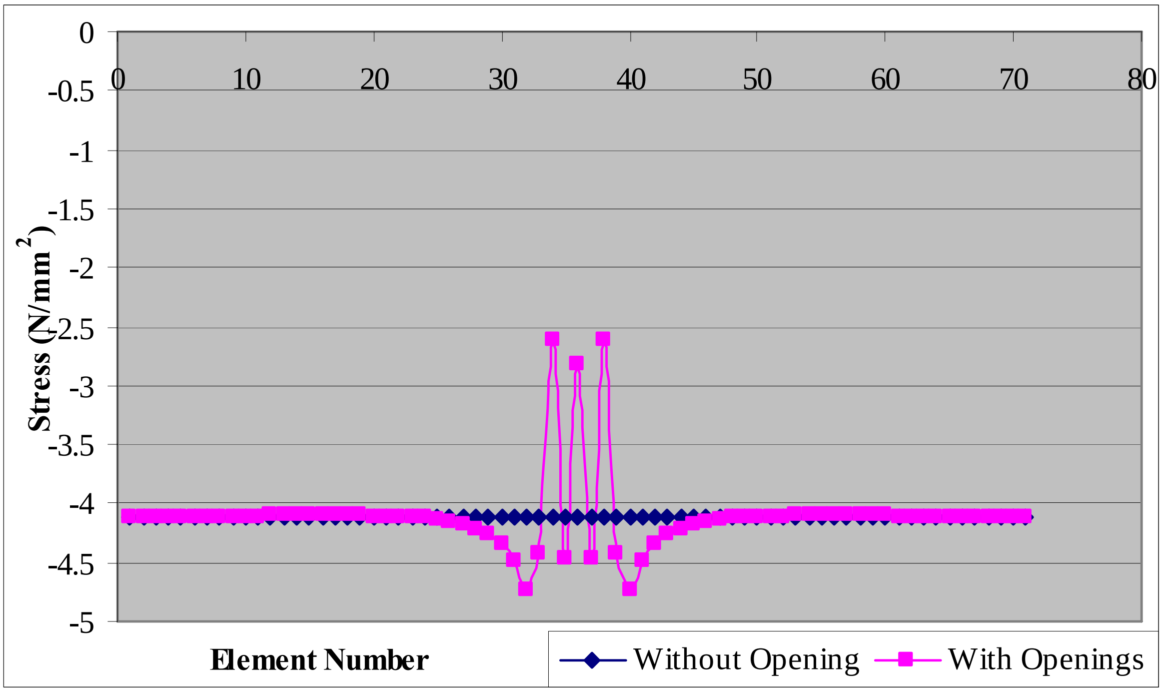


FIGURE: 5.23: STRESSES IN SHAFT DUE INLET OPENING AT FOUNDATION LEVEL

In FIGURE: 5.23 numbers on X - axis represent the element numbers of shaft at any particular level, here at foundation level. Element number 34, 36 and 38 are the locations of the openings for inlet/outlet openings. Values on Y - axis represent the stresses in the elements. Graph with blue color represent the stress level in various elements of shaft due to permanent loads without any opening. Similarly the graph with magenta color represents the stress level in various elements of shaft due to permanent loads with presence of opening for inlet/outlet. For simplicity here only inlet openings have been presented. If all the openings present in the shaft, (door opening plus inlet and outlet openings), have been discussed later on.

In FIGURE: 5.23 the graph, with blue color, for the shaft without any openings gives a strait line parallel to X – axis, which indicate uniform distribution of stresses or forces from shaft on foundation level and justify the assumption of uniform upward soil reaction. But in the same figure the graph, with magenta color, for the shaft with inlet/outlet openings shows variations in the stresses at foundation level. The variation of stresses at foundation level has remained local that is in the vicinity of the opening region only.

In the vicinity of the opening the stress level is as high as 4.56 N/mm^2 compared to 4.11 N/mm^2 without any opening in the same region. Therefore the stress level has increased by 16.00%. At the same time the portion exactly below the opening region shows the effect of stress release on the foundation. The minimum stress level in this region has found to be around 2.61 N/mm^2 which indicates stress release of around 60.00%. Moving away from the opening region most of the shaft segments have uniform stress levels.

FIGURE: 5.23 include all the openings to extend this knowledge and to study the distribution of stresses on foundation in presence of all the openings. Here the effects due to door opening (which may not be very significant), inlet and outlet openings are included. In FIGURE: 5.23 the graph with blue color represent the stress levels of shaft at foundation level without any openings and the graph with magenta color represent the stress level of shaft at foundation level with all openings, which include door opening plus inlet and outlet openings.

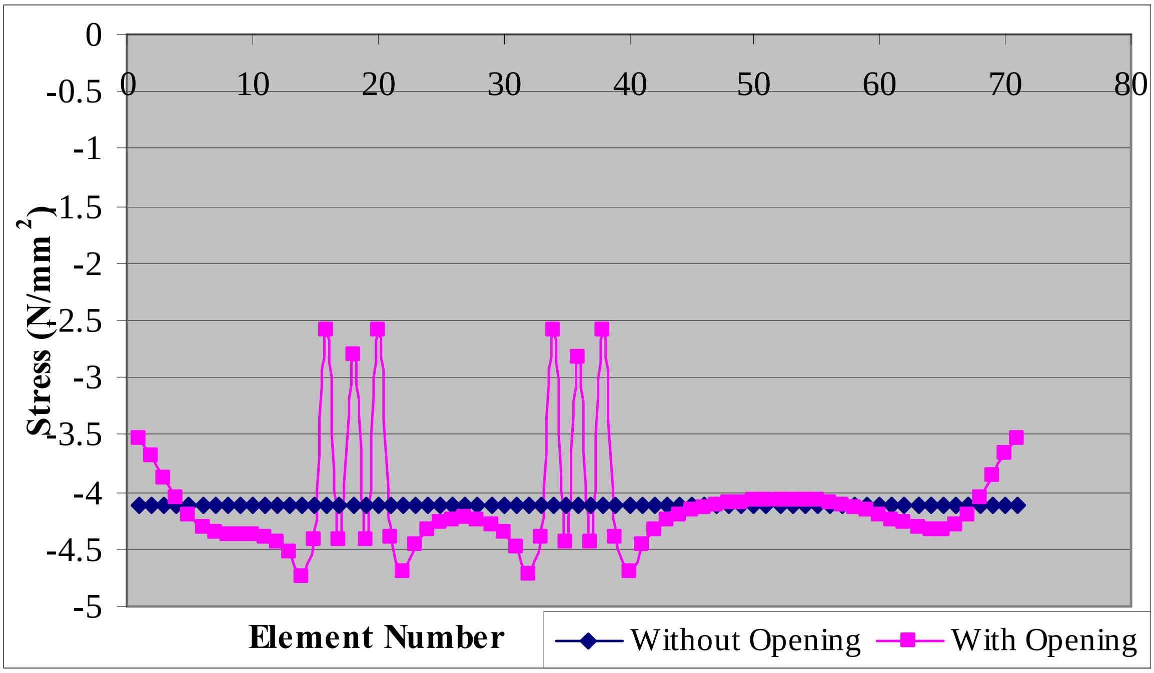


FIGURE: 5.24: STRESSES IN SHAFT DUE TO ALL OPENINGS AT FOUNDATION LEVEL

In above figure element number 1 and 71 represent the region exactly below the door opening at foundation level. Element number 16, 18 and 20 represent the region of inlet openings and element number 34, 36 and 38 represent the region of outlet openings at foundation level.

In above figure graph with blue color represent the stress level in different elements of shaft due to permanent loads without any opening. Similarly graph with magenta color represents the stress level in different elements of shaft due to permanent loads under the influence of all the opening. FIGURE: 5.23 represent the actual stress levels at different locations of the shaft at foundation level under the influence of permanent loads. The actual stress levels show considerable variations when compared with the stress levels of shaft without any opening.

5.9 SUMMARY:

➔ Because of opening the load has to change its path and has to traverse through some alternative path. Therefore stress concentration can be seen on sides of the openings and nearly zero stress can be seen on top and bottom region of opening because of free surface in these regions. Available results confirm the same and stress concentration has been found in the region of ¾ the width of opening on both sides of the opening. The increase in stresses on face of openings for various sizes of openings has been presented in TABLE: 5.3.

TABLE: 5.3: SUMMARY OF PERCENTAGE INCREASE IN STRESSES FOR DIFFERENT DOOR SIZES

Door Size (m)	Percentage Increase in Stresses (%)	
	Gravity Loads	Gravity + Lateral Loads
1.08 x 2.12	47.37	47.06
1.62 x 2.12	71.68	71.32
2.16 x 212	92.98	93.91

➔ If stresses are within certain limit additional reinforcement can be provided to take care of stress concentration. When the stresses exceed beyond certain limit vertical and horizontal ribs shall be provided. After providing vertical and horizontal ribs in ¼ the width of the opening, 16.00% reduction in stresses in gravity loads have been found. Similar results have been found for the combination of gravity plus earthquake loads.

➔ Stress concentration around inlet and outlet pipe openings has been found. The percentage increase in stresses has been to be 78.05% for gravity loads and 77.85% for the combination of gravity plus earthquake loads.

➔ Here for the selected case study the openings for inlet and outlet pipes are located below the natural ground level and near to the foundation level. Therefore the effect of these openings can be seen on the foundation raft.

Practically the portion exactly below the openings has transferred very less load on the foundation raft.

➔ FIGURE: 5.24 represent comparison of actual pattern of load transfer on foundation raft with the uniform pattern of load transfer on the same for gravity loads only. It is quite clear from the figure that the basic assumption of uniform upward soil reaction for permanent loads in design of foundation raft is not correct when openings for inlet and outlet pipes are placed near to the foundation raft.

6 INACCURACIES IN SHAFT CONSTRUCTION

In this chapter an attempt is made to study the effects of constructional inaccuracies during construction of RCC circular shaft. For this purpose selected water tank is located at Ajwa, Vadodara. The water tank has a capacity of 1800000 Liters and total height of 41m from the natural ground level. The height of the shaft is 30m from the natural ground level. The water tank is under construction and nearing completion. Details and design of the same have been discussed in the earlier chapter.

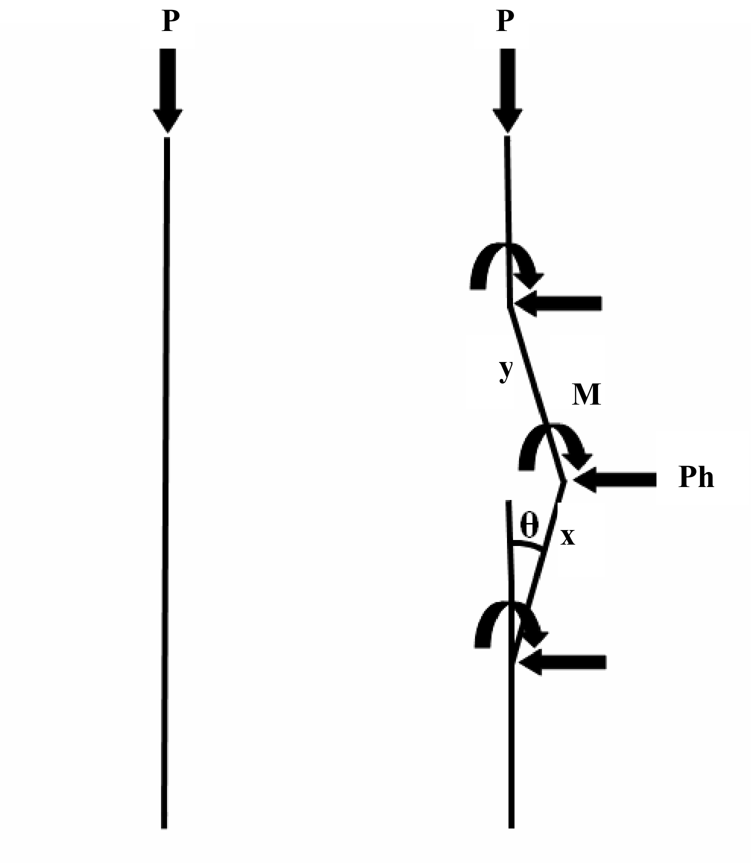
6.1 INTRODUCTION:

When sliding formwork is used, construction of shaft becomes very speedy but care should be taken to maintain verticality, circularity and uniformity of thickness. Any variation in geometry would result into additional stresses. In conventional practice the RCC shaft supporting elevated water tank is constructed by jump form method. In this method the shaft wall of about 1.0m to 1.2m height is cast in stages. Many times inaccuracy as regards shaft wall being out of plumb in some stages occurs. The out of plumb may be in few stages along the total height of wall. Though certain inaccuracies are permissible, however when inaccuracy is beyond permissible tolerance additional stresses in addition to stresses due to self weight of shaft plus container including weight of water are developed. But the Indian Standard Code IS: 11682 – 1985 (design of RCC staging for overhead water tanks) has been silent about the minimum allowable tolerances in the construction of shaft of elevated storage reservoirs. Therefore it is difficult for the designers to limit the permissible inaccuracies in construction of shaft. Currently designers are dependent on their own experience for the same. Therefore allowable tolerances in the construction of shafts are not uniform all over.

In the present work an effort has been made to study effects of construction inaccuracies on the supporting shaft of elevated water tanks. For the anticipated purpose, inaccuracies have been introduced in the shaft deliberately at various levels and the resulting additional stresses in the shaft are calculated. Here for the purpose of study three types of inaccuracies in shaft are considered. The same are modeled at various levels, starting from 4.50m to 32.50m height. Bur for the purpose of presentation of work only three arbitrarily selected levels have been illustrated. Inaccuracies studied are,

- Displacement of complete shaft from its original position
- Bulging of shaft
- Bulging and contraction of shaft

At each level the inaccuracies have been introduced as some degree of verticality. FIGURE: 6.1 (a) show any one face of the shaft wall with perfectly in vertical position. Such shafts are subjected to membrane stresses only. Now because of some construction inaccuracy the shaft has gone out of plumb which is shown in FIGURE: 6.1 (b) by the length x. As soon as the site staff will find such inaccuracy, they will try to correct the inaccuracy and in that process the next layer will be brought to its original position which is shown in FIGURE: 6.1 (b) by length y. Therefore now the shaft has again come back into its original position. But now the shaft is not perfectly vertical throughout its height and now the shaft looks like as shown in FIGURE: 6.1 (b). Therefore shaft is now subjected to a radial force (P_h) and moment (M) in addition to the vertical force (P) as shown in FIGURE: 6.1 (b).



(a) (b)

(a) TRUE POSITION

(b) SHAFT WITH ECCENTICITY IN VERTICALITY

FIGURE 6.1: SHAFT WALL ALONG HEIGHT

6.2 MODELING:

For the purpose of study, the eccentricities are introduced at different levels. At each level the eccentricities are introduced as various angles starting from 0.50 degree to 4.00 degree at interval of 0.50 degree. The displacement at each angle (θ) in terms of length (millimeter) is tabulated in TABLE: 6.1.

TABLE: 6.1: ECCENTICITIES IN VERTICALITY IN mm

θ (degree)	Tolerance Level - 1 (mm)	Tolerance Level - 2 (mm)
0.50	04.63	09.25
1.00	09.25	18.50
1.50	13.88	27.76
2.00	18.51	37.02
2.50	23.14	46.28
3.00	27.78	55.55
3.50	32.42	64.83
4.00	37.06	74.12

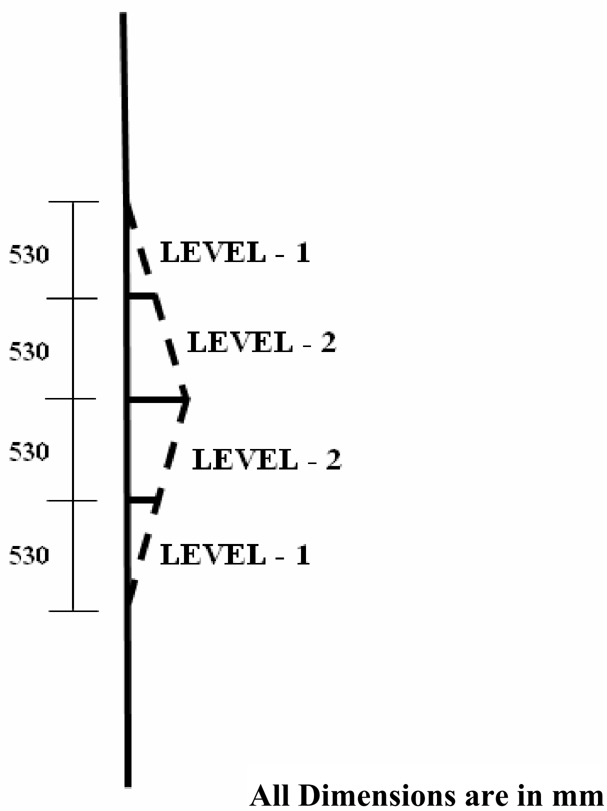


FIGURE 6.2: ECCENTRICITY IN SHAFT WALL AT EACH LEVEL

As mentioned earlier, generally the formwork is of height 1.00m and the element height used in the model is 0.53m. Therefore the eccentricity at each level is introduced in two successive layers to simulate actual site condition. For example if any particular layer of formwork has gone out of plumb by 1.00 degree (18.85mm, TABLE: 6.1) then to incorporate the same site condition the elements of height 0.53m of level – 1, shown in FIGURE: 6.2, are considered out of plumb by half of the actual eccentricity (9.43mm, TABLE: 6.1). The remaining eccentricity (9.43mm) is introduced in next level, level – 2. Which will simulate actual site condition and the whole layer of 1.00m height of shaft is now out of plumb by 1.00 degree. Further the same eccentricities are introduced in reverse order in the next upper two layers of model to bring the shaft in its true position again.

- **MODELING FOR DISPLACEMENT OF SHAFT:**

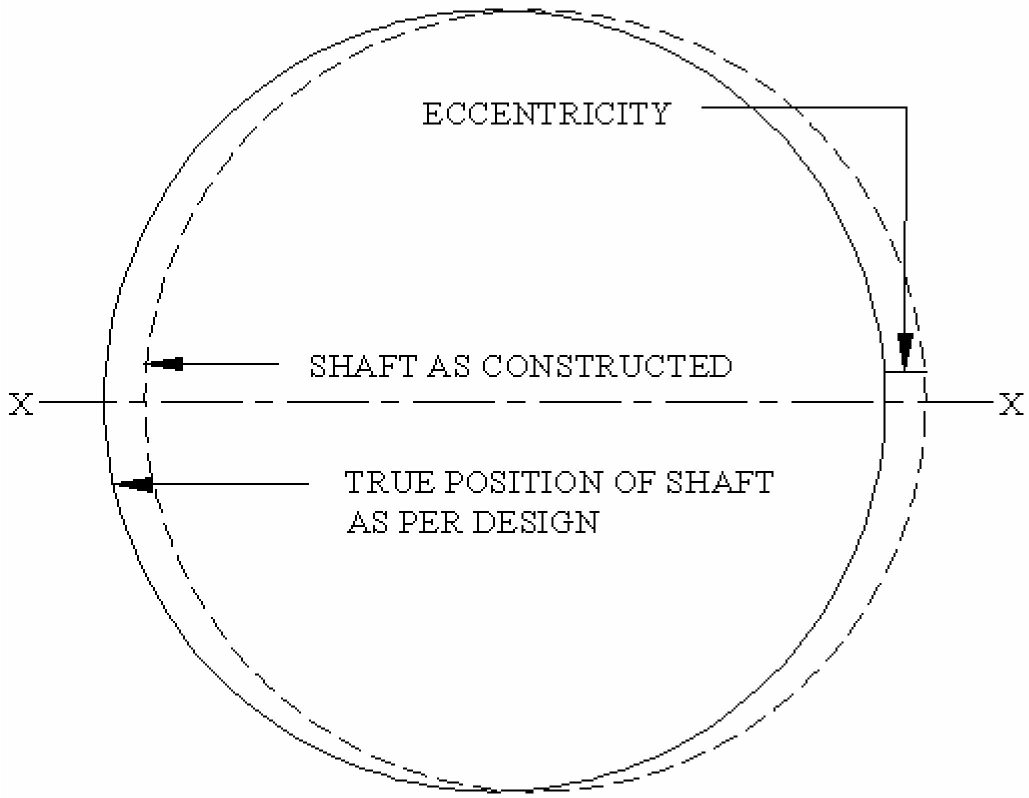


FIGURE 6.3: DISPLACEMENT OF SHAFT (PLAN)

FIGURE: 6.3 illustrate the eccentricity in the shaft due to verticality in plan. In the figure the line full line is the true centre line of shaft. The one with doted line represent shaft as constructed on the site.

Here for the modeling purpose it has been assumed that because of construction inaccuracy at any particular level, the shaft has been displaced by certain amount parallel to X–axis but the diameter of the shaft has remain same even after error in construction. As a result of this the shaft has the maximum displacement exactly on the X-axis but as we move away from the X-axis the tolerances are reducing. Therefore maximum additional stresses are bound to occur in the elements exactly on the X-axis and as we move away from the X-axis the additional stresses are reducing.

- **BULGING OF SHAFT:**

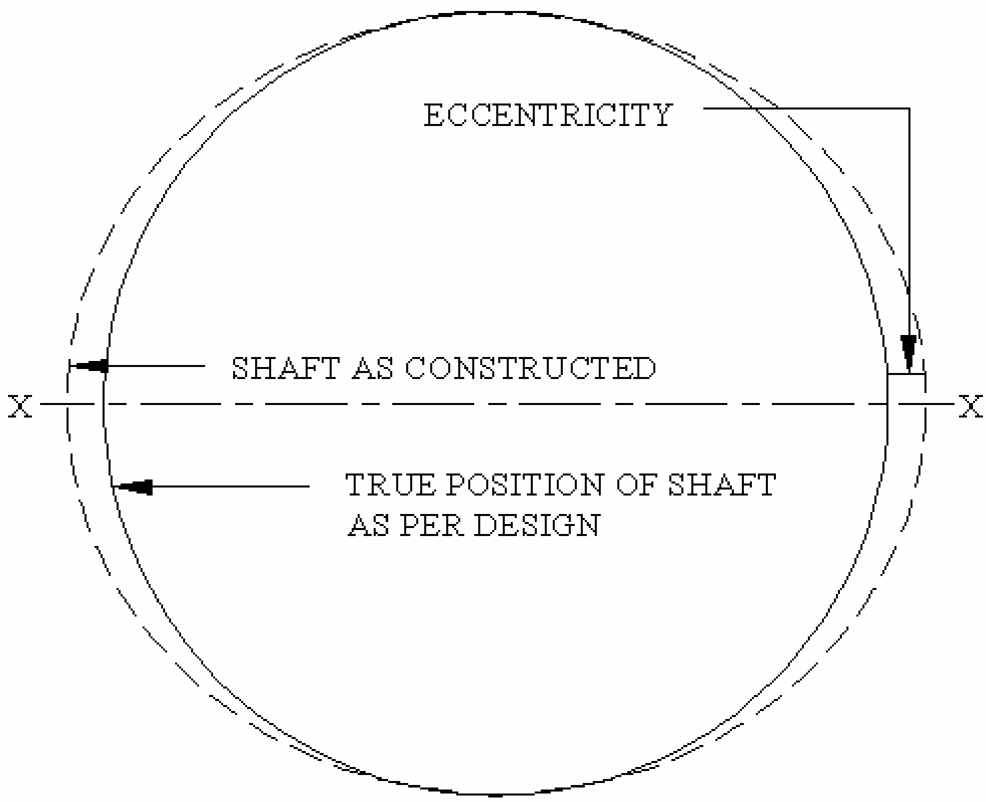


FIGURE 6.4: BULGING OF SHAFT (PLAN)

FIGURE: 6.4 illustrate the bulging shaft due to eccentricity in plan. In the figure the line full line is the true centre line of shaft. The one with doted line represent shaft as constructed on the site.

Here for the modeling purpose it has been assumed that because of construction tolerances at any particular level, the shaft has been bulged by certain amount parallel to X–axis and therefore the diameter of the shaft has increased in the same direction but the diameter in the other direction has remain same. As a result of this the shaft has the maximum eccentricity exactly on the X-axis but as we move away from the X-axis the eccentricities are reducing. Therefore maximum additional stresses are bound to occur in the elements exactly on the X-axis and as we move away from the X-axis the additional stresses are reducing.

- **BULGING AND CONTRACTION OF SHAFT:**

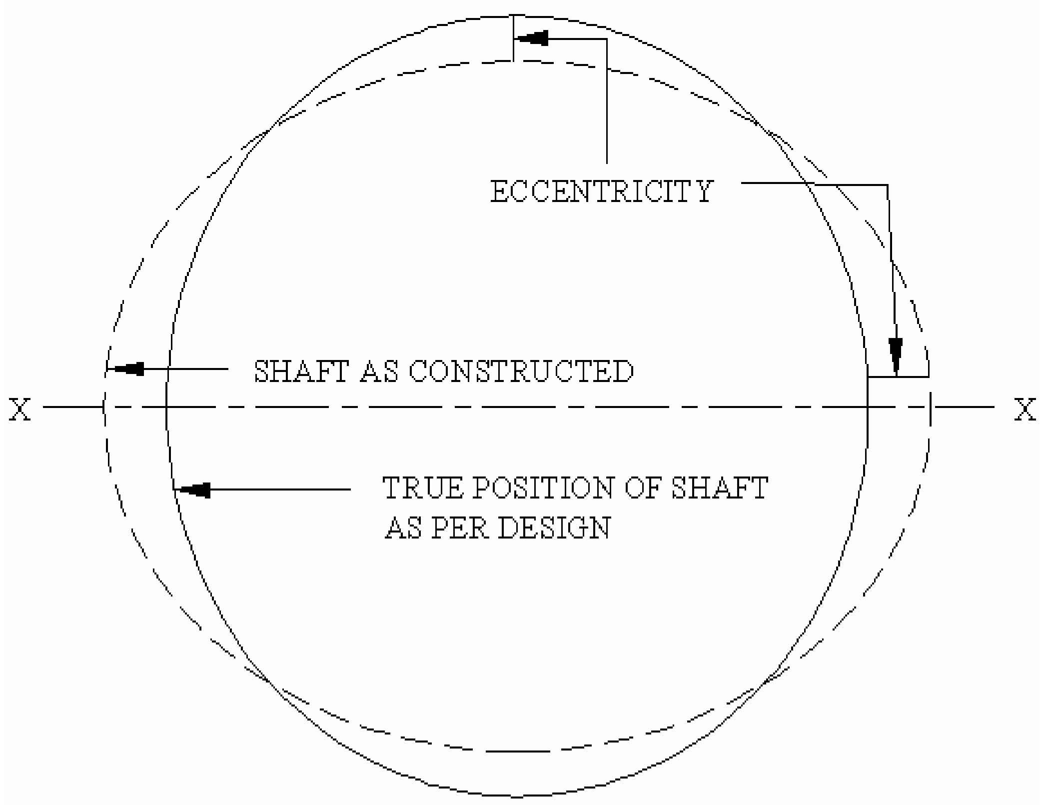


FIGURE 6.5: BULGING AND CONTRACTION OF SHAFT (PLAN)

FIGURE: 6.5 illustrate the bulging and contraction of shaft due to eccentricity in plan. In the figure the line full line is the true centre line of shaft. The one with doted line represent shaft as constructed on the site.

Here for the modeling purpose it has been assumed that because of construction inaccuracy at any particular level, the shaft has bulged by certain amount parallel to X–axis and therefore the diameter of the shaft has increased in the same direction but at the same time the shaft has contracted in the other direction therefore diameter in this direction has decreased and as a result of this the shaft has taken the shape of an ellipse. Therefore the shaft has the maximum displacement exactly on the X-axis but as we move away from the X-axis the eccentricities are reducing and again as we arrive at the extreme distance from the X–axis the shaft has maximum eccentricities in the other direction. Therefore in this case the effects of inaccuracies are found in most locations on the perimeter of shaft.

6.3 EFFECTS OF DISPLACEMENT OF SHAFT:

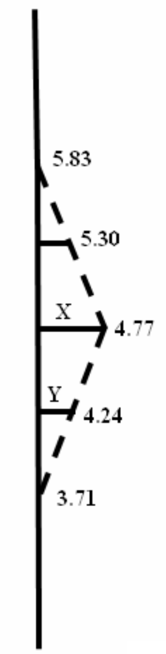
As mentioned earlier the modeling is done for all different layers starting from 4.50m to 32.50m although for the purpose of presentation of the work only three arbitrarily selected levels have been illustrated.

- Out of plumb at 4.77m elevation
- Out of plumb at 15.90m elevation
- Out of plumb at 32.33m elevation

Here for the study of the inaccuracies only permanent loads are taken into consideration. As these loads are permanent any increase in stresses is not allowed for such additional stresses due to constructional inaccuracies.

- **OUT OF PLUMB AT 4.77m ELEVATION:**

As mentioned earlier in the modeling part, for introducing eccentricity at 4.77m level the eccentricities needs to be introduced starting from one level below (4.24m) and then it should be reversed for the upper layers to simulate actual site condition in the modeling.



All Dimensions are in meter

FIGURE 6.6: DISPLACEMENT IN SHAFT WALL AT 4.77m

The model for eccentricity at 4.77m level is illustrated in FIGURE: 6.6. In the FIGURE: 6.6 X indicates maximum eccentricity at level 4.77m for the respective degrees as mentioned in TABLE: 6.1 and Y indicates the eccentricity needs to be incorporated one level below to simulate the actual site condition. Because of such inaccuracies the additional forces are illustrated in FIGURE: 6.1 and the same are tabulated in TABLE: 6.2.

TABEL: 6.2: BENDING STRESSES IN SHAFT WALL AFTER DISPLACEMENT AT 4.77m

θ (degree)	Eccentricity (e) (mm)	Direct Stress (N/mm^2)	Moment due to eccentricity (kN.m)	Bending Stress (N/mm^2)	% Increase in Stresses (%)
0.50	9.25	3.99	2.20	0.54	13.50
1.00	18.50	3.99	4.33	1.06	26.58
1.50	27.76	3.99	6.52	1.60	40.02
2.00	37.02	3.99	8.61	2.11	52.85
2.50	46.28	3.99	10.71	2.62	65.74
3.00	55.55	3.99	13.04	3.19	80.04
3.50	64.83	3.99	15.13	3.71	92.87
4.00	74.12	3.99	17.23	4.22	105.76

In TABLE: 6.2 the bending stresses in shaft at 4.77m from top of foundation are tabulated. It is very clear from the resultant stresses that as the degree of inaccuracy increases the stresses are increasing proportionately. The last column represents the percentage increase in stresses because of bending stresses compared to direct stresses in the shaft. A graph is plotted for the percentage increase of stresses compared to direct stresses at different degrees of inaccuracies in FIGURE: 6.7.

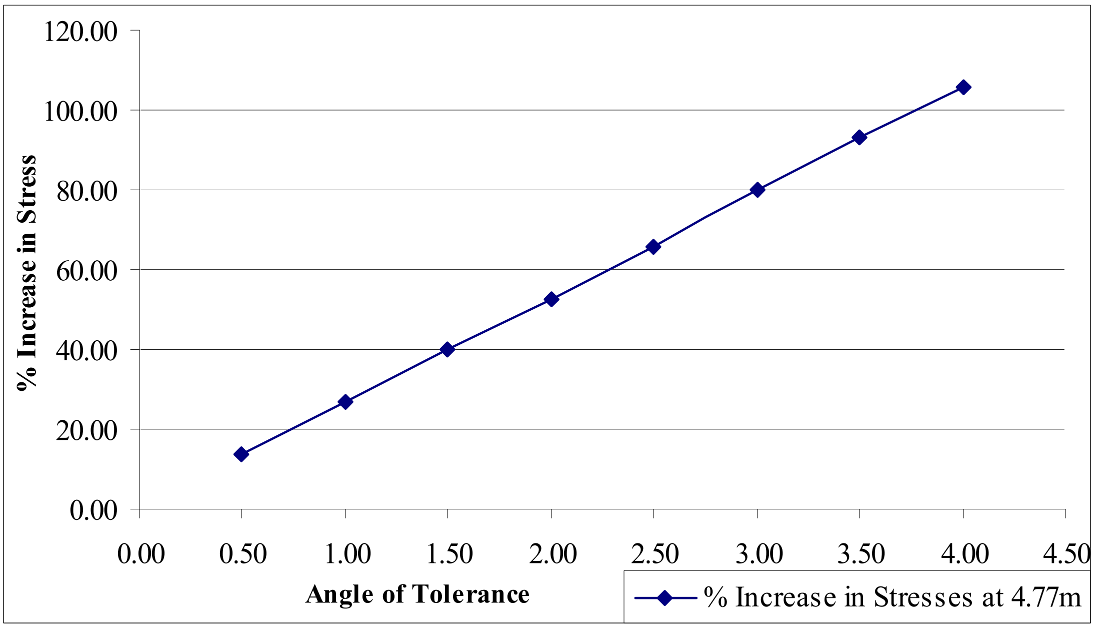


FIGURE 6.7: PERCENTAGE INCREASE OF STRESSES IN SHAFT WALL AFTER DISPLACEMENT AT 4.77m

As mentioned in FIGURE: 6.1 other forces acting on the shaft wall because of such inaccuracy are circumferential forces and shear forces. These forces are tabulated in TABLE: 6.3 at 4.77m from top of foundation.

TABEL: 6.3: CIRCUMFERENTIAL AND SHEAR FORCES IN SHAFT AFTER DISPLACEMENT AT 4.77m

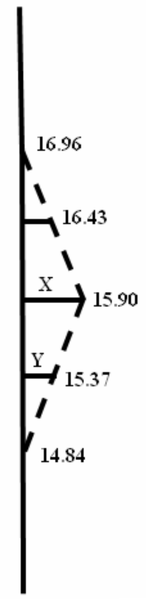
θ (degree)	Out of plumb (mm)	Circumferential Forces (N)	Shear Forces (N)
0.50	9.25	25.80	6.45
1.00	18.50	51.60	12.90
1.50	27.76	77.40	19.35
2.00	37.02	105.35	25.80
2.50	46.28	129.00	32.25
3.00	55.55	156.95	38.70
3.50	64.83	182.75	45.15
4.00	74.12	208.55	49.45

It is clear from the results tabulated in TABLE: 6.3 that the circumferential and shear forces in the shaft wall due to inaccuracies are very small and can be neglected.

- **OUT OF PLUMB AT 15.90m ELEVATION:**

For introducing eccentricity at 15.90m level the eccentricities needs to be introduced starting from one level below (15.37m) and then it should be reversed for the upper layers to simulate actual site condition in the modeling.

The model for inaccuracies at 15.90m level is illustrated in FIGURE: 6.8. In the FIGURE: 6.8 X indicates maximum eccentricity at level 15.90m for the respective degrees as mentioned in TABLE: 6.1 and Y indicates the eccentricity needs to be incorporated one level below to simulate the actual site condition. Because of such inaccuracy the additional forces are tabulated in TABLE: 6.4.



All Dimensions are in meter

FIGURE 6.8: DISPLACEMENT IN SHAFT WALL AT 15.90m

TABEL: 6.4: BENDING STRESSES IN SHAFT WALL AFTER DISPLACEMENT AT 15.90m

θ (degree)	Out of plumb e (mm)	Direct Stress (N/mm^2)	Moment due to eccentricity (kN.m)	Bending Stress (N/mm^2)	% Increase in Stresses (N/mm^2)
0.50	9.25	3.73	1.98	0.48	13.00
1.00	18.50	3.73	3.96	0.97	26.00
1.50	27.76	3.73	6.02	1.47	39.53
2.00	37.02	3.73	7.97	1.95	52.33
2.50	46.28	3.73	9.93	2.43	65.20
3.00	55.55	3.73	12.13	2.97	79.64
3.50	64.83	3.73	14.11	3.46	92.64
4.00	74.12	3.73	16.10	3.94	105.71

In TABLE: 6.4 the bending stresses in shaft at 15.90m from top of foundation are tabulated. The last column represents the percentage increase in stresses because of bending stresses compared to direct stresses on the shaft. When the percentage increase of stresses at 15.90m for various degrees of eccentricities are compared with the percentage increase at 4.77m level (TABLE:

6.2) it shows similar results. A graph is plotted for the percentage increase in stresses compared to direct stresses at different degrees of eccentricities in FIGURE: 6.9.

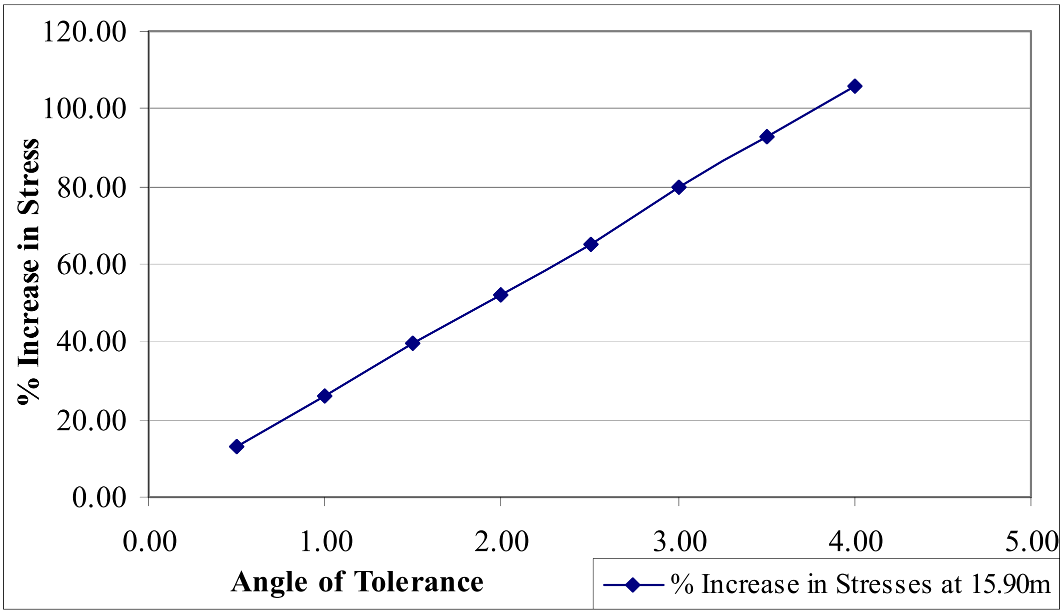


FIGURE 6.9: PERCENTAGE INCREASE OF STRESSES IN SHAFT WALL AFTER DISPLACEMENT AT 15.90m

The other forces acting on shaft wall because of such inaccuracies are circumferential forces and shear forces. These forces are tabulated in TABLE: 6.5 at 15.90m from top of foundation.

TABEL: 6.5: CIRCUMFERENTIAL AND SHEAR FORCES IN SHAFT AFTER DISPLACEMENT AT 15.90m

θ (degree)	Out of plumb (mm)	Circumferential Forces (N)	Shear Forces (N)
0.50	9.25	24.08	5.81
1.00	18.50	48.38	11.61
1.50	27.76	73.32	17.63
2.00	37.02	96.75	23.44
2.50	46.28	120.40	29.03
3.00	55.55	146.85	35.48
3.50	64.83	170.50	41.28
4.00	74.12	194.36	47.09

It is clear from the results tabulated in TABLE: 6.5 that the circumferential and shear forces in the shaft wall due to inaccuracies are very small and can be neglected.

- **OUT OF PLUMB AT 30.21m ELEVATION:**

For introducing eccentricities at 30.21m level the eccentricity needs to be introduced starting from one level below (4.24m) and then it should be reversed for the upper layers to simulate actual site condition in the modeling.

The model for eccentricity at 30.21m level is illustrated in FIGURE: 6.10. In the FIGURE: 6.10 X indicates maximum eccentricity at level 30.21m for the respective degrees as mentioned in TABLE: 6.1 and Y indicates the eccentricity needs to be incorporated one level below to simulate the actual site condition. Because of such inaccuracy the additional forces are tabulated in TABLE: 6.6.

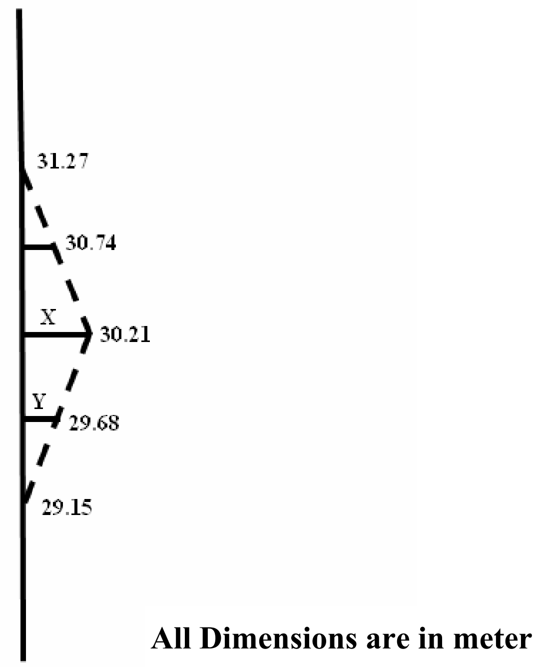


FIGURE 6.10: DISPLACEMENT IN SHAFT WALL AT 30.21m

TABEL: 6.6: BENDING STRESSES IN SHAFT WALL AFTER DISPLACEMENT AT 30.21m

θ (degree)	Out of plumbe (mm)	Direct Stress (N/mm^2)	Moment due to eccentricity (kN.m)	Bending Stress (N/mm^2)	% Increase in Stresses (N/mm^2)
0.50	9.25	3.37	1.72	0.42	12.50
1.00	18.50	3.37	3.52	0.86	25.58
1.50	27.76	3.37	5.38	1.32	39.10
2.00	37.02	3.37	7.16	1.75	52.03
2.50	46.28	3.37	8.94	2.19	64.97
3.00	55.55	3.37	10.95	2.68	79.58
3.50	64.83	3.37	12.76	3.12	92.73
4.00	74.12	3.37	14.59	3.57	106.03

In TABLE: 6.6 the additional stresses acting of shaft at 30.21m from top of foundation are tabulated. The last column represents the percentage increase in stresses because of bending stresses compared to direct stresses on the shaft. The percentage increase of stresses at 30.21m for various degrees of eccentricities are similar to the earlier both the level results. A graph is plotted for the percentage increase in stresses compared to direct stresses at various degrees of eccentricities in FIGURE: 6.11.

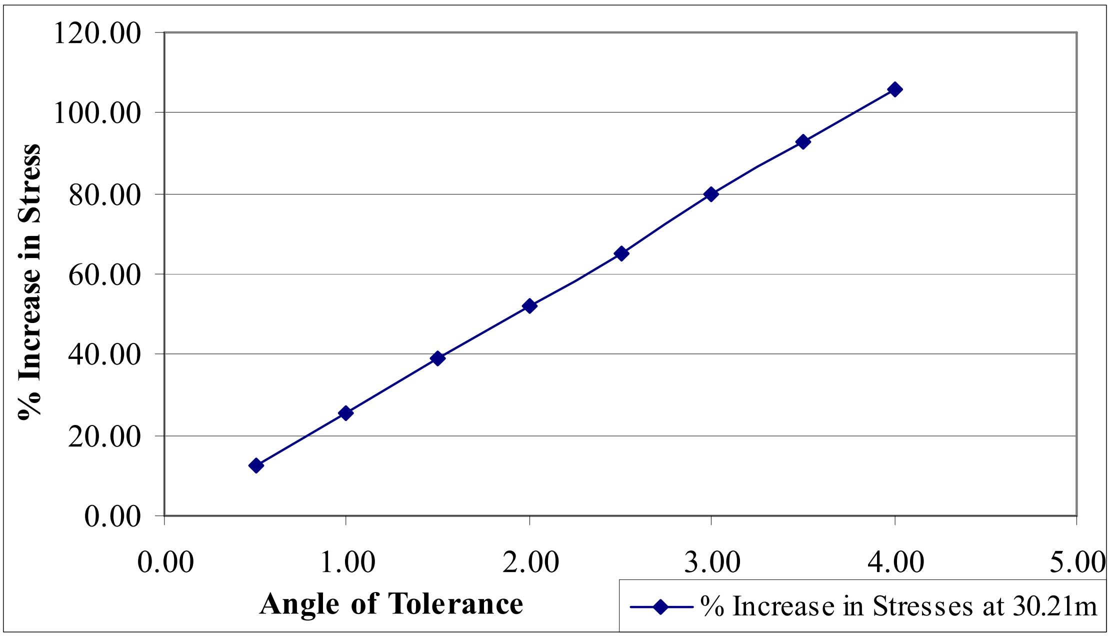


FIGURE 6.11: PERCENTAGE INCREASE OF STRESSES IN SHAFT WALL AFTER DISPLACEMENT AT 30.21m

The other forces acting on shaft wall because of such inaccuracies are circumferential forces and shear forces. These forces are tabulated in TABLE: 6.7 at 30.21m from top of foundation.

TABEL: 6.7: CIRCUMFERENTIAL AND SHEAR FORCES IN SHAFT AFTER DISPLACEMENT AT 30.21m

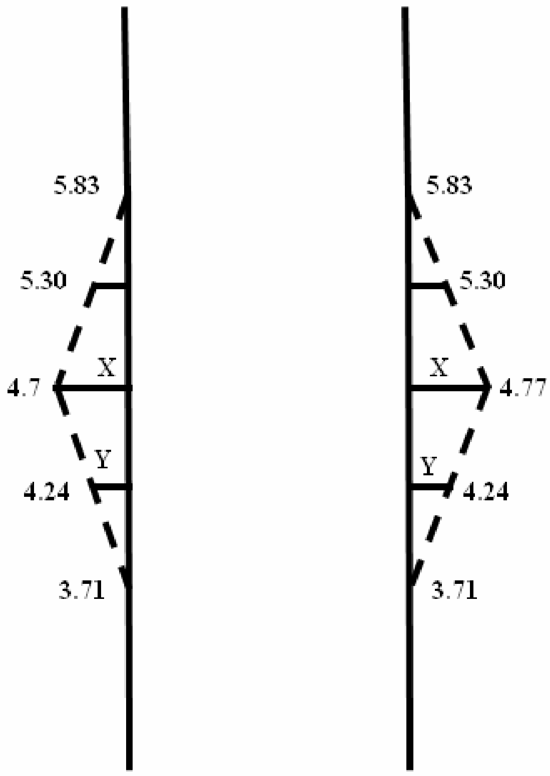
θ (degree)	Out of plumb (mm)	Circumferential Forces (N)	Shear Forces (N)
0.50	9.25	24.08	5.81
1.00	18.50	48.38	11.61
1.50	27.76	73.32	17.63
2.00	37.02	96.75	23.44
2.50	46.28	120.40	29.03
3.00	55.55	146.85	35.48
3.50	64.83	170.50	41.28
4.00	74.12	194.36	47.09

It is clear from the results tabulated in TABLE: 6.7 that the circumferential and shear forces in the shaft wall due to inaccuracies are very small and can be neglected.

6.4 EFFECTS OF BULGING OF SHAFT:

In the previous section the results of various levels establish the fact that the effects of inaccuracy in construction at different level are localized and remain more or less the same. For other modeled inaccuracies also the fact remains the same. Therefore in this problem the produced results are only of lower layer of the shaft, where maximum stresses are their and therefore effect of construction inaccuracies are also maximum around this level. It is worth to note that even though the additional stresses are high at lower level but the percentage increase of additional stresses remains more or less same for all levels.

As mentioned earlier in the modeling part, for introducing construction inaccuracy at 4.77m level the eccentricity needs to be introduced starting from one level below (4.24m) and then it should be reversed for the upper layers to simulate actual site condition in the modeling as it is done for earlier case. Further in this case the diameter of the shaft does not remain same for both the axis as it was assumed in the previous case. Because of bulging parallel to X-axis the diameter of the shaft has increased in the same direction while the diameter has remained same for the other axis.



All Dimensions are in meter

FIGURE 6.12: BULGING IN SHAFT WALLS PARALLEL TO X-AXIS AT 4.77m

The model for construction inaccuracies at 4.77m level is illustrated in FIGURE: 6.12. In the FIGURE: 6.12 X indicates maximum eccentricity at level 4.77m for the respective degrees as mentioned in TABLE: 6.1 and Y indicates the eccentricity needs to be incorporated one level below to simulate the actual site condition. Because of such construction inaccuracies the additional forces are illustrated in FIGURE: 6.1 and the forces are tabulated in TABLE: 6.8.

TABEL: 6.8: BENDING STRESSES IN SHAFT WALL AFTER BULGING AT 4.77m

θ (degree)	Out of plumbe (mm)	Direct Stress (N/mm^2)	Moment due to eccentricity (kN.m)	Bending Stress (N/mm^2)	% Increase in Stresses (N/mm^2)
0.50	9.25	3.99	2.07	0.51	12.71
1.00	18.50	3.99	4.16	1.02	25.53
1.50	27.76	3.99	6.30	1.54	38.67
2.00	37.02	3.99	8.32	2.04	51.07
2.50	46.28	3.99	10.32	2.53	63.34
3.00	55.55	3.99	12.53	3.07	76.91
3.50	64.83	3.99	14.49	3.55	88.94
4.00	74.12	3.99	16.45	4.03	100.97

In TABLE: 6.8 the additional stresses acting of shaft at 4.77m from top of foundation are tabulated. It is very clear from the resultant stresses that as the degree of inaccuracy increases the stresses are increasing proportionately. The last column represents the percentage increase in stresses because of bending stresses compared to direct stresses on the shaft. A graph is plotted for the percentage increase in stresses compared to direct stresses at various degrees of construction inaccuracy in FIGURE: 6.13.

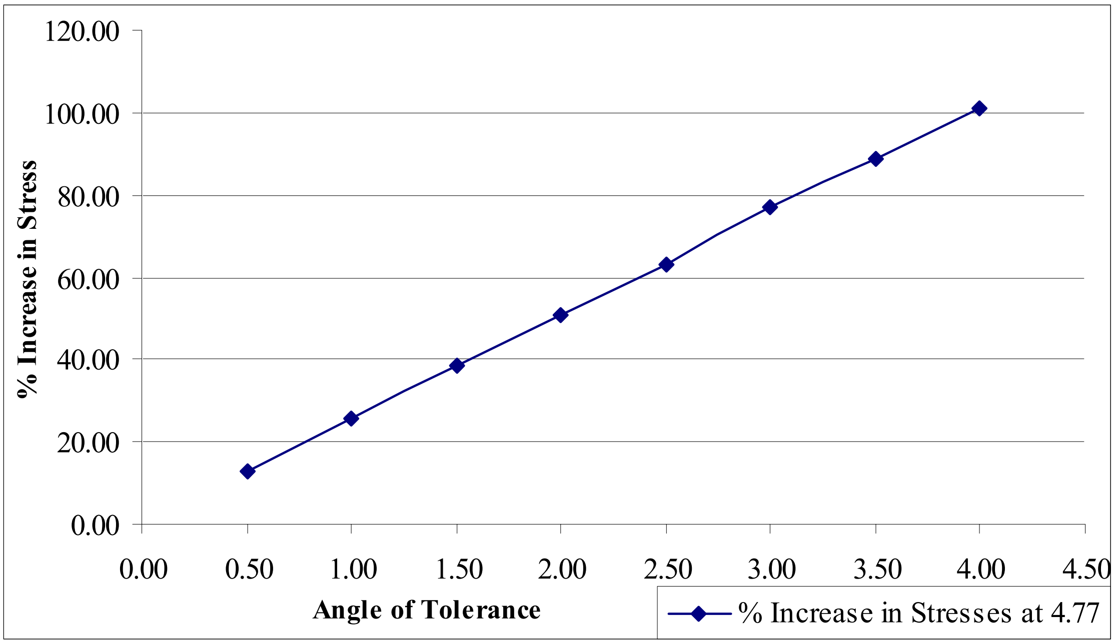


FIGURE 6.13: PERCENTAGE INCREASE OF STRESSES IN SHAFT WALL AFTER BULGING AT 4.77m

The other forces acting on shaft wall because of such inaccuracies are circumferential forces and shear forces. These forces are tabulated in TABLE: 6.9 at 4.77m from top of foundation.

TABEL: 6.9: CIRCUMFERENTIAL AND SHEAR FORCES IN SHAFT AFTER BULGING AT 4.77m

θ (degree)	Out of plumb (mm)	Circumferential Forces (N)	Shear Forces (N)
0.50	9.25	25.80	6.45
1.00	18.50	51.60	12.90
1.50	27.76	77.40	19.35
2.00	37.02	103.20	25.80
2.50	46.28	126.85	30.10
3.00	55.55	154.80	36.55
3.50	64.83	178.45	43.00
4.00	74.12	204.25	49.45

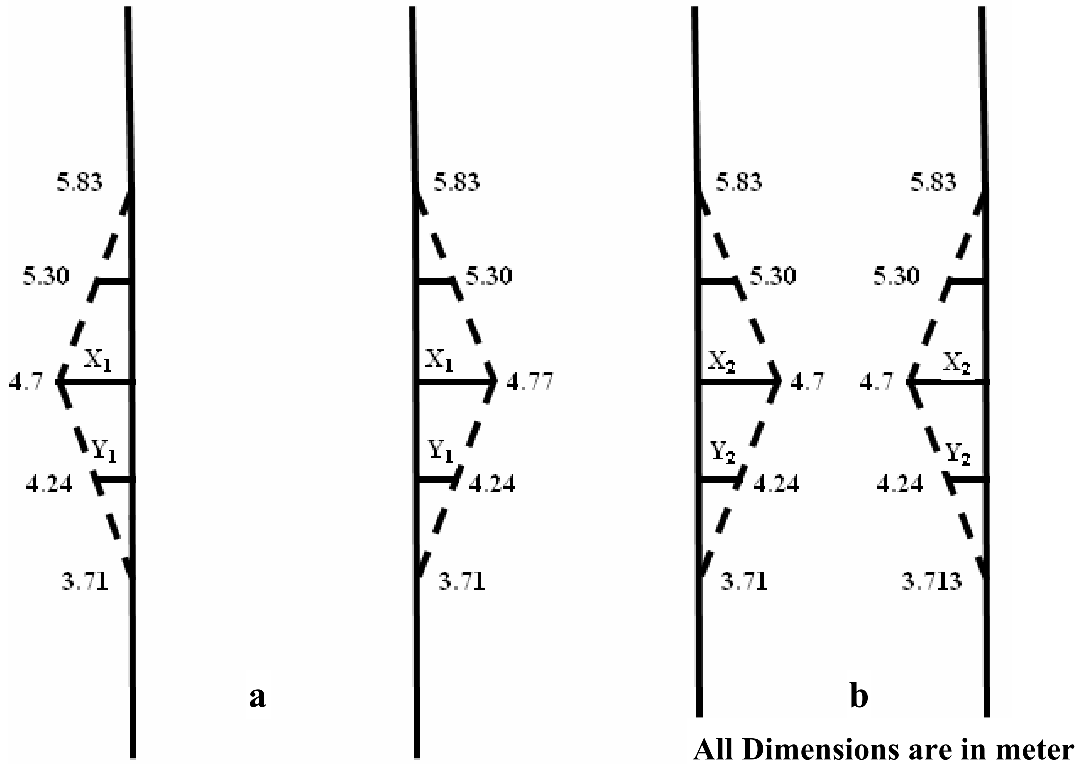
It is clear from the results tabulated in TABLE: 6.9 that the circumferential and shear forces in the shaft wall due to inaccuracies are very small and can be neglected.

6.5 EFFECTS OF CHANGE IN SHAPE OF SHAFT:

Similar to the previous case in this problem also results of lower layer of the shaft are presented because of the fact that the effects of construction inaccuracies at various levels remain localized more or less the same. Therefore in this problem the produced results are only of lower layer of the shaft, where maximum stresses are their and therefore effect of construction inaccuracy are also maximum at this level. It is worth to note that even though the additional stresses are high at lower level but the percentage increase of additional stresses remains more or less same for all levels.

As mentioned earlier in the modeling part, for introducing inaccuracies at 4.77m level the eccentricity needs to be introduced starting from one level below

(4.24m) and then it should be reversed for the upper layers to simulate actual site condition in the modeling as it is done for earlier case. In this case inaccuracies of more complex nature are considered. Here the shaft has bulged in the direction parallel to the X-axis and contracted in the direction perpendicular to the y-axis. As a result of this the shaft has taken the shape of an ellipse.



(a) SECTION PARALLEL TO X-AXIS

(b) SECTION PERPENDICULAR TO X-AXIS

FIGURE 6.14: BULGING AND CONTRACTION IN SHAFT WALL AT 4.77m

The model for such tolerances at 4.77m level is illustrated in FIGURE: 6.14. In the FIGURE: 6.14 X_1 indicates maximum eccentricity parallel to X-axis at level 4.77m for the respective degrees as mentioned in TABLE: 6.1 and Y_1 indicates the eccentricity needs to be incorporated one level below to simulate the actual site condition parallel to X-axis. Similarly X_2 indicates maximum eccentricity perpendicular to X-axis at level 4.77m for the respective degrees as mentioned in TABLE: 6.1 and Y_2 indicates the eccentricity needs to be incorporated one level below to simulate the actual site condition perpendicular to X-axis. Because of such construction inaccuracies the additional forces are illustrated in FIGURE: 6.1 and the forces are tabulated in TABLE: 6.10.

TABEL: 6.10: STRESSES IN SHAFT WALL AFTER BULGING AND CONTRACTION AT 4.77m

θ (degree)	Out of plumb e (mm)	Direct Stress (N/mm^2)	Moment due to eccentricity (kN.m)	Bending Stress (N/mm^2)	% Increase in Stresses (N/mm^2)
0.50	9.25	3.99	2.05	0.50	12.58
1.00	18.50	3.99	4.15	1.02	25.47
1.50	27.76	3.99	6.35	1.56	38.98
2.00	37.02	3.99	8.42	2.06	51.68
2.50	46.28	3.99	10.51	2.57	64.51
3.00	55.55	3.99	12.86	3.15	78.93
3.50	64.83	3.99	14.99	3.67	92.01
4.00	74.12	3.99	17.12	4.19	105.08

In TABLE: 6.10 the additional stresses in shaft at 4.77m from top of foundation are tabulated. It is very clear from the resultant stresses that as the degree of inaccuracy increases the stresses are increasing proportionately. The last column represents the percentage increase in stresses because of bending stresses compared to direct stresses on the shaft. A graph is plotted for the percentage increase in stresses compared to direct stresses at different degrees of construction inaccuracies in FIGURE: 6.15.

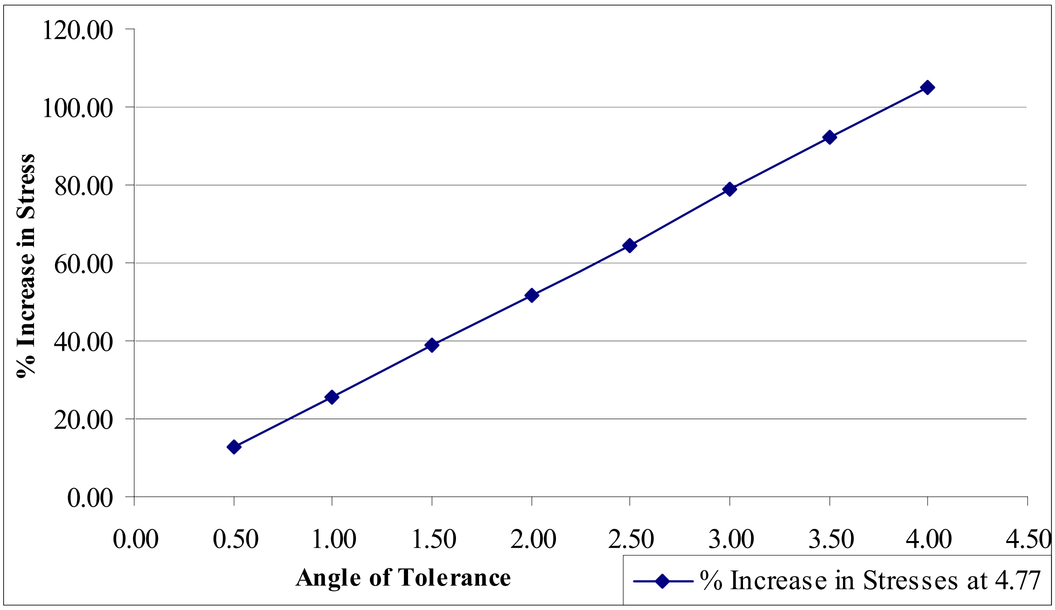


FIGURE 6.15: PERCENTAGE INCREASE IN STRESSES IN SHAFT WALL AFTER BULGING AND CONTRACTION AT 4.77m

The other forces acting on shaft wall because of such inaccuracies are circumferential forces and shear forces. These forces are tabulated in TABLE: 6.11 at 4.77m from top of foundation.

TABEL: 6.11: CIRCUMFERENTIAL AND SHEAR FORCES IN SHAFT AFTER DISPLACEMENT AT 4.77m

θ (degree)	Out of plumb (mm)	Circumferential Forces (N)	Shear Forces (N)
0.50	9.25	22.36	6.45
1.00	18.50	45.15	12.90
1.50	27.76	66.65	19.35
2.00	37.02	88.15	25.80
2.50	46.28	109.65	32.25
3.00	55.55	131.15	38.70
3.50	64.83	152.65	45.15
4.00	74.12	172.00	49.45

It is clear from the results tabulated in TABLE: 6.11 that the circumferential and shear forces in the shaft wall due to inaccuracies are very small and can be neglected.

6.6 SUMMARY:

➔ As one would expect because of inaccuracies, in form of verticality, circularity or uniformity in thickness, in construction of RCC circular shaft the structure has to experience additional forces and the same has been established.

➔ Analysis results for RCC shaft after introduction of different inaccuracies even for gravity loads has shown considerable increase in forces in form of bending moments. Bending stresses because of these moments are presented tabular form in TABLE: 6.12.

TABEL: 6.12: SUMMARY OF INCREASE IN STRESSES DUE TO INACCURACIES

Inaccuracy θ (degree)	% Increase in stresses due to moments (%)
0.5	12.00
1.0	25.00
1.5	38.00
2.0	50.00
2.5	63.00
3.0	76.00
3.5	88.00
4.0	100.00

➔ The other forces caused by such inaccuracies are circumferential forces and shear forces. The results show that the effects of inaccuracies are not significant for these forces. Circumferential and shear forces caused by inaccuracies are quite small and can be taken care by the minimum reinforcement itself. Therefore no further remedies are required for such forces and can be neglected.

➔ Locally some inaccuracies may occur in shaft. From the view point of design it is advisable to keep 10.00% to 15.00% margin between actual stresses and permissible stresses. On that basis the permissible inaccuracy in shaft wall shall not be more than 1 cm per 110 cm ($\theta = 0.5^\circ$) locally.

➔ Further it can be concluded that for the same percentage of additional permitted stresses the measured outside diameter at any section shall not vary from the specified diameter by more than ± 0.1% or 25mm whichever is less.

➔ Even though the Indian Code is silent about construction tolerances in shaft of elevated water tank, some other codes may be referred for the same. American code (ACI 371R -98) has given guide lines for construction tolerances for elevated water tanks.

- Vertical Alignment of center point:

  The center point of shaft shall not vary from its vertical axis by more than 0.15 % of height of shaft at the time of measurement, or 50 mm whichever is less. Locally, the center point of the shaft shall not be changed by more than 1 cm per 120 cm.

- Diameter:

  The measured outside shaft diameter at any section shall not vary from the specified diameter by more than 20 mm plus 0.1 % of the specified theoretical diameter.

- Wall thickness:

  The measured wall thickness shall not vary from the specified wall thickness by more than – 6 mm or +10 mm for walls which are 250 mm thick or less, or by more than –10 mm or + 20 mm for walls more than 250 mm thick. A single wall thickness measurement is defined as the average of at least four measurements taken over a 60 degree arc.

7 SUMMARY AND FUTURE SCOPE OF WORK

This chapter includes summary of conclusions and future scope of work.

7.1 SUMMARY:

➔ Because of opening load has to change its path and has to traverse through some alternative path. Therefore stress concentration can be seen on sides of the openings and nearly zero stress can be seen on top and bottom region of opening because of free surface in these regions. Available results confirm the same and stress concentration has been found in the region of ¾ the width of opening on both sides of the opening. The increase in stresses depends on size and location of opening.

➔ If stresses are within certain limit additional reinforcement can be provided to take care of stress concentration. When the stresses exceed beyond certain limit vertical and horizontal ribs shall be provided. After providing vertical and horizontal ribs in ¼ the width of the opening, considerable reduction in stresses in gravity loads have been found. Similar results have been found for the combination of gravity plus earthquake loads.

➔ Here for the selected case study the openings for inlet and outlet pipes are located below the natural ground level and near to the foundation level. Therefore the effect of these openings can be seen on the foundation raft. Practically the portion exactly below the openings has transferred very less loads on the foundation raft.

➔ Further it has been found that actual pattern of load transfer on foundation raft is quite different, when compared with the uniform pattern of load transfer on the same in presence of all the openings. It can be concluded that the basic assumption of uniform upward soil reaction for permanent loads in design of foundation raft is not correct when openings for inlet and outlet pipes are placed near to the foundation raft.

➔ Analysis results for RCC shaft after introduction of different inaccuracies for gravity loads only has shown considerable increase in stresses in form of bending moments. Because of these moments bending stresses are found to be at least 12.00% higher for inaccuracy of ($\theta = 0.5°$), when compared with stresses in shaft in its true position. Further the bending stresses are at least 25.00% higher for inaccuracy of ($\theta = 1.0°$). For the inaccuracy of ($\theta = 4.0°$) the bending stresses are found to be at least 100.00% higher, when compared with stresses in shaft in its true position.

➔ The other forces caused by such inaccuracies are circumferential forces and shear forces. The results show that the effects of inaccuracies are not significant for these forces. Circumferential and shear forces caused by inaccuracies are quite small and can be taken care by the minimum reinforcement itself. Therefore no further remedies are required for such forces and can be neglected.

➔ Locally some inaccuracies may occur in shaft. From the view point of design it is advisable to keep 10.00% to 15.00% margin between actual stresses and permissible stresses. On that basis the permissible inaccuracy in shaft wall shall not be more than 1 cm per 110 cm ($\theta = 0.5°$) locally.

➔ Further it can be concluded that for the same percentage of additional permitted stresses the measured outside diameter at any section shall not vary from the specified diameter by more than ± 0.1% or 25mm whichever is less.

➔ Two mass model suggested in proposed draft code IS: 1893 (Part II) gives realistic evaluation of dynamic properties of tank and shall be adopted.

➔ For the selected case study the increase in base shear and base moment is 260% in tank full condition and 330% in tank empty condition as per proposed draft code IS: 1893 (Part II), when compared with IS: 1893 – 1984. Available results show that the forces are unjustifiably high in proposed draft code IS: 1893 (Part II).

➔ The selected water tank of case study is situated in seismic zone III. If the same tank would have been in zone V then the seismic forces on the tank are 2.25 time higher then present forces. With such forces the design of supporting shaft and foundation raft is impractical to implement and may indirectly ban shaft supported elevated water tanks.

➔ Response Reduction Value (R) suggested for OMRF is 3 while suggested T value for shaft supported water tank is 1.8 which gives and impression that shaft supported elevated water tanks are much weaker then OMRF. Therefore suggested R value in the draft code seems to be impractical and the issue is still under discussion by designers.

7.2 FUTURE SCOPE OF WORK:

➔ A practical and justifiable R value for shaft supported elevated water tank should be derived.

➔ Buckling analysis of shaft should be carried out to know buckling load capacity of shaft and to validate formulas derived in various books (Plain and Reinforced Concrete, Vol. 2, Jai Krishna and Jain) for the same.

➔ Load distribution around openings should be studied using better software to understand the stress levels around openings of circular and other shape.

➔ Inaccuracies for wall thickness in shaft of elevated water tank shall be studied and tolerance limits for the same should be derived.

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APPENDIX A LATERAL LOAD DISTRIBUTION

Lateral force due to earthquake or wind produce axial tension or compression at different locations of a shaft as well as shear force in a shaft. Magnitude and sign of axial forces produced at different locations depend upon their positions with respect to axis of bending, similar to distribution of bending stresses. Likewise magnitude of shear force at different locations depends upon its position from neutral axis of the section, similar to distribution of shear stresses across a cross section. Finally the design of shaft section is based on worst combination of axial force, bending moment ad shear force.

Let,

N = Total number of shaft nodes

A = Cross sectional area of each element

r = Radius of Shaft

Therefore,

Total Cross Sectional Area = N.A

Polar Moment Of Inertia = Ip = N.A.r^2

Moment of Inertia = Ix = Iy = Ip/2

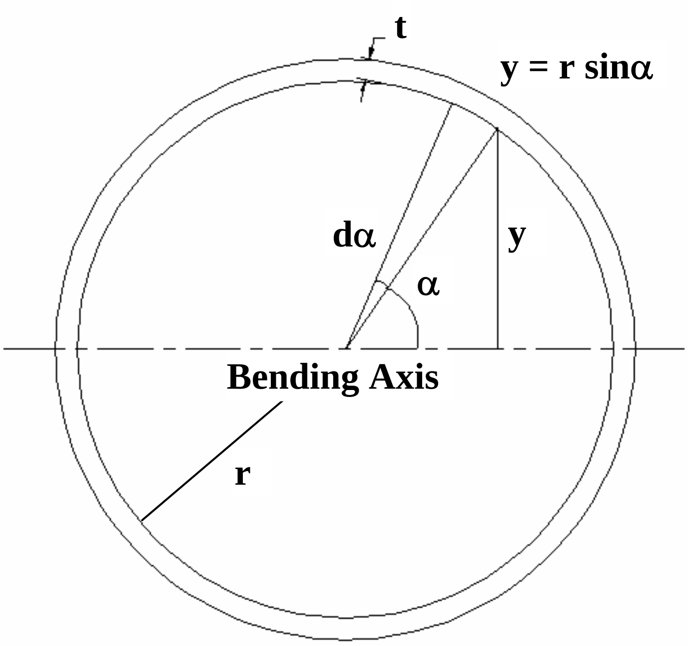

FIGURE: A: 1: ANALYSIS OF SHAFT FOR LATERAL FORCES

i. Shear Force in shaft due to lateral force (S):

Shear force at any node due to lateral force,

$q = (S\ a\ y) / (I\ b)$

Where,

S= Lateral force

$I = Ix = Iy = N.A.r^2/2$

$b = 2.t / \cos\theta$

$\int a\ y$ = sum of moment of elemental areas about neutral axis

$= \int (r.d\alpha.t)\ (r.\sin\alpha)$

$= 2.r^2.t.\cos\theta$

Substituting all the values in equation

$q = 2.S.\cos^2\theta / N.A$

ii. Axial Forces in shaft due to moment (M):

Axial force at any node of shaft due to bending moment

= Stress * Area

$= (M.y / Ix) * A$

Where,

$Ix = \sum Ay^2$

$= A \sum (r\sin\theta)^2$

$= Ar^2 \sum (\sin^2\theta)$

For foundation of shaft either full raft or an annular raft can be provided. The latter has the advantage that because of a higher uniform soil pressure under permanent loads, it minimizes possible gradual tilting of a foundation when the structure is subjected to lateral loads. Analytical procedure for direct load and external moment for both foundation systems is as follows.

a) Annular Raft:

Analysis of such raft under the effect of direct load and due to external moment is explained as follows.

i. Radial and Tangential Moments due to Direct Load:

Such load is exerted by the dead weight of a shaft. (FIGURE: B: 1) and it is assumed to cause uniform soil reaction. For analysis purpose, the raft is divided into two portions at r = βa. Equations for inner and outer portions can be written down as

Constants:

$Y_1 = -\beta^4 + (8a^2 \beta^2 \ln \beta) - (\beta^2 Y_2) - (Y_3 \ln \beta)$

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

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

$Y_4 = -8 \alpha^2$

$Y_5 = -\beta^4 + (8 \beta^2 \ln \beta) - (\beta^2 Y_6) - (Y_7 \ln \beta)$

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

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

$Y_8 = -8.00$

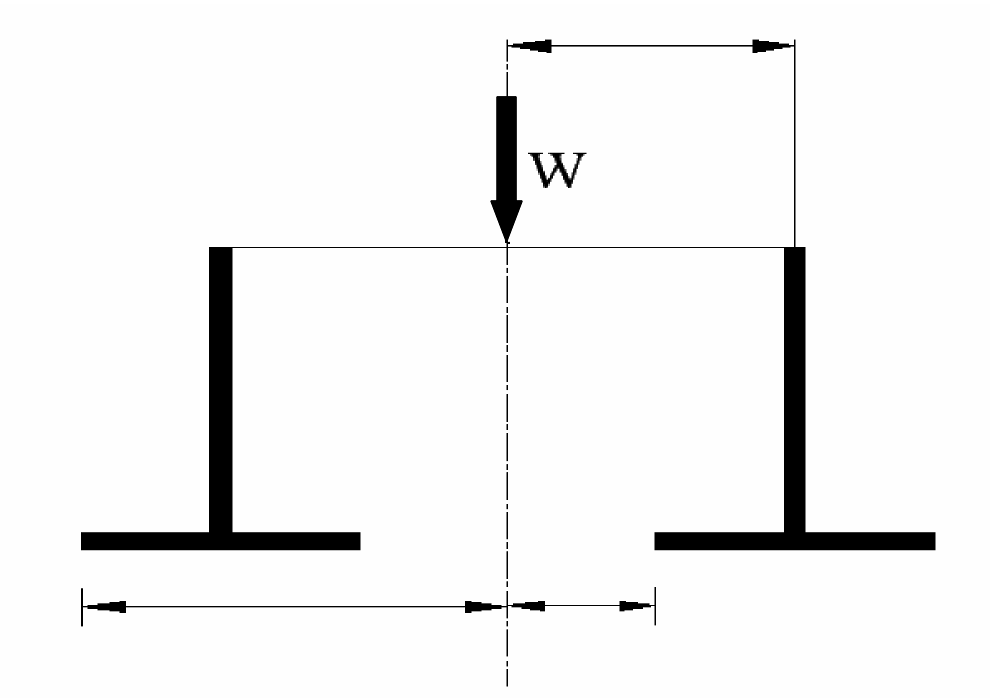

FIGURE: B: 1: UNIFORM AXIAL FORCE ON SHAFT

Moments:

For $f < \beta$

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

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

For $f > \beta$

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

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

Note: Results are tabulated in TABLE: 4.0.

ii. Radial and Tangential moments due to moments:

External moment M due to transverse loads from either wind or earthquake effects is considered to be transmitted to the raft at a radius $r=\beta a$ (FIGURE: B: 2) with a force varying linearly across the shell diameter. Equations for inner and outer portions can be written down as

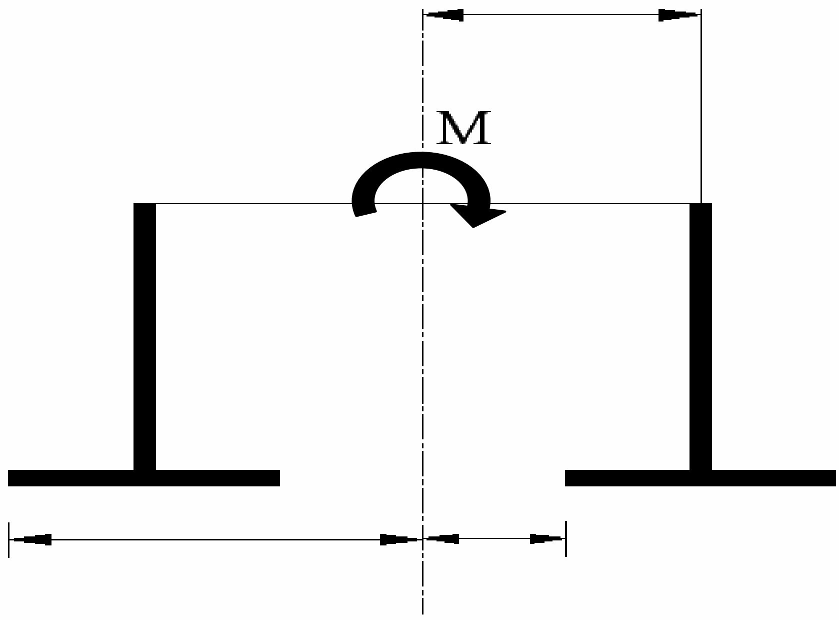

FIGURE: B: 2: EXTERNAL MOMENT ON SHAFT

Constants:

$Y_1 = -\beta^4 - (\beta^2 Y_2) - (Y_3 / \beta^2) - (Y_4 \ln \beta)$

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

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

$Y_4 = 12 \alpha^4$

$Y_5 = - \beta^4 - (\beta^2 Y_6) - (Y_7/ \beta^2) - (Y_8 \ln \beta)$

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

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

$Y_8 = 12.00$

Moments:

For $f < \beta$

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

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

For $f > \beta$

$M\ ri = (qa^2 / 192) * (-20.6 f^3 -6.3 f Y_6- (1.7/ f^3) Y_7- Y_8 (1.15 / f)) * \cos \theta$

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

b) Full Raft:

Often, a complete raft is provided instead of an annular one. The procedure of analyzing such a raft is explained as under.

i. Radial and Tangential moments due to Direct Load:

Constants:

$Y_1 = (1.96\ \beta^4) + (8\ \beta^4 \ln \beta) + (2.52\ \beta^2)$

$Y_2 = -2.52 - (8 \ln \beta) - (2.96\ \beta^2)$

$Y_3 = 0$

$Y_4 = 0$

$Y_5 = (16\ \beta^4 \ln \beta) - (5.48\ \beta^2) + (1.96\ \beta^4)$

$Y_6 = 5.48 - (2.96\ \beta^2)$

$Y_7 = -8\ \beta^2$

$Y_8 = -8.00$

Moments:

For $f < \beta$

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

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

For $f > \beta$

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

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

ii. Radial and Tangential moments due to moments:

Constants:

$Y_1 = (-0.19\ \beta^4) + (5.46\ \beta^2) - 3$

$Y_2 = (3 / \beta^2) - 5.46 - (0.81\ \beta^2)$

$Y_3 = 0$

$Y_4 = 0$

$Y_5 = (-0.19\ \beta^4) + (5.46\ \beta^2) - 3 - (12 \ln \beta)$

$Y_6 = -5.46 - (0.81\ \beta^2)$

$Y_7 = 3\ \beta^2$

$Y_8 = 12.00$

Moments:

For $f < \beta$

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

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

For $f > \beta$

$M\ ri = (qa^2 / 192) * (-20.6\ f^3 - 6.3\ f\ Y_6 - (1.7/ f^3)\ Y_7 - Y_8\ (1.15 / f)) * \cos\theta$

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