Design and Analysis of compartment silo

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Design and Analysis of compartment silo

Major Project

Submitted in partial fulfillment of the requirements

For the degree of

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

By

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May 2006

CERTIFICATE

This is to certify that the Major Project entitled "**Analysis and Design of Compartment Silo**" submitted by **Ms. Rachana Bhatt (04MCL001)**, towards the partial fulfillment of the requirements for the degree of **Master of Technology in Civil Engineering** (**Computer Aided Structural Analysis and Design**) of Nirma University of Science and Technology, Ahmedabad is the record of work carried out by her under my supervision and guidance. In my opinion, the submitted work has reached a level required for being accepted for examination. The results embodied in this major project, to the best of my knowledge, haven't been submitted to any other university or institution for award of any degree or diploma.

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ABSTRACT

Working to the fast pace of development that is to be taken care of by professional engineers and globalization of professional services to be rendered, it becomes important to appraise them in the area of design for industrial structures, which has a wide spectrum to be dealt with.

There has been a rapid change due to industrial advancement requiring different type of structures like bunkers and silos. Bunker and silo is one of the storage structure required for industrial plants to store various types of materials such as coal, grains, cement etc. Hence it is necessary to appraise regarding the design procedures for such structures which includes the study of codal provisions leading to analysis and design along with detailing of the same.

From last few decades, there is a considerable advancement in manufacturing technologies for various industries. With progressive increase in size of raw and finished material for manufacturing units there is a corresponding increase in size of storage unit also.

Particularly in cement manufacturing industries due to the changed market conditions regarding the availability and supply of cement, it is now becoming common to produce different types of cement in the same cement factory, so as to improve marketability and profitability. This has necessitated introduction of multi-section silo, which has the facility of storing different types of cement and using the same extraction system.

One of the major problems faced in the cement factory is that of cement getting set in the silos, choking the extraction system. As the silo is needed for continuous storage and extraction, maintenance becomes problematic. In a multi-section silo, same type of cement can be stored in more than one compartment, and while storing and extracting operations are done in one section, the other sections could be emptied, cleaned and maintained.

The study undergone is the step in the direction to give relevant technical information about the detailed designing methods using STAAD.Pro.2004 for the analysis of ***6 compartment silo.***

It is difficult to get all necessary information in one compact volume and hence, an effort has been made to give relevant technical information, illustrating the theoretical background and codal provisions along with detail design procedures, normally to be followed. The given information will serve as a first reference to deal with design of such structures; however, one should go through various other references to have an in-depth understanding to enable to design efficiently such structures.

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NOMENCLATURE

D	Diameter of Silo.
H	Height from the cone apex level to the Silo storage level.
r	Radius of the Silo.
W	Bulk density of material.
Φ	Angle of internal friction/repose.
δ	Angle of wall friction
μ_f	Co efficient of wall friction while wall filling
μ_e	Co efficient of wall friction while wall emptying.
λ	Pressure ratio.
λ_f	Pressure ratio while filling.
λ_e	Pressure ratio while emptying.
A	Area of one compartment.
R	Hydraulic mean radius.
P	Perimeter of one compartment.
Cd	Overpressure factors.
Pz	Janssen pressure at depth Z
Pdes.	Design pressure = CdPz
d	Diameter of one compartment.
μ	Coefficient of friction between cement particles.
μ´	Coefficient of friction between cement particles and concrete wall.
t	Wall thickness in m.
Ti	Inside temperature.
To	Outside temperature.
Ca	Surface conductance of concrete to air = 15 kcal/m²h°c.
Cc	Thermal conductivity of reinforced concrete = 1.75 to 2 kcal/m²h°c.
Cs	Surface conductance of concrete to storage materials 2 kcal/m²h°c.
k	Co efficient of heat transmission.
Δ_T	Temperature difference across silo wall.
ε_t	Co efficient of thermal expansion of concrete.

$M_{\Delta T}$	Moment due to temperature difference across the silo wall
Ec	Modulus of elasticity of concrete
Mu	Ultimate bending moment
Pt	Calculated percentage of reinforcement.
V	Shear force
d	Effective depth of section
D	Total depth section
Wcr	Crack width in silo wall in cm.
β	Co efficient depending upon silo foundation system
σ_{sa}	Actual steel stress under permanently acting loads
δ	A factor depending upon bond characteristics of steels (0.09 for plain bars and 0.05 for deformed bars)
As	Area of reinforcement in silo wall at the height under consideration cm^2
Act	Concrete area in tension per unit height of wall in cm^2/m.
Po	As/Act = Geometric percentage of tensile reinforcement with respect to the concrete area in tension
φ	Diameter of reinforcing bar in cm.
M.F.	Multiplying factor
σ_0	Uniaxial permissible stress in concrete
F	Reference stress
Mr	Meridional moment
$M\theta$	Tangential moment
j	Constant in working stress method
Y_1 to Y_8	Constants taken from tables for foundation design By Tall Chimney by S.N.Manohar.

1. INTRODUCTION

1.1 GENERAL

Structure for the storage of solids is generally referred to by the name Bin. A bin is simply an upright container and the name includes shallow containers known as bunkers and tall structure known as silo. Steel bins are usually rectangular or circular in cross section and are supported on column as shown in figure: 1.1

Reinforced concrete bins are usually rectangular but tall silos are cylindrical as shown in figure 1.2 Normally 500 to 2000 tones of granular material is stored in a single bin and large quantities of the order of 10000 tones or more are to be stored, the material is divided into two or more bins forming a continuous nest or Battery as shown in figure1.3. In the multi-bin configuration, even the intermediate bin forming the intestinal space and the pocket bin formed at the end are used for the storage. Prestressed concrete silos are also extensively used for the storage of cement, and clinker, alumina, sugar etc. For the storage of powdery material in plastic industry, fiber-reinforced plastic silos are used.

Vertical storage of materials has the following advantages:

(a) Handling is more economical
(b) Simultaneous and continuous loading and unloading are possible
(c) Maintenance costs are lower
(d) Moisture free storage is possible
(e) Dust losses are reduced
(f) Space is utilized more efficiently.

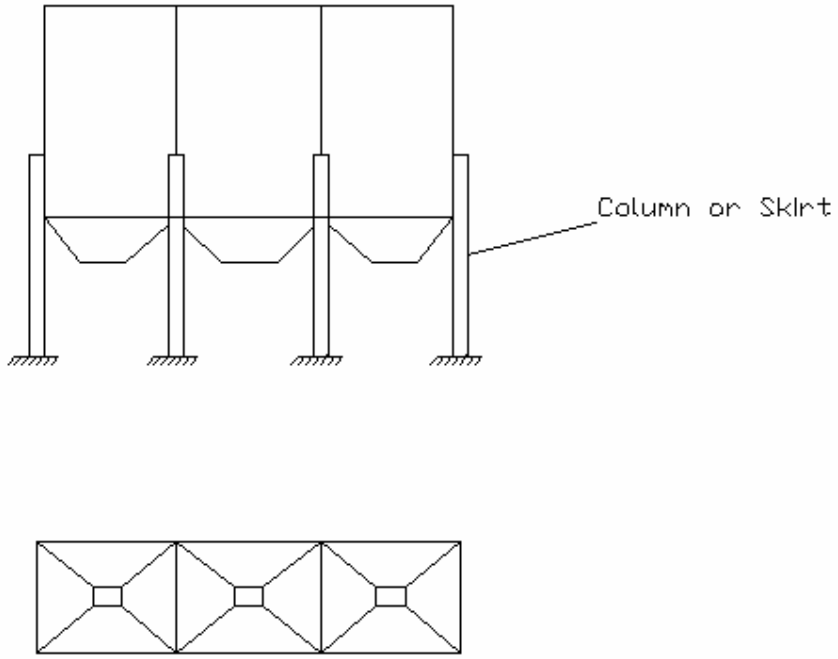


Fig 1.1

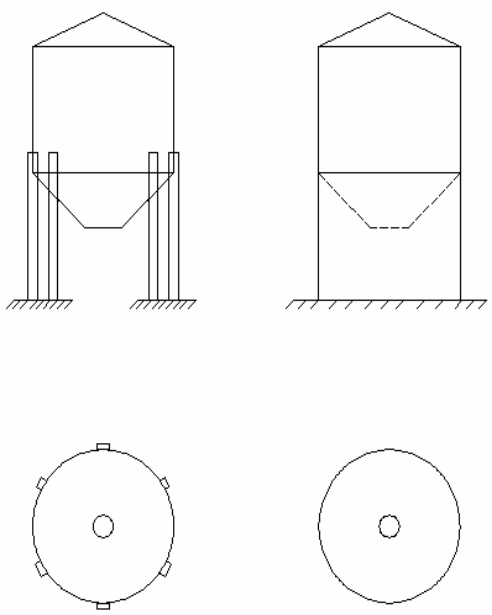


Fig 1.2

FIGURE: 1.1 AND1.2 GENERAL ARRANGEMENTS OF BUNKER AND SILO

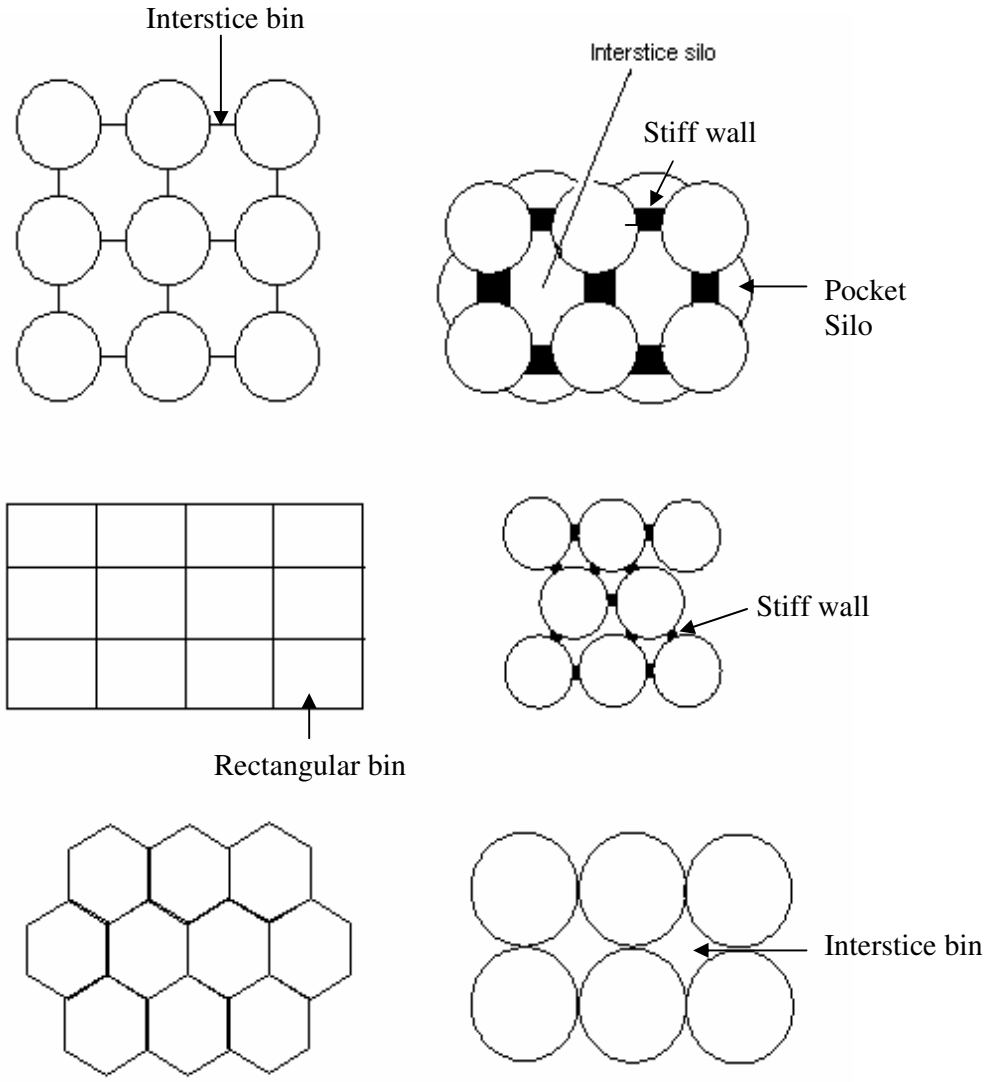


FIGURE 1.3 DIFFERENT GEOMETRICAL SHAPES OF SILOS

1.2 CLASSIFICATION AND DEFINITIONS

The terms bin for silo and bunker have different meanings in different part of world and may vary from author to author. In the United State the term bin generally includes both silos and bunkers. Deep bins are silos and shallow bins are bunkers.

The propositions of a bin especially the ratio of material depth to least material dimension affect the behavior of stored material both at rest and during discharge. Assuming that bin geometry affects pressures, to select the proper basis for the pressure computation the bin is classified either as a silo (deep bin) or a bunker (Shallow bin).

Accurate classification may soon be feasible but presently the following methods are widely used in practice:

a) Empirical approximation – Practiced by engineers and also supplemented by Soviet code:

   H > 1.5 A where A is diameter for circular bin and shorter length for rectangular or square bin

   If storage structure in question satisfies above, it is consider as a silo or otherwise it is a bunker.

b) An approximation based on the position of the plane of rupture:

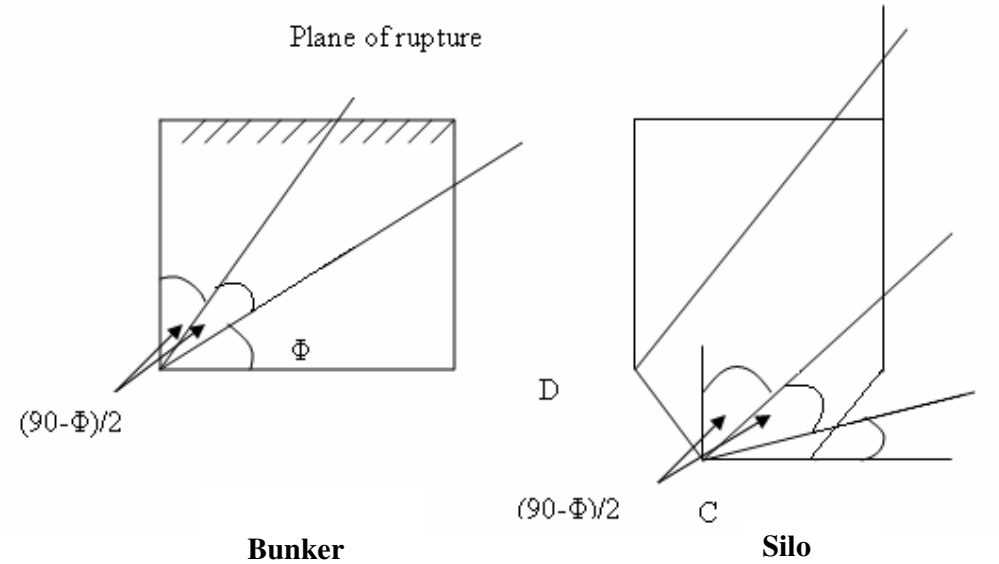


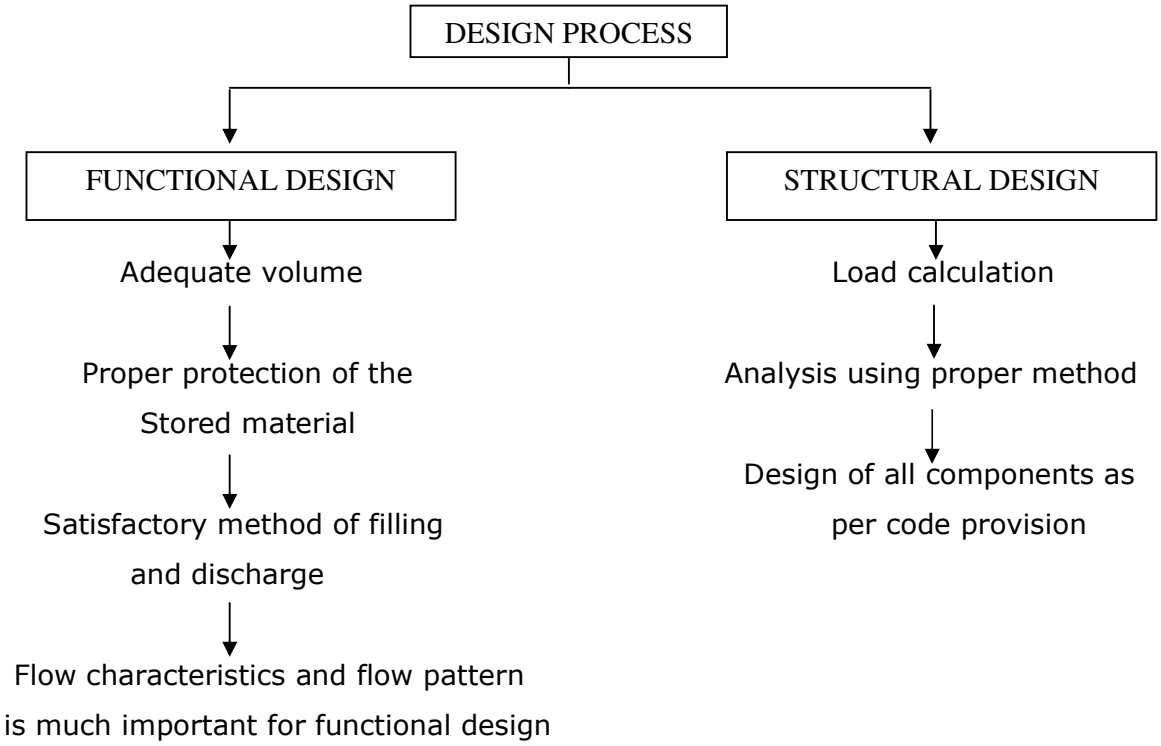
FIGURE: 1.4 BINS OF TWO DIFFERENT DEPTHS

Figure 1.4 shows bins of two different depths. The plane of rupture is determined by the Coulomb theory. If the rupture plane intersects the top surface of the stored material, the bin is a bunker otherwise it is a silo.

However engineers do not agree on the location of the plane of rupture. Some would start the plane at the bottom of the hopper, point C of figure 1.4 while other would pass it through point D, at the bottom of the vertical wall. Thus by one interpretation the bin would be a silo; by the other a bunker. This is recognized by the ACI 313 standard, which for simplicity and without significant error allows all vertical containers, regardless of their height to depth ratio to be considered as silo. Therefore under ACI 313 method of pressure calculation for silo is also used for bunkers.

1.3 DESIGN PROCESS OF BINS

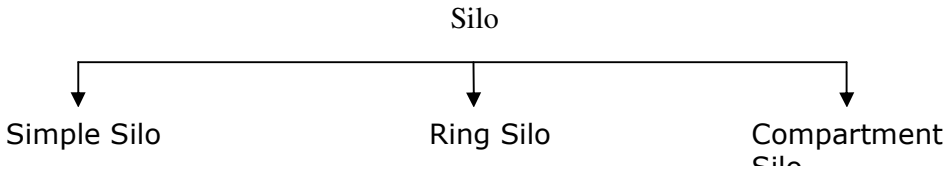
1. FUNCTIONAL DESIGN
2. STRUCTURAL DESIGN



FLOWCHART: 1.1 DESIGN STEPS

1.4 TYPES OF SILOS

As per the storage point of view different types of silos can be descritised in the following manner.



SIMPLE SILO

It is elevated storage structure having facility to store just one type of material with single extraction system.

RING SILO

It is elevated storage structure consisting of outer & inner silo having facility to store two different types of material with extraction system for each silo.

COMPARTMENT SILO

It is elevated storage structure having facility to store different types of material in a segmented silo with extraction system for each compartment. For storing different kinds of materials, compartment silos are found to be economical in relation to silo capacity, structural dimensions and general arrangement of machinery equipments. In India, the storage of cement is usually being done in a conventional single silo and different types of system are employed to extract cement. So far, improvement in the design of silos was limited to employing better extraction system and to minimize the dead storage in the silo. Due to the changed market conditions regarding the availability and supply of cement, it is now becoming common to produce different types of cement in the same cement factory, so as to improve marketability and profitability. This has necessitated introduction of multi-section silo, which has the facility of storing different types of cement and using the suitable extraction system. One of the major problems faced in the cement factory is that of cement getting set in the silos, choking the extraction system. As the silo is needed for continuous storage and extraction, maintenance becomes problematic. In a multi-section silo, same type of cement can be stored in more than one compartment, and while storing

and extracting operations are done in one section, the other sections could be emptied, cleaned and maintained. The civil and mechanical costs were almost the same as compared with previous designs.

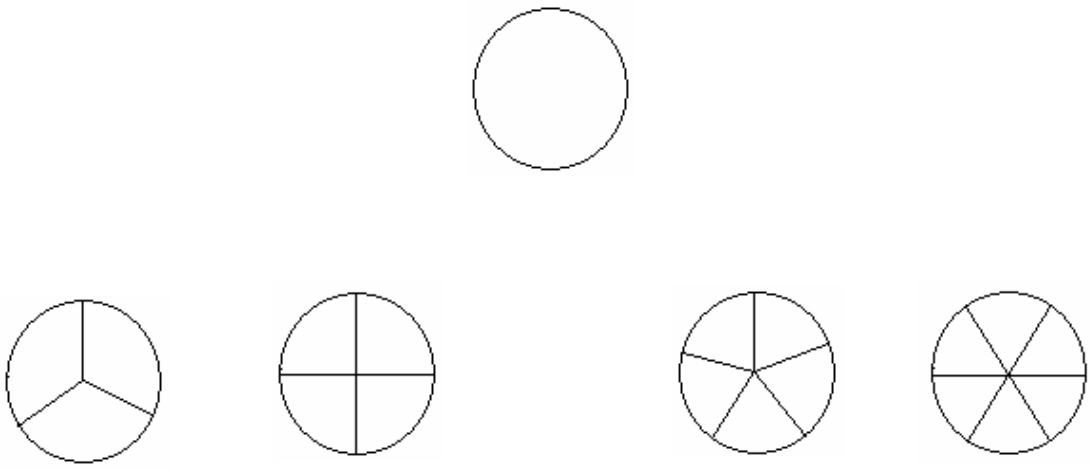


FIGURE: 1.5 [A] DEVELOPMENTS OF COMPARTMENT SILOS FROM SINGLE CELL SILO

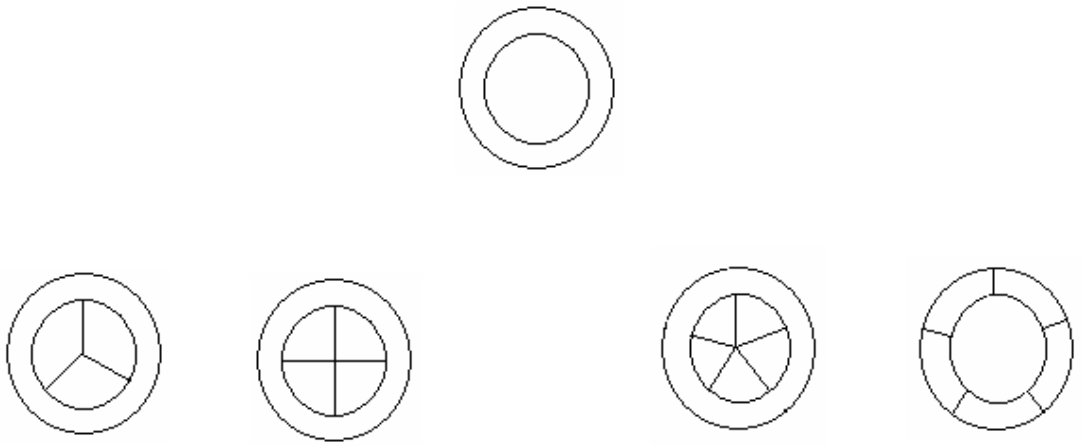


FIGURE: 1.5 [B] DEVELOPMENTS OF COMPARTMENT SILOS FROM RING SILO

2. LITERATURE REVIEW

2.1 GENERAL

Literature survey is carried out to review various criteria which are to be considered for structural design of silos. Such applications include stresses developed in wall for different stored material, analysis of silo bottom cone by FEM and tower silo foundations. In literature survey, main emphasis is given on the published papers that describe the implementation of silo structure by various structural problems.

2.2 STORAGE OF POWDERS AND BULK SOLIDS IN SILOS [1]

(Dietmar Schulze, www.dietmar-schulze.com)

Problems with storage of bulk solids in bins and silos can be avoided if they are designed with respect to the flow properties of the bulk solid which has to be stored. This paper shows the basic rules for the design of silos.

Figure 2.1 shows silos and the pressures and stresses, respectively, acting in the silos. While the pressure (for fluids we will use the word "pressure") would increase linearly downwards if the silo would have been filled with a fluid (a), the course of the vertical stress (for bulk solids we will use the word "stress") in a silo filled with a bulk solid is rather different (b,c): In the latter case in the vertical (cylindrical) section of the silo the vertical stress increases in a digressive way. If the height to diameter ratio of the silo is sufficiently large (usually: > 3), a constant vertical stress is attained. This means that the vertical stress will not increase further even if the filling height is much larger.

The reason for this course is the shear stresses acting between the bulk solid and the silo walls even if the bulk solid is at rest. Due to the shear stresses, the silo walls carry a part of the weight of the bulk solid. A method for the calculation of the stress course in the vertical section was derived by Janssen already in 1895. The stresses acting in a hopper are different from those in the vertical section.

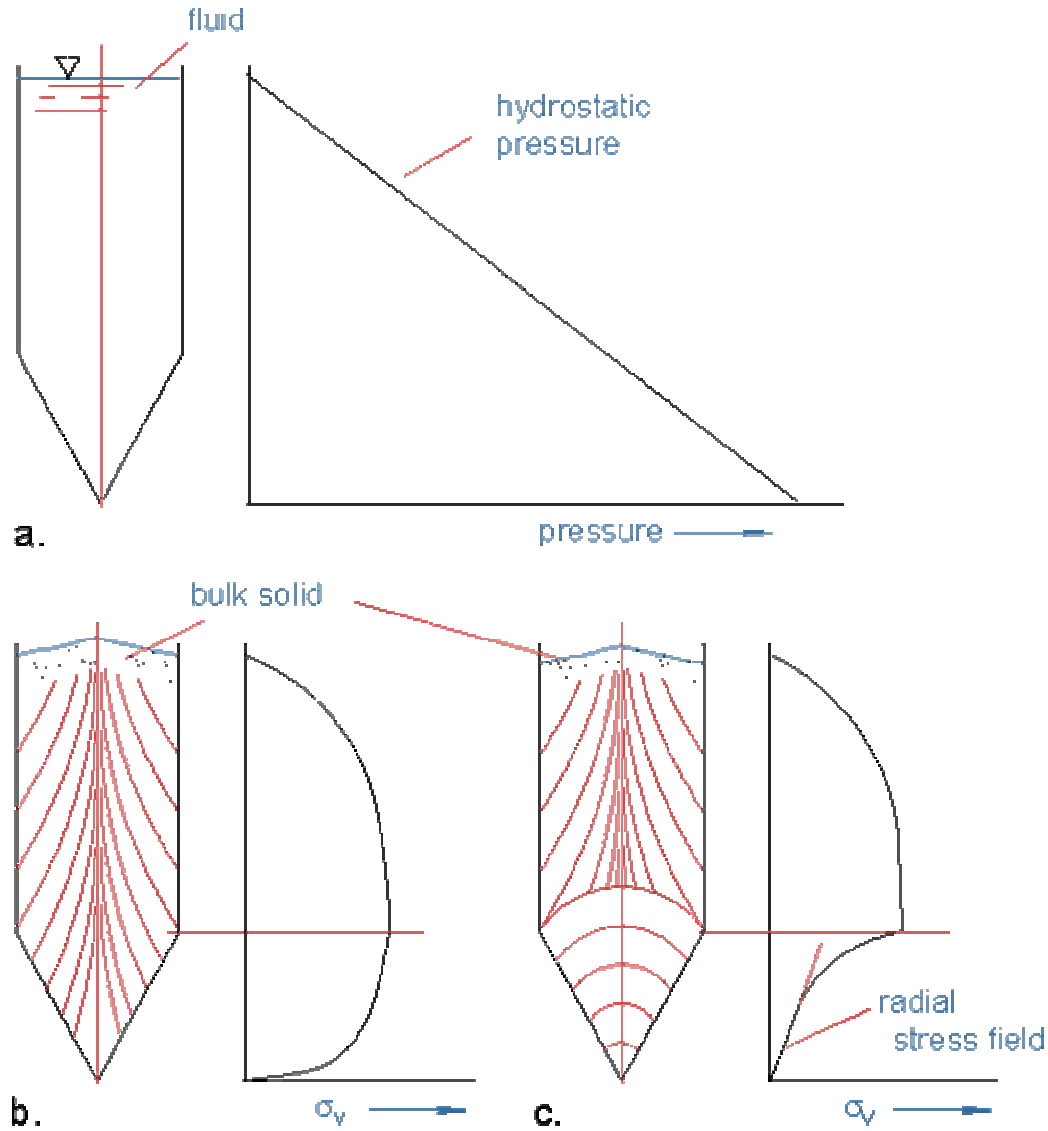


FIGURE 2.1: A. PRESSURE IN A SILO FILLED WITH A FLUID (IMAGINARY); B. VERTICAL STRESS AFTER FILLING THE SILO WITH A BULK SOLID; C. VERTICAL STRESS AFTER THE DISCHARGE OF SOME BULK SOLID

Just after filling an empty silo, the so called filling stress state (also: active stress state, figure 2.1b) prevails, where the vertical stress in the hopper decreases less in the upper part of the hopper and then more near the imaginary hopper apex. As soon as some bulk solid is discharged for the first time after

filling, the stresses in the hopper change and the so-called emptying stress state (also : passive stress state prevails, figure 1c). When flowing downwards in the hopper, the bulk solid is compressed in the horizontal direction so that the walls of the hopper carry a larger part of the weight of the bulk solid and, hence, the vertical stress in the lower part of the hopper is clearly smaller than after filling.

In the emptying stress state the vertical stresses in the lower part of the hopper are nearly proportional to the distance to the imaginary hopper tip or, in other words, the stresses are proportional to the local hopper diameter. This linear course of stress is called the radial stress field. In principle, in the vertical section of the silo the stresses remain unchanged at discharge.

The design of silos in order to obtain reliable flow is possible on the basis of measured material properties and calculation methods. Because badly designed silos can yield operational problems and a decrease of the product quality, the geometry of silos should be determined always on the basis of the material properties. The expenses for testing and silo design are small compared to the costs of loss of production, quality problems and retrofits.

2.3 STRESSES IN SILO [2]

(Dietmar Schulze, www.dietmar-schulze.com)

Knowledge of the stresses acting in silos is important for many applications:

- Silo design for strength (e.g. DIN 1055 part 6)
- Silo design for flow
- Loads on feeders and inserts
- Driving torque of feeders
- Design of silos in which a specific maximum stress is not exceeded (e.g. to avoid vibrations, particle attrition or extreme time consolidation)

The calculation methods used by an engineer who is interested in avoiding flow problems and in feeder design differ from the calculation methods of a civil engineer (e.g. DIN 1055 part 6) who is interested in the stability of the silo structure. The civil engineer would choose the parameters for calculating silo stresses so that the major part of the load from the bulk solid is carried by the

silo walls, whereas the engineer who has to calculate the feeder load and the required driving power would assume that the silo walls carry only a minor part of the load of the bulk solid. The stress distribution across the periphery of the silo is another example of the different points of view: whereas a strong irregular distribution of the stresses on the silo wall is quite unimportant for the design of a feeder, these different stresses cannot be neglected for the structural design of the silo walls.

For the stress calculation, a bulk solid is considered as continuum instead of single particles. Because of this the methods of continuum mechanics can be applied. If different sloped cuts through an element of bulk solid are considered, it can be seen that different shear and normal stresses are acting at different cutting planes as shown in figure: 2.1. The stresses σ_h and σ_v acts in different directions differ from each other. In a bulk solid there is one direction where the maximum normal stress is acting. This maximum normal stress is called major principal stress σ_1. The minimum normal stress which acts perpendicular to σ_1 is called minor principal stress σ_2. In contrast to a fluid, a bulk solid at rest can transmit shear stresses. While the pressure in a container filled with a fluid increases linearly with the depth (figure 2.2), the weight of the bulk solid in a silo is carried partly by the silo walls because of the shear stresses (friction at the silo wall) so that the stress does not increase linearly with the depth like the pressure of a fluid.

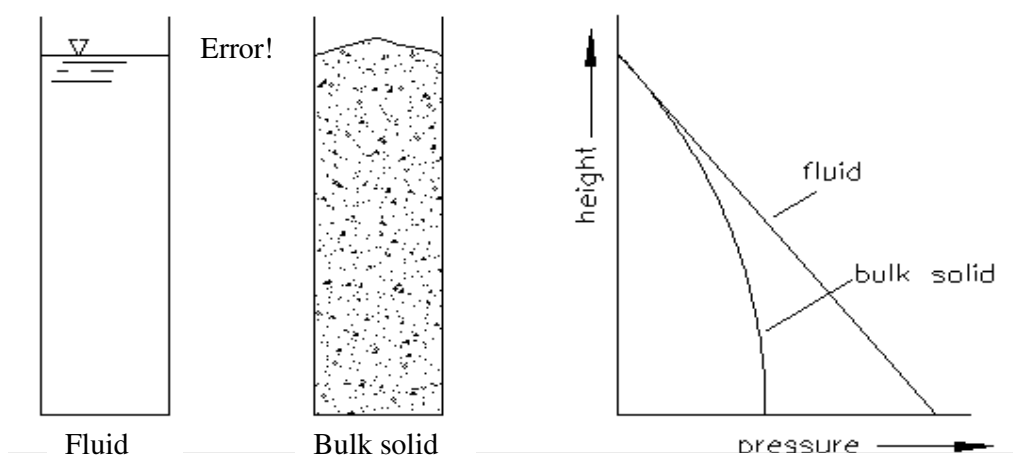


FIGURE 2.2: PRESSURES IN FLUIDS AND STRESSES IN BULK SOLIDS (IN PRINCIPLE)

2.4 FINITE ELEMENT ANALYSIS OF INVERTED CONE FOR 20000 T CAPACITY CEMENT SILO. [3]

(A K Samanta, IE (I) Journal, Vol 84, August 2003)

The capability of analyzing complex structures to a greater extent is possible nowadays with the introduction of advanced numerical methods like finite element analysis, and that of modern computers software like STAADPro. In this paper, the author has compared the results of software/FEM analysis with those calculated following standard formulae. In this case it is a 20,000 t RCC cement silo with inverted right circular cone (12326 mm high) having a ring beam at the base of the cone. In static condition, the cone carries material pressure (vertical as well as lateral) along with its self-weight. As a result meridional and hoop stresses (internal) develop in the cone shell, which are usually compressive in nature. The inverted cone has been modeled in STAADPro with hinge support at the column locations for the simplicity of analysis. The entire vertical height of the cone has been divided into 13 equal divisions and the perimeter has been divided into 48 equal divisions. The external loads have been calculated at each node using the standard formulae and than loads are applied as a nodal load. In second method, analysis of inverted cone is done using standard formulae of Safarian and Harris and the shell stresses at the base level are calculated. Result of both methods is compared and final conclusion is made.

The proper modeling of a problem, where actual support condition is configured, can only depict the true nature of deformation as well stress variation. Since the base of the inverted cone is supported on a number of columns having very less area of cross section, the column can not restrict the possible extension of the ring at the openings. Also the column can not resist the rotation at the supporting joints and as a result, the part of the periphery at the same level between the supporting columns is subjected to considerable deformation. The results by standard formulae have been obtained considering the base continuously supported at the base and also the design stresses are much on conservative side which requires almost 25% higher reinforcement than that following FEM analysis. So, FEM is much better tool to understand its behavior

depending on the height to base diameter ratio, non-uniform loading pattern provided a proper supporting condition is simulated.

2.5 TOWER SILO FOUNDATIONS [4]

(M. Bozozuk, CBD-177, May 1976)

Tower silos, tall, bullet-shape farm structures, are used to store forage crops for feeding cattle. The first tower silos, constructed about 40 years ago, were small, predominantly wooden structures although a few were made of concrete. Since then they have increased in number, size, and capacity, most made of concrete but some of steel. In 1974 more than 3,000 tower silos were erected in Ontario and Quebec, representing an investment of over $30 million by the agricultural industry in these provinces alone. Some recent structures are fully automated exceed 100 ft (31 m) in height, and have a storage capacity of more than 2500 tons.

Over the years a silo builder has improved the design and construction of the above-ground portion of silos but, in contrast, very little has been done to improve the foundation. Towers have generally been erected on foundations prepared by the farmer, who often lacked the necessary technology for adequate design and construction. The practice was reasonably successful when silos were small because the bearing pressures applied to the soils were low. As bigger silos were erected, however, and the applied foundation pressures approached the bearing capacity of the soils, many structures settled considerably, some tilted, and some overturned completely. This Digest outlines the problem and indicates the need for a soil investigation to determine the allowable bearing capacity and compressibility of the soil and thus enable proper foundation design. Many tower silos constructed on clay soils have ring-shaped concrete foundations. To reduce costs, concrete floors are seldom provided. When the silos are filled with silage, part of the load is transmitted through the cylindrical walls to the footings and the remainder is carried directly by the soil inside the ring foundation. The underlying clays compress vertically under the weight of the loaded structure in such a way that the applied loads are distributed uniformly to the soil over the whole area enclosed by the circular foundation.

Non-uniform placement of silage during filling has caused many problems. When the load from the weight of the silage is off center the pressure bulb will be distorted, as shown in Figure 2.3. Strong winds acting on a tall silo can produce the same effect. The local overstressing of the foundation soil may cause tilting, and unless the problem is remedied it may increase with time until the silo overturns.

Pressure bulbs will overlap, as shown in Figure 2.3(A), if two or more silos are constructed too close to each other. Because pressures are additive, the resulting pressure bulb will be much larger and will extend to greater depths. The soil in the overlap zone will be subjected to higher stresses and the foundations over this region will settle more, causing the silos to tilt towards each other.

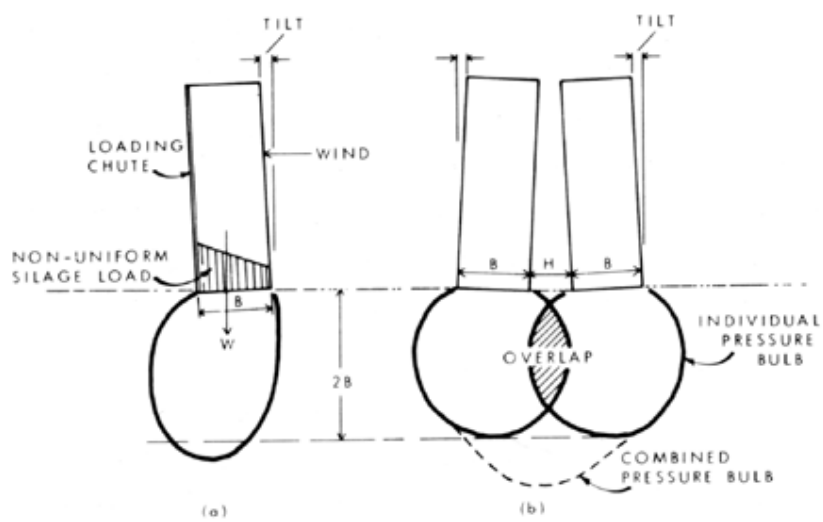


FIGURE: 2.3 (A) NON-UNIFORM AND OVERLAPPING PRESSURE BULBS.

FIGURE 2.3(B). LEANING TWIN SILOS CAUSED BY NON UNIFORM SETTLEMENT IN ZONE OF OVERLAPPING PRESSURE BULBS.

The agricultural industry should be aware of the many aspects of design and construction of tower silos on clay soils. A contractor's experience may be sufficient for building average size structures, but for very large tower silos there should be a thorough soil investigation carried out under professional guidance. It is not possible to build an economical and safe foundation unless the properties of the foundation soils are known and considered in the design. Furthermore, quality control and on-site inspection is required during construction.

Performance requirements for tower silos also have to be established. For example, how much vertical settlement or tilting can be allowed without the performance of the structure being affected? Is there an optimum limit for the ratio of height to diameter of the silo? Answers to these and other questions can be obtained by studying the performance of existing silos and from the instrumentation of new structures on different soils.

3. CODES AND STANDARDS

3.1 INTRODUCTION

Structural design requirements for conventional silos are covered in codes, while design requirements of compartment silo are not covered in any code so far. Several countries have already adopted codes and standards for conventional silo and bunker design and construction. Few of them are as below:

IS 4995 – 1974: Criteria for design of reinforced concrete bins for the storage of granular & powdery materials.
(Part: 1) General requirement and assessment of bin loads
(Part: 2) Design criteria

DIN 1055 (part: 6) - 1987: "Design Loads for Building; Loads in Silo Bins" Silo Code in Germany.

ACI-313-91: "Standard Practice for Design and construction of concrete Silos and stacking tubes for storing granular material" Silo code in the United States

ACI-313R-91 "Commentary on standard practice for design and construction of concrete silos and stacking tubes for storing granular material" silo code in the United states.

3.2 COMPARISON OF IS 4995, ACI 313-91 AND DIN 1055(PART: 6)

The critical comparison of the three codes is as tabulated below.

4. DESIGN CRITERIA

4.1 MATERIAL PRESSURE

4.1.1 PRESSURE DUE TO GRANULAR AND POWDERY MATERIAL IN SILO

Early silo designers not recognizing the importance of vertical friction between stored material and the silo wall, assumed lateral pressure to vary hydrostatically. This assumption often gives wasteful conservative results.
Subsequently analytical methods were developed that consider wall friction. These methods provide means for computing

1) Pressure of the stored material against vertical wall, and flat bottom
2) Friction forces and wall compression forces
3) Vertical pressure at various depths in the stored material itself.

Some of these methods give static pressures only. During filling or emptying of the silo the pressure will be higher than static. These methods are based on equilibrium of the stored material in a static condition. Elastic interaction with the bin structure is not considered, nor is strain energy in either the stored material of the structure. These analytical methods correlate with test measurements with varying degree of agreement.

4.1.1.1 Janssen method for computing static pressure [1]

This method is based on equilibrium of a thin horizontal layer of stored material as shown in figure 4.1
Equating the vertical forces to zero gives

$$qA + \gamma A dy = A\,[q + dy(dq/dy)] + \mu\,'p(U dy)$$

In which, q = static vertical pressure at depth Y
A = Area of horizontal c/s
U = Perimeter of horizontal c/s
p = Pressure of stored material at depth Y

Substituting kq for p and hydraulic radius R for A/U and rearranging the equation

$$dq/dy = \gamma - (\mu' k/R)q$$

The solution to this differential equation is the Janssen formula for vertical pressure at depth Y,

$$q = \gamma R/\mu' k\,[1-e^{-(\mu' ky/R)}]$$

By introducing the ranking coefficient for active earth pressure- the ratio of horizontal pressure to vertical and finally the equation for horizontal pressure is

$$p = \gamma R/\mu'[1-e^{-(\mu' ky/R)}] \qquad \text{..... (4.1)}$$

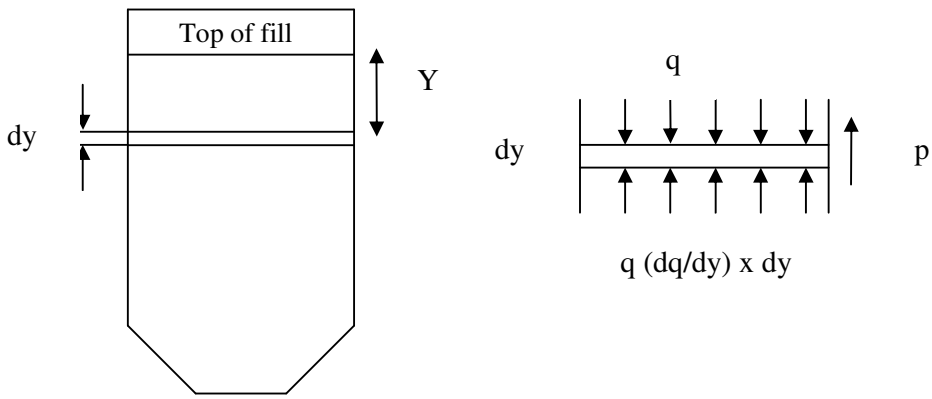


FIGURE: 4.1 HORIZONTAL LAMINA FOR DERIVATION OF JANSSEN'S FORMULA

4.1.1.2 The Reimbert method for computing static pressure[1]

In 1953 and 1954 Marcel and Andre Reimbert presented their method for computing static pressure due to stored material. Their derivation shows that at larger depth Y the curve of lateral pressure becomes asymptotic to the vertical axis. At that depth the lateral pressure reaches maximum, which is shown as p_{max}. It has equal vertical pressure above and below

$$\gamma A dy = \mu' p_{max} U dy$$

Thus,

$$p_{max} = \gamma R/\mu'$$

In which R= hydraulic radius = A/U.

Vertical pressure at that location is:

$$q_{max} = p_{max} / k = \gamma R/(\mu' k)$$

The Reimbert equation for static pressure area as follow:

Vertical pressure at depth Y below stored material surface:

$$q = \gamma\left[Y\left(\frac{Y}{C}+1\right)^{-1}+\frac{h}{3}\right] \qquad \text{..... (4.2)}$$

Lateral static pressure at depth Y:

$$P = p_{max}\left[1-\left(\frac{Y}{C}+1\right)^{-2}\right] \qquad \text{..... (4.3)}$$

The terms in above equation are easily determined.

For circular silos the terms p_{max} and C (characteristic abscissa) in above equations are:

$$p_{max} = \gamma D/(4\mu')$$

$$C = \frac{D}{4\mu' k}-\frac{h}{3}$$

4.1.1.3 The Airy's method for computing static pressure [1]

The method of Wilfred Airy presents separate solution for bunkers and silos. Airy's equation was derived considering static equilibrium of wedge shaped portion of stored material above the plane of rupture.

Figure 4.2 shows dimension used in Airy's equation for bunkers.

Lateral pressure at depth Y:

$$P = \frac{1}{2}\gamma Y^2\left[\frac{1}{\sqrt{\mu(\mu+\mu')}+\sqrt{1+\mu^2}}\right]^2 \qquad \text{..... (4.4)}$$

In which, $\mu = \tan \rho$

Vertical pressure at depth Y: $q = p/k$

Figure 4.2 shows similar dimension for silos. Airy's derivation for a silo leads to:

Lateral pressure at depth Y:

$$P = \frac{\gamma D}{\mu+\mu'}\left[1-\sqrt{\frac{1+\mu^2}{\frac{2Y}{D}(\mu+\mu')+1-\mu\mu'}}\right] \qquad \text{..... (4.5)}$$

Vertical pressure at depth Y:

$$q = p/k$$

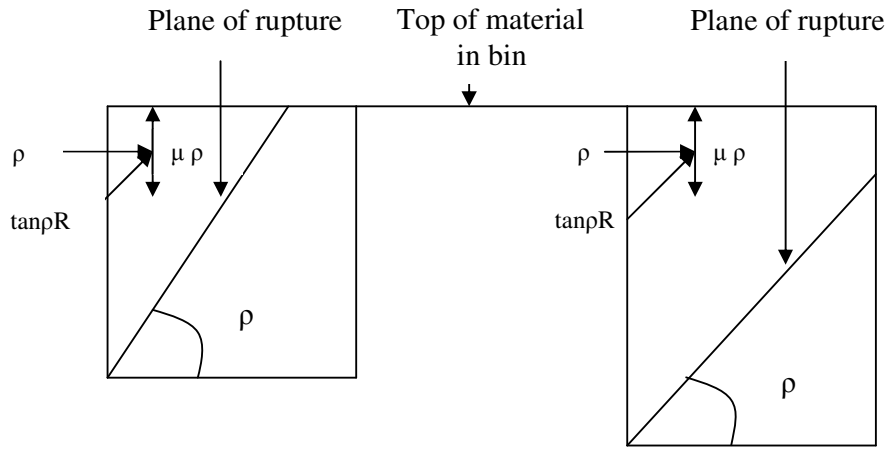


FIGURE: 4.2 DIMENSION FOR USING AIRY EQUATION FOR BUNKERS AND SILOS

4.1.2 COMPARISON OF METHODS FOR COMPUTING STATIC PRESSURE

Static pressures using Janssen's, Airy and Reimbert methods for different combination of silo proportion, stored material and friction coefficient between stored material are computed for the silo wall. The limited number of cases considered and the following conclusion may be drawn:

LATERAL PRESSURE: For bunkers and the upper portion of silos, The Reimbert method gives the highest pressures and the Janssen's method the lowest. At depth below about 3.2D, however Reimbert's method indicates a lower pressure than Janssen's method.

VERTICAL PRESSURE: For condition considered as above, the Reimbert's method gives the lowest pressure- regardless of silo size, material properties, or position in silo. Likewise the Airy's equation always gives higher pressure.

4.2 DIFFERENT FLOW PATTERN

4.2.1 Mass Flow

This type of flow pattern which occurs when the entire silo contents are set in motion whenever any solid withdrawn from the outlet. This pattern results in a FIFO (first-in, first-out) flow of solid in the silo. The flow rate is uniform and almost independent of the height of the solid in the silo. The entire volume of solid is live with no stagnant or dead zones that do not flow. Mass flow silos must have hopper bottom sufficiently steep and smooth so as to ensure that the entire silo contents flow whenever the outlet is activated.

4.2.2 Funnel Flow

It is a flow pattern, which occurs when the solid is confined to a channel that forms within the basic (stagnant) material. This pattern results in a LIFO (Last-in, first –out) flow of the solids in the silo. Funnel flow silos have significantly large dead storage since solids in the dead zones may pack. The silo has to be emptied periodically. In this type of flow pattern there is no sliding on the wall and solid slides on solid. Figures 4.3 shows basic flow pattern for mass flow and funnel flow.

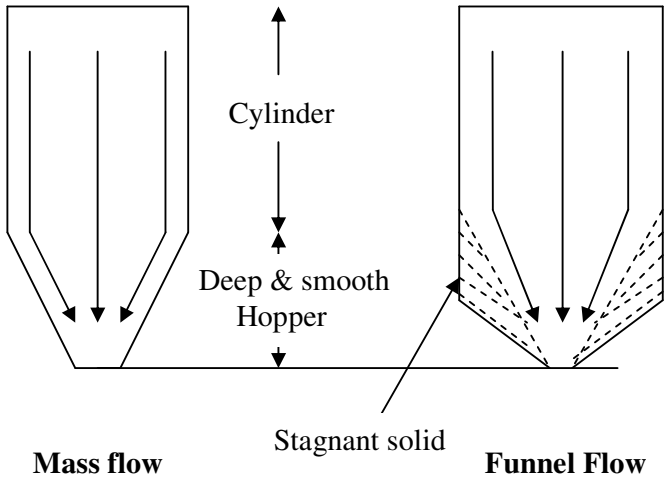


FIGURE: 4.3 BASIC FLOW PATTERN

4.2.3 Expanded Flow

Expanded flow pattern features a combination of mass flow and funnel flow. Flow usually starts within channel above the outlets, similar to funnel flow while the rest of the solid is static. As the silo empties, the static solid sloughs into the flow channel and the silo eventually empties. An expanded flow silo has a hopper consisting of two sections. The lower section is a steep mass flow hopper selected to prevent arching and rat- holing (piping) of the solid. The upper section is self-cleaning hopper, which permits the solid to slide into the mass flow section as the silo empties.

4.2.4 Arching

Arching occurs when solids form a stable dome or arch across the outlet. Arching will obstruct the flow of solid out of the silo. Arching occurs when either the outlet is not sufficiently large to assure unobstructed gravity flow or the silo is discharged at the maximum flow rate of the system. These conditions require additional analysis and I.S. Code is not explicit on increase on wall pressure due to arching. All codes agreed that vertical pressure on silo bottom need not be considered more than 100 percent of weight of silo contents. This can be avoided by suitably sizing the outlet.

FIGURE: 4.4 ARCHING

4.3 FLOW IRREGULARITES

4.3.1 Pulsation

Pulsating occurs when the slope of the hopper wall plots in the uncertain boundary region between mass flow and funnel flow. Pulsation results from repetitive formation and collapse of an obstruction to flow. The frequency of pulsation, which is usually between 1/5 and 10 Hz, is directly proportional to the

rate of outflow while the amplitude tends to be higher at low flow rates than at high. A pulsation head of stored material above the location of the pulsation source is required; where the head becomes low enough, pulsation ceases. Out flow from a bin during pulsation is uniform. This means that material flow below the pulsation source is continuous.

Objectionable pulsation occurs only with materials containing at least a fraction of coarse particles say greater than 0.2 in. Since the designer cannot select the material and transition required in most silos, the designer can only attempt to minimize the amplitude of pulsation.

4.3.2 Shocks

Periodic shocks are expected in funnel flow bins containing coarse solids with little fines. Such material would include cement, clinker, coal and corn. In large silos these shocks can be destructive. As a constant outflow rate shock severity increases with the length of the presiding interval. Uniformity of feed rate is not affected by the shocks, but the top level of material in the bin usually remains stationary during the interval, dropping an abruptly during the shock. The shocks diminish in strength and then vanish. This shock densifies the material in the flow channel, which reforms with higher wall pressures capable of stabilizing the material around it. As material discharges the wall pressures decrease and a new collapse occurs.

Shocks can be eliminated or at least minimized by expanding the flow channel to a base diameter. Shocks of this type do not occur in cohesive materials that develop stable stagnant channels. Such materials resist shocks through counteracting void air pressure gradients. Severe shocks can originate within a flowing material when the hopper slope friction are on the boundary between mass and funnel flow. To prevent these shocks hopper slopes should be steep enough so that plot continuously in the mass flow region. A stainless steel linear or sometimes an epoxy coating, or special plastic lining may provide a solution in an existing hopper that is not quite steep enough for continuous mass flow.

4.4 EFFECT OF OPENING- ECCENTRIC FLOW

If the bulk solid in a silo does not flow downwards uniformly across the cross-section but in an eccentric flow zone, then this behavior is called eccentric flow. Typical reasons for eccentric flow are:

- Funnel flow silo with an asymmetrically formed flow zone, especially in the case of an eccentric outlet opening.
- Silo with a feeder, which withdraws the bulk solid only from a part of the outlet opening.
- Asymmetric hopper.
- Silo with more than one outlet opening of which not all are in use.

The non-uniform stress distribution can be explained as follows. A vertical section of a silo (diameter d) with an assumed flow zone (diameter df). The bulk solid in the flow zone, which flows downwards, affects not only the silo wall but also the bulk solid at rest (dead zone) with downwards – directed shear stresses i.e. the bulk solid in the flow zone transmits a part of its weight to the silo walls and to the bulk solid at rest (the shear stresses are drawn in their direction of action). Therefore, the stress in the flow zone decreases whereas the stress in the dead zone increases. In reality the situation is more complicated.

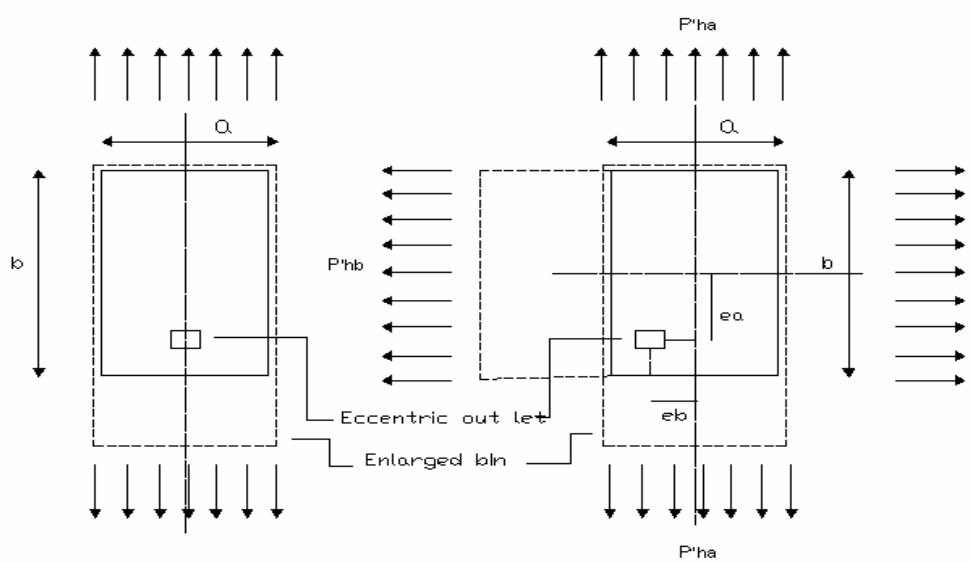


FIGURE: 4.5 (A) RECTANGULAR BINS

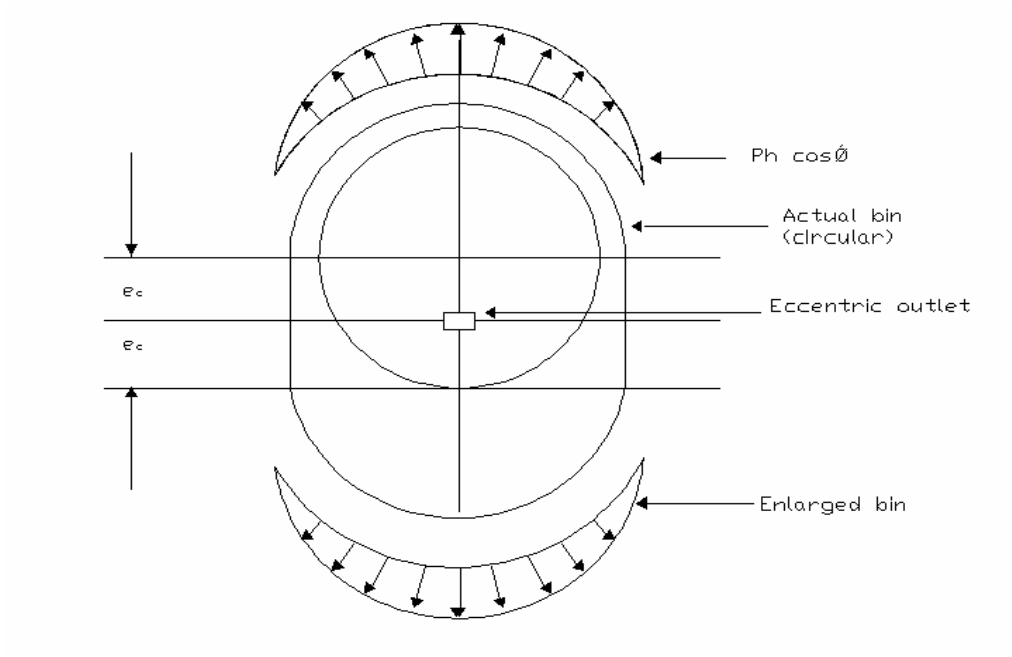


FIGURE: 4.5 (B) CIRCULAR BIN

Fig.4.4 (a) and (b) shows effect of outlet for rectangular bin and circular bin. Experimental study shows that withdrawal of granular and powdery material through eccentric opening causes lateral pressure changes much different from that of concentric opening. Early test shows that the pressure increase due to eccentric discharge compared with that of concentric discharge occurs on the opposite side to the discharge opening eccentricity and that decrease on the side nearer to the opening.

The cause of non-uniform pressure is the nonsymmetrical flow of material and cause horizontal and vertical bending moments, which must be considered in design of silo wall.

The following discussion introduces several approximate methods for computing additional lateral pressure P_{ecc} due to eccentric discharge. Figure: 4.5

ACI 313-77 APPROACH

Assume the increase of design lateral pressure to be at least 25% of the static pressure at the bottom of the silo when an opening is next to the silo wall. If the eccentricity (e) of the opening from the center of the silo is less than radius (r) consider the increase to be at least 25% times e/r. Assume this design pressure

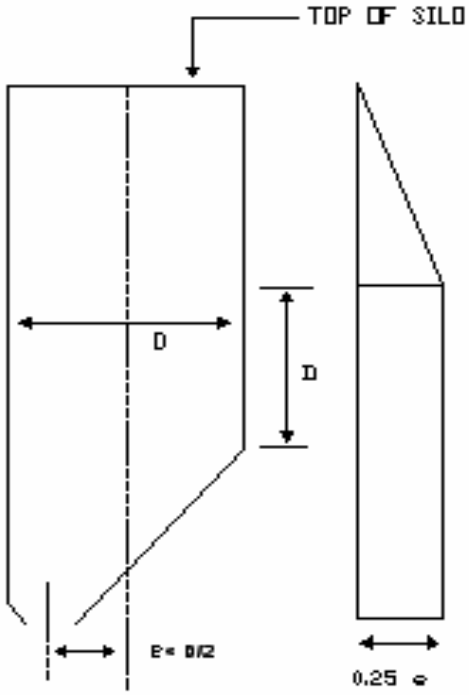


FIGURE: 4.6 PRESSURE INCREASE-ECCENTRIC DISCHARGES

Increase to constant from the top of the hopper at a height equal to D and to reduce linearly from that point to top of silo. The increase need not be multiply by C_d. Expressed as an equation the design pressure at any depth Y below the surface of the stored material would be P_{des}.

$$P_{des} = C_d \times P_{st} + P_{ecc}$$

4.5 LOAD CONSIDERATION

The principle loads for silo and bunker design come from action of the stored material loads or forces from other sources including; dead load, equipment load wind, floor and roof live loads' seismic loads; forces from thermal effects and forces applied at restrain of attached items.

4.5.1 DEAD LOAD

It includes weight of silo-its self weight,walls, roof, ring beam, hopper – plus the weight of item supported by the silo. These loads include inside and outside stairway and service platform, equipment on silo roof overhead gallery etc.
Many of these dead loads can only be estimated in the early phase of design. Dead load of silo bottom that rest on independent support should be separated from those affecting the silo walls.

4.5.2 LIVE LOAD

Pressures due to stored material are treated as a live load. Live load on platforms, roof, and floors should be as required by applicable building code or larger. To determine the wall load the designer must know the flow condition. Different methods of determining design pressure must be used according to whether the flow is funnel flow, mass flow or some other type.
Aeration may be used to cause a semi fluid condition in the stored material. Pressure intensity in that cased is express as

$$P = 0.6 \times \gamma \times Y$$

In which, γ = weight per unit volume, and
Y = depth of aerated material

4.5.3 WIND LOAD

All silo structure should be designed to resist the overturning effects caused by wind or earthquake forces. Wind and earthquake load should not be assumed to act simultaneously. Which ever is more will be taken in design. Practically all codes permit an increase of allowable stresses and soil bearing values when either wind or earthquake acts alone or in combination with the dead load and the live load. Most building codes allow a one third increase above the usual values permitted.
Wind load for silo can be in any lateral direction and generally should be considered as positive pressure on the wind ward side and negative on lee ward

side. Foundation pressure and column stresses may be the worse cases with wind acting on the full silo the following reduction factor may be applied depending on cross sectional shape.

Hexagonal or octagonal single silo 0.8

Round single silos 0.6

No reduction factor should be applied to silo groups.

4.5.4 EQUIPMENT LOAD

Equipment item may apply severe live load on the structure. Theoretically equipment manufacture should be able to predict the load their equipment will impose but if this equipment is vibrating it may bring out changes in other loads. For example the stored material may become compacted acquiring higher density and altered flow characteristic with resultant changes in lateral and vertical pressure.

Belt conveyor and their support system can bring large live and dead load to a silo.

It is recommended that pinned or fixed connection of conveyor support to a silo roof not be allowed.

4.5.5 LOADS DUE TO THERMAL EFFECT

Two types if thermal effect may need to consider. The first is a through the wall temperature gradient important in concrete wall cause by storing material. That is much higher than the air temperature around the silo.

Daily temperature changes due to intense sunlight may cause expansion and contraction of silo groups. Stresses due to this action are large enough so cause wall to crack. Seasonal temperature change can have similar effect. Steel silo can fail by daily thermal expansion and contraction. Thermal stresses have to be considered during design process.

The temperature of hot material in silo is not uniform, but drops appreciably near the inside wall. This drop may be considered when determining the design temperature difference ΔT, between inner and outer wall faces.

$$\Delta T = T_2 - T_1$$

T_2 & T_1 may be calculated by heat transfer equation $Q = U \times A \times (\Delta T)$

Where, Q = heat transfer (kg/hr)

U = heat transfer coefficient

A = Area of the cross section of wall

The temperature difference ΔT within the wall is a portion of the total design temperature difference corresponding to K_t, the ratio of the thermal resistance of the wall alone to that of the cement, wall, and outside air combined.

$$\Delta T = K_t\ [Ti_{des} - T_0]$$

The determination of Kt is by the heat transfer principle factor Kt for various thicknesses is taken by curve as shown in figure: 4.6

The ultimate thermal bending moment is calculated as per code guidelines and required area of steel reinforcement is provided in case of R.C.C. Silo. The area of steel should be located near the outer (colder) face. Vertical tensile thermal stress is usually offset by vertical dead load compressive stresses so that added vertical temperature steel for the effect of hot material is normally not needed.

IS 4995 has given formula as below

$$\Delta T = t/C_c \times (Ti - To)/\ (1/k);$$

$$1/k = 1/C_a + t/C_c + 1/C_s$$

Where, t = thickness of bin wall in m

C_a = surface conductance of concrete to air = 15 in kcal/m^2hºC

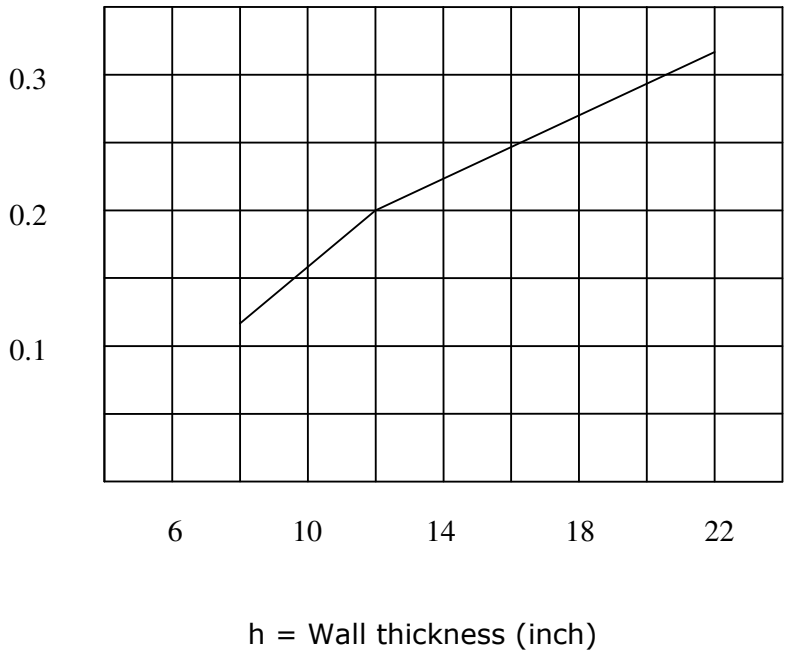
C_c = thermal conductivity of reinforced concrete

= 1.75 to 2 kcal/mº h

C_s =surface conductance of concrete to stored material

= 2 kcal/m2hºC

Kt = {0.08h / (94.09-0.08h)}



FLOWCHERT: 4.1 TEMPERATURE DROP THROUGH WALL – ACI METHOD

Stresses and bending moment as per IS 4995

Bending moment due to change in temperature is calculated as per following expression

$$M\Delta T = E_T \times \Delta T \times E_c \times 1/t$$

Where, E_T = Coefficient of thermal expansion of concrete = 11×10^{-6} / per degree per cm

E_c = modulus of elasticity of concrete = $18000\sqrt{\sigma cu}$; Kg / cm^2

I = moment of inertia of the bin wall.

4.5.6 SEISMIC LOAD

Earthquake load may affect both stability and strength of silo and bunker. Walls and columns supporting silos and bunkers may be particularly vulnerable to earthquake forces.

In the absence of better code requirement on seismic design of silo to resist lateral load different approach is given by ACI313-77 on silo and bunker. The suggested approach is based on the following assumptions.

1. Seismic forces can act in any horizontal direction. Vertical seismic force is neglected for most aspect of seismic design.
2. Only a fraction of the stored material weight need be considered when computing seismic force. For simplicity the seismic forces on the fraction is treated as lateral static force applied at the centroid of the entire stored mass and acting in any horizontal direction. Generally 80% of the weight of the stored material as an effective weight, W_{eff} on which to base computation of lateral force.
3. The weight of the stored material in a suspended hopper should not be reduced to an effective weight value.
4. When silo bottom are on support that are independent of the silo walls these two independent structures will share the lateral force from seismic action on the effective weight, $W_{eff.}$

4.5.7 BLAST LOAD

If quarry is very near the plant site, load due to quarry blasting is to be considered.

4.5.8 LOADS FROM EXTERNAL RESTRAINTS

A Silo is a flexible membrane. The walls of an isolated circular silo under uniform internal pressure around its circumference expand radially. Such a membrane has high horizontal membrane tensile stresses, but no horizontal bending moment.

However if at any point the silo wall is attached to something that restrain its radial movement, the wall is dented and significant horizontal and vertical bending moments occur. These bending moment when their effect is added to the hoop tension and vertical compression could cause wall failure. Often the silo designer becomes aware of the connecting item only after the main silo design has been completed.

4.5.9 LOAD COMBINATION

The designer should consider all reasonable load combinations. Combination that is extremely improbable is usually neglected. Table no: 4.1 shows a matrix of load types and suggested combinations for silo and bunkers. Combination not shown would ordinarily be ignored.

TABLE: 4.1 LOAD COMBINATIONS FOR SILOS AND BUNKER DESIGN

	Combination				
Type of load	A	B	C	D	E
Dead load	X	X	XX	XX	XX
Floor and roof live load	X	±		X	
Material pressure					
Static	X	X		X	
Overpressure	X	X		X	
Thermal loads	X	X		X	
Equipment loads	X	X		X	
Wind loads		X	X		
Earthquake loads				X	X
Restraint loads	±			±	

X: consider

XX: consider but reduce if required by code

±: consider or neglect which ever is more severe

5. CONCRETE SILOS

5.1 PERMISEBLE STRESS

5.1.1 STRESS IN CONCRETE

The permissible stress in tension (direct and due to bending) and shear shell conform to the values specified in table of IS 4995(part II) 1974.The permissible tensile stress due to bending applies to the outside face of the bin. In members less than 225mm thick and in contact with the material on one side the permissible stress in bending are also applicable to side in contact with stored materials.

5.1.1.1 STRESS IN CONCRETE FOR RESISTANCE TO BUILDING

For resisting buckling, The maximum compressive stress on net wall section deducting all wall opening, recesses, etc shall not exceed $0.25\sigma_{cu}$ where σ_{cu} is the compressive strength of concrete at the age of 28 days.

5.1.2 STRESS IN STEEL REINFORCEMENT

The permissible stress in steel reinforcement shall confirm to the value in table: 2 (cl.4.3 IS: 4995, part II).

5.1.3 INCREASE IN PERMISSIBLE STRESS

Where stress due to wind and earthquake are combined with those due to dead load, live load and impact loads and temperature and shrinkage effect the stress may be exceeded up to a limit of 33.33 percent.

5.2 WALL REINFORCEMENT [1]

Wall reinforcement must serve three purposes:

1. It must strengthen the wall to resist computed forces and bending moments resulting from the stored materials and other loading.
2. It must distribute tensile cracking so as to keep crack width within Permissible limits.

3. It must resist vertical and horizontal bending moments so as to distribute the effect of the local concentration of pressure to a larger area of silo wall.

Vertical bending moment may cause horizontal cracking. Such bending moment can result from non uniform pressure on walls, from temperature difference between stored materials and outer wall surface or from external concentrated loads. Vertical steel to resist these bending moments also provides supports for the horizontal steel.

5.2.1 VERTICAL REINFORCEMENT [1]

Vertical reinforcement in silo walls has an important function of aiding to distribute lateral loads irregularities vertically to successive layers of horizontal reinforcement. It serves to resist vertical tension due to following causes:

1. Temperature or moisture changes in the wall when the wall is restrain or not free to move in vertical direction.
2. Vertical bending moment at levels where the wall is restrained against circular elongation.
3. Vertical bending moment due to eccentric loads, such as those from hopper Edges or attached auxiliary structures.
4. Vertical bending moment due to concentric loads at the transition of a "material Hopper" during unloading.
5. Temperature differentials between inside and outside wall surfaces or between silos.

5.2.2 HORIZONTAL REINFORCEMENT [1]

Horizontal reinforcement in circular silos or bunkers is usually in a single layer for walls about 8 inches thick. Such reinforcement and wall thickness are found in small to moderate size structure up to about 30 feet in diameter. Large structure more in thickness should have two layers of reinforcement .Location of reinforcement within wall thickness is important. A structural advantage of having horizontal reinforcement near the inside and outside face of the walls increases the effective depth d; the steel will more effectively resist horizontal bending. Two-layer reinforcement is recommended especially for silos of moderate or large diameter.

5.2.3 REINFORCEMENT AROUND WALL OPENING [1]

Major wall opening in silos are usually confined to walls below the bottom structures. They range from small openings from ducts or pipes, to opening for belt conveyors to those for passage of large trucks or railway cars.

From structural stand point it is a good practice:

1. To keep the total area of openings as small as possible.
2. To avoid having several opening along the same vertical line or at the same elevation (especially if they remove an appreciable height or length of wall).
3. To keep large openings well below the silo or bunker bottom structure.

To avoid trouble at the top of wide openings in the lower wall Safarian and Harris [1] suggest that the top of these openings should be far below the bottom component of the silo. That distance should preferably be not less than the largest of the following.

1. Two third of the opening width.
2. Three times the wall thickness.

Extra reinforcement around the opening should be provided and the section should have additional reinforcement. This added reinforcement serves four purposes:

1. To help the wall above a wide opening to span as a beam acting as lintel.
2. To transfer horizontal tensile force from one side of the opening to the other.
3. To strengthen the wall at each side of the opening so that it can carry vertical force.
4. To control cracking and to take care of stress concentration at corners.

The recommendations given by ACI 313 are as follows:

1. At openings in the pressure zone (container part): replace all horizontal reinforcement with 1.2 times that is intercepted by the opening. Half of this horizontal reinforcement is placed above the opening and the other half below the opening.
2. At openings not in the pressure zone (supporting system): replace all horizontal reinforcement that is intercepted by the opening. Half of this horizontal reinforcement is placed above the opening and the other half below the opening.

5.2.4 SPLICES OF REINFORCEMENT [1]

Reinforcement in silos and bunkers normally is lap-spliced. Horizontal reinforcements are spliced for tension. Vertical bars may have only compression splices, in case of lateral loads or vertical bending moment the bars are in tension and in this case tension splices is needed.

A tension lap-splice may be a starting point for failure particularly if the concrete cover over the bars and the spacing between bars are both small. If all hook bars in a circular silo where lap spliced at the same location of the wall circumference, a plane of weakness would be started and extended for the full height of the silo.

To overcome to this problem ACI 313 require that hoop reinforcing splices in circular wall be staggered horizontally by at least one lap length (l) or three feet. That laps not coincide its position more frequently than every third bar as shown in figure.

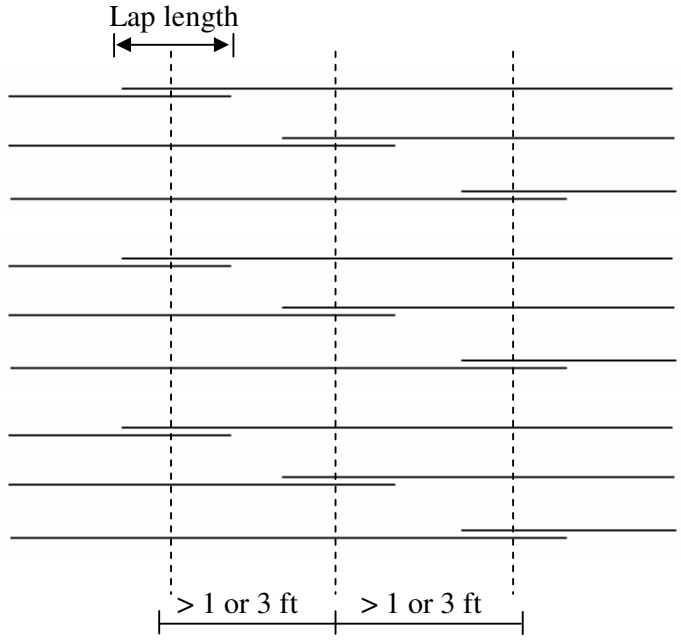


FIGURE: 5.1 HOOP SPLICES

5.2.5 DETAILING PRACTICE FOR WALL REINFORCEMENT [2]

While finalizing detailing and designing, the designer should keep in view the construction aspect of silo. The following table is based on the recommendations given in IS code. Where IS code is not specific, recommendations of ACI are given.

TABLE: 5.1 DETAILING PRACTICE – WALL REINFORCEMENT (DEFORMED BAR)

	Circumferential (hoop) steel	Vertical steel, Ext. wall	Vertical steel Int. wall
Min. steel (Percentage of Gross area of wall)	0.25	0.20	0.15
Spacing (mm)			
Minimum			
Single row	200	225	300
Double row	400*	450*	600*
Maximum			
Single row	75	75	75
Double row	100	100	100
Location	Outer face of wall	Tied inside to hoop steel	Tied inside to hoop steel

* Stagger on each face of wall

5.3 BOTTOMS FOR CONCRETE SILO

Bottoms of the concrete silos are of many types such as:

1. A horizontal slab with one or more discharge opening.
2. A concrete hopper of conical, pyramidal or other shapes
3. An inverted reinforced concrete cone with multiple discharge opening
4. A steel hopper

Silo bottoms may be supported by the silo walls, by separate bottom-supporting walls that extended down to the foundation, by pilasters connected to the lower silo walls, by independent columns or by a combination of these. In some cases, the foundation mat itself serves as the silo bottom. Selecting a silo bottom is a question of economy and the method of material withdrawal to be used. Self-cleaning silo bottoms should have a smooth inside surfaces and their inclined wall should have a slope steeper than the angle of the material to be stored.

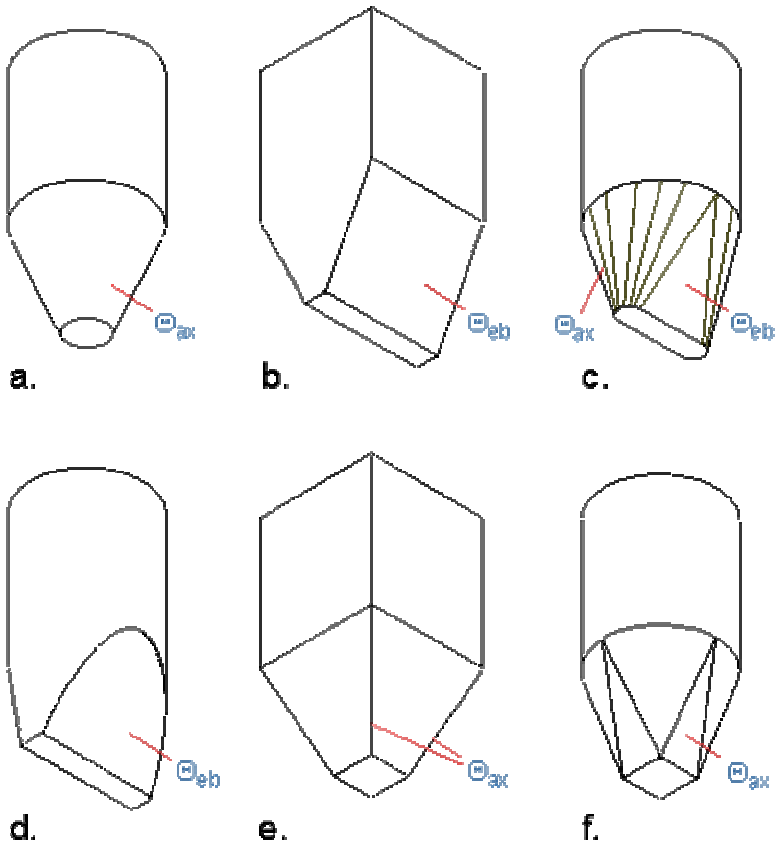


FIGURE: 5.2 DIFFERENT FORMS OF HOPPERS

5.3.1 CONICAL CONCRETE HOPPERS

The conical concrete hopper shown in figure: 5.2 (a) may be rigidly attached to the silo wall but ordinarily is supported by a concrete ring beam around the upper perimeter of the hopper. The bottom of the cone preferably sho7uld not be restrained or supported. The conical hopper shell is subjected to two tensile membrane forces. The tangential force is in the plane of shell and horizontal. The meridional force per unit width at depth Y is computed from equilibrium of the loads on the cone below that depth. The total meridional tension at any horizontal plane passing through the hopper shall be such that its vertical component is equal to total vertical pressure on that plane plus the self-weight of the hopper and its content below the plane. The meridional reinforcement shall extend sufficiently into the vertical wall to secure adequate bond. The hoop tension at any level of the conical hopper shall be determined as follows:

$$\text{Hoop tension} = r_h \, P_n \, \text{cosec}\, \alpha$$

Where, r_h = radius of hopper at the plane under consideration,

P_n = normal design pressure at the plane under consideration

Both meridional and tangential forces are maximum at the upper edge of the silo hopper and approach zero at the lower edge. The minimum acceptable thickness for the cone would be determined considering the acceptable crack width as for silo wall.

5.3.2 PYRAMIDAL CONCRETE HOPPERS

Pyramidal hoppers shown in figure: 5.2 (e) are used in rectangular silos and can be supported by edge beams or by being built with the silo walls. A structural disadvantage of the pyramidal hopper is that its walls are subjected to both biaxial tension and two – way bending. Further the hopper walls may also have in plane bending when they are flex rally continuous with the silo walls above and when the silo walls have large openings immediately below.

5.3.3 BOTTOM OF THE OTHER SHAPES

As per the IS 4995 the bottom of other shapes such as bottom with one or more sloping sides with the remaining sides vertical, bottoms provided with special emptying arrangements etc. shall be designed based on principle of mechanics and sound engineering practice. Bottom of other shape like inverted cone is also used to keep more space below the cone for handling material and can be possible to keep supporting system height as less as possible.

5.4 SPECIAL SILOS

5.4.1 DOUBLE CYLINDRICAL CONCENTRIC SILO (RING SILO)

A small inner silo within another of larger diameter as shown by figure: 5.3 is occasionally used in the sugar and cement industries. This types of silos is provided for storing two different kinds of material.

The stored material loading in ring silo is considerably different from the loading in single silo. Particularly difference is the behavior of the material stored between the inner and outer silo walls.

The simplest approach for computing pressure from the material between the silo walls is to assume the container between the outer and inner wall to be similar to a rectangular silo. This assumption is considered reasonable by the fact that the hydraulic radius of both cross sections is same. for a rectangular cross section with hydraulic radius R is equal to one half of the short length and for ring silo hydraulic radius R is equal to one half of the difference of the radius as shown in Figure: 5.3

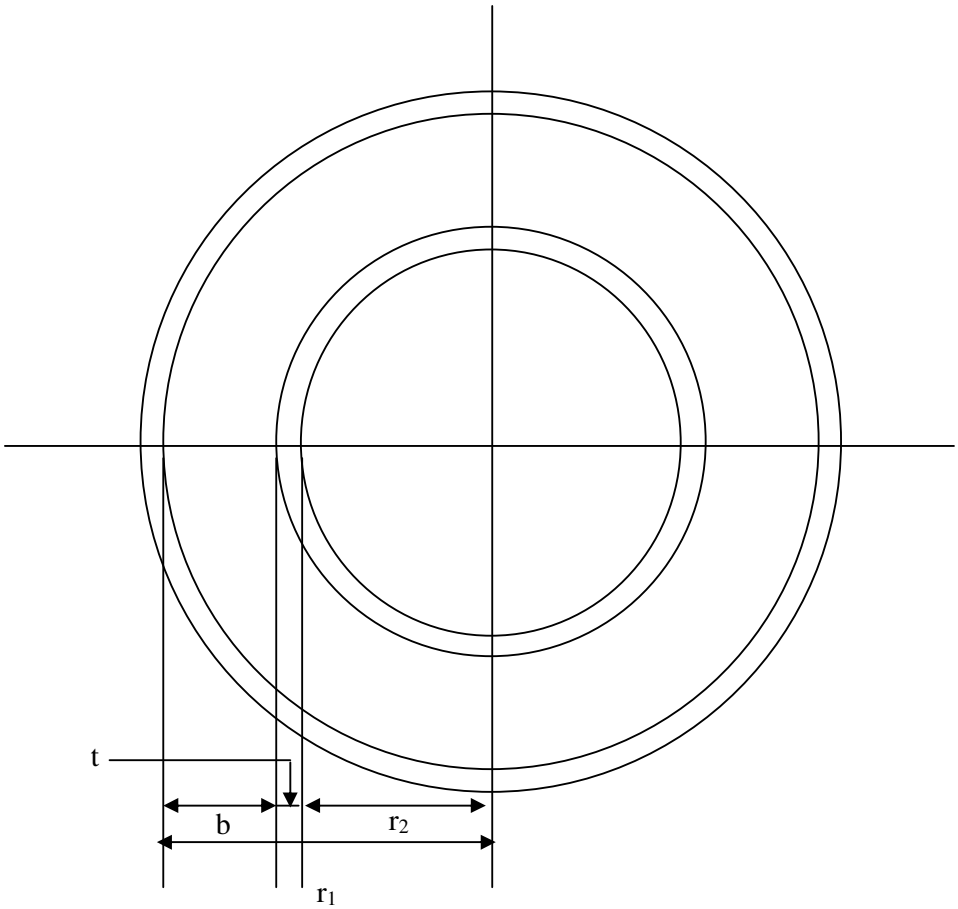


FIGURE: 5.3 RING SILO CROSS SECTION

Area of outer silo $= \pi[r_1^{\,2} - \{r_2 + t\}^2]$

Perimeter of outer silo $= 2\pi[r_1 + \{r_2 + t\}]$

R = Area/Perimeter

= b/2

5.4.2 HOMOGENIZING OR BLENDING SILO

Homogenizing silos have been developed to meet the need for large volume storage along with blending or homogenizing of a wide range of dry pulverized free flowing materials that require homogeneity in the process flow. These silos provide functions, storage and blending in a single unit. Such silos can be of either steel or concrete. They can be built as single silo or different silo. Some of which may be multistory compartment.

Once a loading is determined there is little difference between structural design procedure for blending and homogenizing silos and those for ordinary storage

silos. The walls of both concrete and steel blending silos should be airtight. In concrete silo special attention should be given to joints between wall and roof slab and between wall and bottom slab. A sealing compound should be selected to assure a permanent seal.

5.4.2.1 Pressure in Homogenizing silo

By extensive study on the behavior of powdery material in silo under aeration and homogenizing condition several conclusions have been made.

1. In the homogenizing chamber the material behave as a liquid having a hydrostatic pressure distribution equal to that of a liquid whose unit weight is 0.6 times the unit weight of the unaerated actual material.

2. High speed filling of powdery material especially in narrow silo may produce higher lateral pressure more than pressure due to normal gravity filling. Marten found that the lateral pressure due to such high speed filling is a function of the velocity of the material. Vf and a factor Cr that depends on the nature of the powdery material. Marten equation for the lateral pressure occurring during such filling is as under.

$$P \text{ filling} = YC_f V_F \ (kN/m^2)$$

In which, V_F in m/hr

Y = Density of material

5.4.3 PRESTRESSED CONCRETE SILO

Large circular silos over 18 to 20m diameter generally required prestressing require prestressing to counteract the hoop tension. Prestressing is essentially required for the wall of rectangular silos, which are subjected to tension and bending. Also in silos for the storage of cement and clinker, large inside temperatures are encountered which result in considerable expansion of the cell walls. The restraining of which would lead to considerable bending moments. These cannot be balanced without cracking unless prestressing is applied. Vertical bending moments also develop due to arching of the stored solid and hence the walls are to be prestressed both horizontally and vertically.

5.4.4. FIBRE-REINFORCED PLASTIC SILO

Recently FRP silos have found favourable in the plastic industry for the storage of polymeric products. The smooth non-corrosive finish of the internal wall ensures non-contamination of the product and the low wall friction minimizes potential hold up during discharge. FRP is tough, impact resistant and is generally not affected by ageing or weathering. It is also simple to anchor brackets, nozzles, man ways and other attachments to the FRP silo during manufacture.

5.5 CONSTRUCTION METHODOLOGY FOR SILO

The design boundary conditions will depend upon the method of construction. The following two methods or constructing silo with central cone are possible:

1. Entire wall is slipformed from top of foundation. At the junction of change of section of wall formwork is altered. Cone bottom is constructed later.
2. The above method is possible only in case of single cell circular silo. In case of ring or compartment silo, the inner wall is supported on central cone. In such case lower portion of silo wall and cone with ring beam are constructed first up to level of starting of inner wall. Silo walls (outer and inner together) are slipformed from this level.

The cone is constructed with inside formwork on scaffolding. Insert plates can be left in lower part of silo wall to support formwork girders, resulting in reduced height of scaffolding. The scaffolding need not be design for full weight of fresh concrete of the cone, as it is concreted in ring shaped sections. Sections already cast, carry load of next sections. In view of this advantage of conical shell, precasting is also feasible even for large size of cone.

5.5.1 SILO CONSTRUCTION TOLERANCES

Tolerance limits as recommended in ACI code are given below:

1. the maximum horizontal deviation (which may consist of translational and rotational components) of any point on the structure relative to a corresponding point at the base of structure shall be not over 8 cm at any location for 30 m height silo, nor over 10 cm for silos over 30m height.
2. The variation from true circular cross section in a circular silo and from prescribed diameter shall be not more than 2.5cm or 4 mm/m of diameter, whichever is larger, but is no case more than 7.5cm.
3. The variation from prescribed inside width dimensions for noncircular silos shall not exceed 4 mm/m of width nor 5 cm total.
4. The variation from prescribed wall thickness shall be in the range from 1 cm to 2.5 cm.

6. DESIGN EXAMPLE AND RESULTS

Design a six compartment silo to store cement of total capacity 15000tons.

General data:

Internal diameter of silo	= 18m
Height of silo	= 60.8m
Storage capacity of whole silo	= 15000t
Storage capacity of one compartment	= 2500t
Inverted central cone at bottom with horizontal	= 60°
Top of cone from G.L	= 18.5m
Grade of steel	= Fe 415
W = 15.5 kN/m^3 for stress calculations	
= 13 kN/m^3 for volume calculations	
Outside temperature	= 100°
Inside temperature	= 5°
Soil strata is weathered rock with safe bearing capacity at 3.5m depth	= 550kN/m^2

6.1 DESIGN OF SILO ROOF SLAB

In such silos, there are numbers of irregularly spaced openings at roof top. Normal practice is to provide steel beams framing these openings. In this theoretical example, these opening data are not available by suppliers; only a design method of covering top slab is indicated.

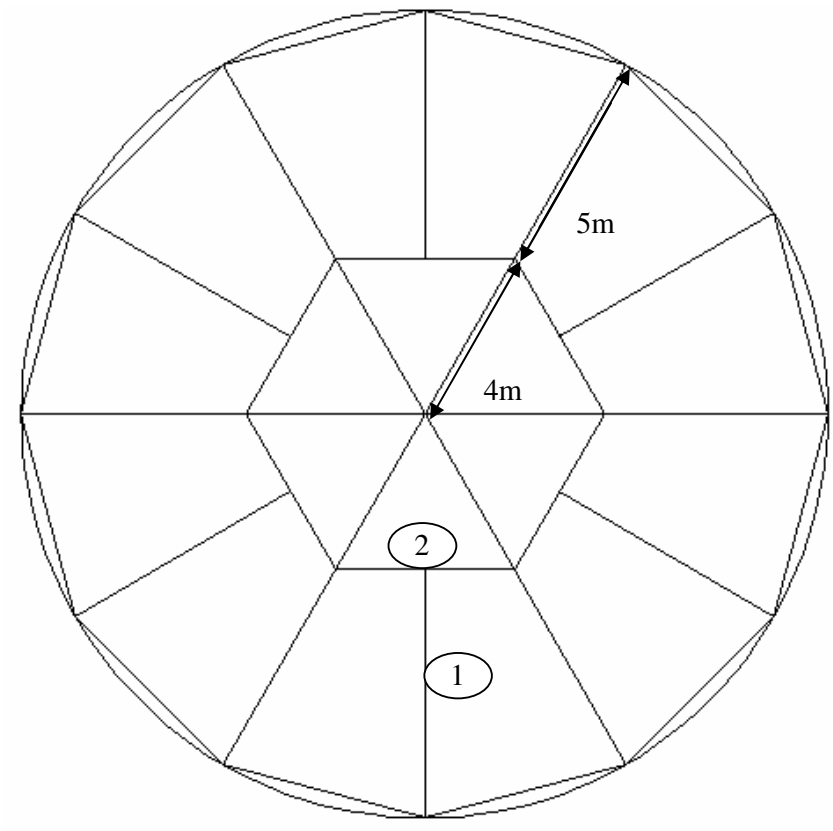


FIGURE: 6.1 GEOMETRY OF SLAB

6.1.1 ASSUMED LOAD ON SILO ROOF

Self weight of slab = thickness of slab x density of cement

= 0.25 x 25 = 6.25 kN/m²

Dust load on slab = 6 kN/m²

Plaster load on slab = 1.25 kN/m²

Equipment load (includes loads due to plates, conveyor support, weight of conveyor as given by equipment suppliers) = 10 kN/m^2

Total load on slab = 23.5 kN/m^2

Factored load on slab = 1.5 x 23.5 = 35.25 kN/m^2

6.1.2 DESIGN OF TRIANGULAR SLAB

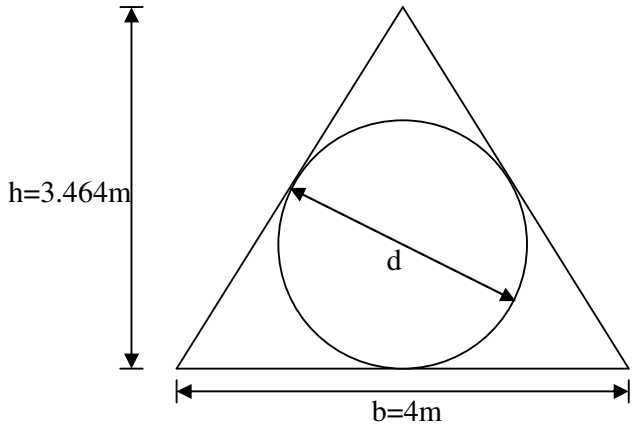


FIGURE: 6.2 TRIANGULAR SLAB

From, Reynolds's handbook, Table: 57[3]

Here, h = 2 x sin 60° = 3.464m

$$d = \frac{2 \times b \times h}{b + \sqrt{(b^2 + 4 \times h^2)}}$$

$$= \frac{2 \times 4 \times 3.464}{4 + \sqrt{4^2 + 4 \times 3.464^2}}$$

= 2.31 m

Assume slab thickness 200mm

Bending moment at the centre = $wd^2/30$[3]

= $35.25 \times 2.31^2 / 30$

= 6.27kNm

Bending moment at the support=$wh^2/30$[3]

=$35.25 \times 3.464^2/30$

= 14.1kNm

Reinforcement at the centre

$Mu/bd^2 = 6.27 \times 10^6/1000 \times 160^2$

= 0.245 N/mm^2

From SP: 16, table: 2[4]

Pt = 0.085

Ast = Pt x b x d / 100

= 0.085x1000x160/100

= 136 mm^2

Provide 12T – 200 mm c/c both ways

Reinforcement at support

$Mu/bd^2 = 14.1x10^6/1000x160^2$

$= 0.551\ N/mm^2$

Pt = 0.158[4]

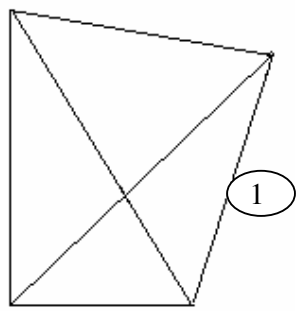
Ast = Pt x b x d / 100

= 0.158 x 1000 x 160 / 100

= 252.8mm²

Provide 12T - 200 mm c/c both ways

6.1.3 DESIGN OF BEAM 1



Load intensity = 35.25 kN/m²

As the load transferred on the beam,

Area of one triangle = 4.1078 m²

Load on beam from one side triangle = 4.1078 x 35.25

= 144.8 kN

U.D.L on the beam from one side triangle = Load / Length of beam

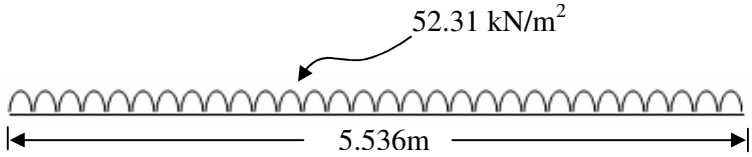
= 144.8 / 5.536

= 26.16 kN/m

There are two similar triangles on both sides of the beam.

Total load on beam from both triangle = 26.16 x 2

= 52.31 kN/m



Beam: 1

Assume beam size = 300 x 450mm

Moment @ center = $wl^2/8$

= 52.31 x 5.54^2 / 8

= 200.7kNm

Mu/bd2 = 200.7 x 10^6/300 x 410^2

= 4.0 N/mm^2 > 2.76 N/mm^2

Therefore, Provide doubly reinforced beam.

d'/d = 40/410[4]

= 0.097

Pt = 1.337

Pc = 0.401

Ast = 1.337 x 300 x 410 / 100

= 1644 mm^2

Asc = 0.401 x 300 x 410 / 100

= 493 mm^2

Provide 6 – 20T bars Ast $_{prov.}$ = 1885 mm^2

Provide 2 – 20T bars Asc $_{prov.}$ = 628 mm^2

6.1.3.1 Check for shear [5]

Total shear force (Vu) = 144.8 kN

Pt = 1.02

From IS: 456[5], Table: 19, For M20 grade concrete,

Design shear strength of concrete (Tc) = 0.62 N/mm^2

Maximum shear stress (Tc $_{max}$) [5] = 2.8 N/mm^2

Here, Tc < Tc $_{max}$ …….Ok.

Shear taken by concrete Vc = Tc x b x d

= 0.62 x 300 x 410

= 76.26 kN

Vu > Vc

Therefore, shear reinforcement is required.

Strength of shear reinforcement (Vus) = 144.8 – 76.26

= 68.54 kN

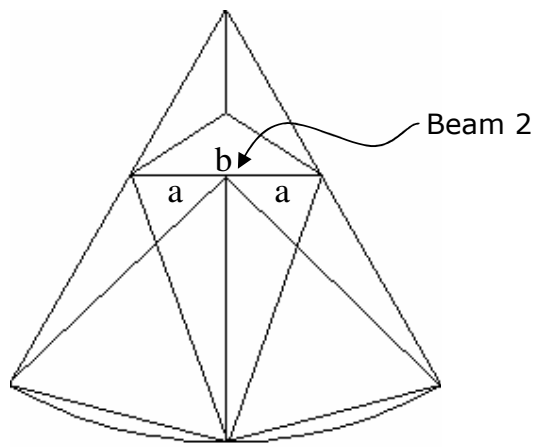
Spacing of the stirrups, $Sv = \frac{0.87\,fyAsvd}{Vus}$

$$= \frac{0.87 \times 415 \times (\pi/4) \times 8^2 \times 2 \times 410}{68.54 \times 10^3}$$

= 217.12 mm^2

Provide 8φ – 150 mm c/c stirrups.

6.1.4 DESIGN OF BEAM 2



Load intensity = 35.25 kN/m²

As the load transferred on the beam,

Area of triangle (a) on one side of the beam = 1.4281 m^2

Load on the beam from one side triangle = 1.4281 x 35.25

= 50.34 kN

U.D.L on the beam from one side triangle = Load / Length of beam

= 50.34/2

= 25.17 kN/m

Area of triangle (b) on other side of the beam = ½ x 4 x 3.464 / 3

= 6.928/3

= 2.309 m^2

Load on the beam from other side triangle = 2.309 x 35.25

= 81.41 kN

U.D.L on the beam from other side triangle = Load / Length of beam

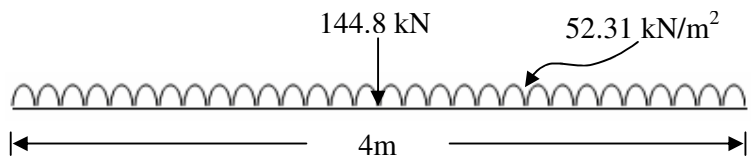
= 81.41 / 4

= 20.35 kN/m

Pt. load on the beam due to beam 1 = 144.8 kN/m

Total U.D.L on the beam = 20.35 + 25.17

= 45.52 kN/m



Beam 2

Due to U.D.L: -

Moment @ support = $wl^2/12$

= 45.52 x 4^2

= 60.69 kNm

Moment @ span = $wl^2/24$

= 45.52 x $4^2/24$

= 30.34 kNm

Due to pt. load:-

Moment @ support = wl/8

= 144.8 x 4/8

= 72.4 kN/m

Moment @ span = wl/8

= 144.8 x 4 / 8

= 72.4KN/m

Total moment @ support = 60.69 + 72.4

= 133.09 kNm

Total moment @ span = 30.34+72.4

= 102.74 kNm

$Mu/bd^2_{support}$ = 133.09 x 10^6 / 300 x 410^2

= 2.63 N/mm^2

Mu/bd^2_{span} = 102.74 x 10^6 / 300 x 410^2

= 2.63 N/mm^2

$Pt_{support}$ = 0.896

Pt_{span} = 0.651

Ast $_{support}$ = 0.896 x 300 x 410 / 100

= 1102.08 mm²

Ast $_{span}$ = 0.651 x 300 x 410 / 100

= 800.73 mm^2

Provide 4 – 20 T bars Ast $_{prov}$ = 1256 mm²

Provide 2 – 20 T + 2 – 16 T bars Ast $_{prov}$ = 1030 mm^2

6.1.4.1 Check for shear [5]

Total shear force (Vu) = 182.08 kN

Pt = 0.84

Tc = 0.572 N/mm^2

Tc $_{max}$ = 2.8 N/mm^2

Tc $_{max}$ > Tc ……Ok

Vc = Tc x b x d

= 0.572 x 300 x 410

= 70.356 kN

Vu > Vc

Therefore, shear reinforcement is required.

Vus = Vu – Vc

= 182.08 – 70.356

= 111.72 kN

$$Sv = \frac{0.87\,fyAsvd}{Vus}$$

$$= \frac{0.87 \times 415 \times (\pi/4) \times 10^2 \times 2 \times 410}{111.72 \times 10^3}$$

= 208.13 mm

Provide 10φ – 150 mm c/c stirrups.

6.1.5 TRAPEZOIDAL SLAB DESIGN

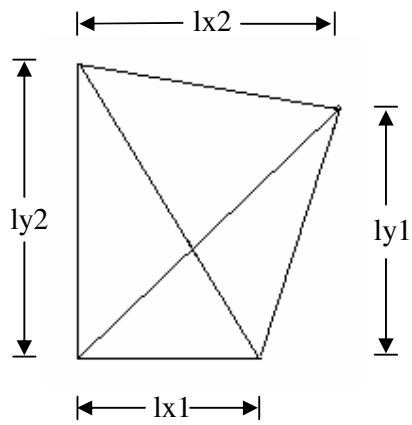


FIGURE: 6.3 TRAPEZOIDAL SLAB

Load intensity = 35.25 kN/m²

For continuous slab, L/d [5] = 26

3.329/d = 26

d = 0.128 m

Assume slab thickness 200 mm

$ly = ly_1 + ly_2 / 2 = 5 + 5.5369 / 2 = 5.2680$ m

$lx = lx_1 + lx_2 / 2 = 2 + 4.6587 / 2 = 3.3293$ m

K = ly / lx = 5.268 / 3.329 = 1.58

For short span ȧx = (-) 0.070

(+) 0.053

For long span ȧy = (-) 0.037

(+) 0.028

Moments per unit width,

At edge Mx [5] = αxwlx²

= 0.07 x 35.25 x 3.33²

= 27.36 kNm

At span Mx [5] = αxwlx²

= 0.053 x 35.25 x 3.33²

= 20.72 kNm

At edge My [5] = αywlx²

=0.037 x 35.25 x 3.329²

= 14.45 kNm

At span My [5] = $\alpha_y w l_x^2$

= 0.028 x 35.25 x 3.33^2

= 10.93 kN.m

For Mx,

At edge Mu/bd² = 27.36 x 10 6/1000 x 160²

= 1.069 N/mm^2

At span Mu/bd² = 20.72 x 10 6/1000 x 160²

= 0.81 N/mm^2

For My,

At edge Mu/bd² = 14.45 x 10² /1000 x 160²

= 0.564 N/mm^2

At span Mu/bd² = 10.93 x 10^6 /1000 x 160²

= 0.427 N/mm^2

For Mx,

At edge pt = 0.313 => Ast = 500.8 mm²

At span pt = 0.236 => Ast = 377.6 mm²

For My,

At edge pt = 0.161 => Ast = 257.6 mm²

At span pt = 0.119 => Ast = 190.4 mm²

For Mx,

Provide at edge 12-200c/c Ast prov. = 565 mm²

Provide at span 12-200c/c Ast prov. = 565 mm²

For My,

Provide at edge 10-200c/c Ast prov. = 393 mm²

Provide at span 10-200c/c Ast prov. = 393 mm²

6.2 PRESSURE CALCULATION OF 6 SEGMENT SILO:

Material to be stored = Cement

Density of material = 15.5 kN/m³

Angle of internal friction (Φ) = 25 °

6.2.1 PRESSURE CALCULATION BY JANSSEN'S METHOD:

(As per IS: 4995 (Part 1) – 1974)[6].

From the three cases given in cl: 5.3 T: 2, the case given below is found to match the criteria of the design.

Granular material with particle dia.>=0.2mm

Angle of wall friction while filling (δ_f) = 18.75

Angle of wall friction while emptying (δ_e) = 15

Co efficient of wall friction while filling (μ_f) = 0.339

Co efficient of wall friction while emptying (μ_e) = 0.268

Pressure ratio while filling (λ_f) = 0.5

Pressure ratio while emptying (λ_e) = 1.0

Area of one compartment (A) = $\Pi \times r^2/6$

$= \Pi \times 9^2/6$

$= 42.39\ m^2$

Perimeter of one compartment (P) = $(2 \times \Pi \times r/6) + (2 \times r)$

= 27.42 m

Hydraulic mean radius of one compartment (R) = A/P

= 42.39/27.42

= 1.55 m

Pressures during filling & emptying condition

As per cl: 6.1.1.1

TABLE: 6.1 DIFFERENT PRESSURES DUE TO FILL BY IS APPROACH

Name of pressure	Filling condition (kN/m^2)	Emptying condition (kN/m^2)
Vertical load transferred to wall per unit area due to friction (Pw)	$=W \times R$ $= 23.96$	$=W \times R$ $= 23.96$
Horizontal pressure on silo Wall (Ph)	$= (W \times R)/\mu_f$ $= 70.59$	$= (W \times R)/\mu_e$ $= 89.43$
Vertical pressure on horizontal cross section of wall (Pv)	$= (W \times R)/(\mu_f \times \lambda_f)$ $= 141.18$	$= (W \times R)/(\mu_e \times \lambda_e)$ $= 89.43$

As in cl: 6.1.1.3 Zo assumes the values given below,

During filling, $Zo_f = R/ (\mu_f \times \lambda_f) = 9.11$

During emptying, $Zo_e = R/ (\mu_e \times \lambda_e) = 5.77$

Lateral pressure $(Pi (z)) = Pi_{max} \times (1-e^{-z/zo})$ ..... (6.1)

Where $Pi_{max} = Ph = (w \times R)/\mu$

So, lateral pressure…

During filling, $Pi (z) = 70.59 \times (1-e^{-z/9.11})$

During emptying $Pi (z) = 89.43 \times (1-e^{-z/5.77})$

TABLE: 6.2 PRESSURE CALCULATION BY IS APPROACH

Depth (Z) (m)	Pressure during Filling (kN/m^2)	Pressure during Emptying (kN/m^2)	Design pressure (kN/m^2)
4	25.09	44.72	44.72
8	41.26	67.08	67.08
12	51.69	78.25	78.25
16	58.40	83.84	83.84
20	62.74	86.64	86.64
24	65.53	88.03	88.03
28	67.33	88.73	88.73
32	68.49	89.08	89.08
36	69.23	89.25	87.90
41.7	69.87	89.36	81.90
44	70.03	89.38	77.50
48	70.23	89.41	70.23

6.2.2 PRESSURE CALCULATION BY JANSSEN'S METHOD

(As per ACI: 313 – 91)[7]

Static pressure given by cl.4.4.2.1 (a)

Angle of repose (Φ) = 25

Co efficient of wall friction (μ) = 0.466

Pressure ratio (λ) = 0.406

Static pressure (Pz) $= \dfrac{WR(1-e^{-\mu\lambda Z/R})}{\mu}$ ..... (6.2)

$$= \frac{15.5\times1.55\times(1-e^{-0.466\times0.406\times Z/1.55})}{0.466}$$

$$= 51.55\times(1-e^{-0.122\times Z})$$

TABLE: 6.3 PRESSURE CALCULATION BY ACI APPROACH

Depth (Z) (m)	Static pressure (Pz) (kN/m^2)	Overpressure factor (Cd)	Design pressure (Pdes = Cd × Pz) (kN/m^2)
4	19.90	1.75	34.82
8	32.09	1.75	56.16
12	39.56	1.9	75.17
16	44.14	1.9	83.87
20	46.95	1.9	89.20
24	48.67	2	97.33
28	49.72	2	99.44
32	50.37	2	100.73
36	50.76	2	101.52
41.7	51.08	2	102.15
44	51.15	1	51.15
48	51.24	1	51.24

6.2.3 AIRY'S METHOD

Co-efficient of friction between cement particles (μ) = 0.466

Co-efficient of friction between cement particles and concrete wall (μ') = 0.34

Diameter of inscribed circle in one compartment = 6 m

Density of cement = 16 kN/m^3

The depth (h') up to which the bin will behave as shallow

$$h' = b\left[\mu + \sqrt{\frac{\mu(1+\mu^2)}{\mu+\mu'}}\right]$$

$$= 6\times\left[0.466 + \sqrt{\frac{0.466\times(1+0.466^2)}{0.466+0.34}}\right]$$

= 7.83 m

Lateral pressure up to 7.83m is calculated as under

$$Ph = Wh\left[\frac{1}{\{\sqrt{\mu(\mu+\mu')}+\sqrt{(1+\mu^2)}\}^2}\right] \qquad \text{..... (6.3)}$$

Now pressure below 7.83m is calculated as below

$$Ph = \frac{Wd}{\mu+\mu'}\left[1-\frac{\sqrt{1+\mu^2}}{\sqrt{\left(\frac{2h}{d}\right)(\mu+\mu')+(1-\mu\mu')}}\right] \qquad \text{..... (6.4)}$$

TABLE: 6.4 PRESSURE CALCULATION BY AIRY'S METHOD

Height from top (Z) (m)	Static Pressure (Ph) (kN/ m^2)
4	21.73
8	43.13
12	53.94
16	61.15
20	66.40
24	70.44
28	73.67
32	76.34
36	78.58
41.7	81.24
44	82.18
48	83.65

6.2.4 COMPARISION OF LATERAL PRESSURE BY VARIOUS THEORIES

TABLE: 6.5 COMPARISION OF PRESSURES

Depth (Z) (m)	Design pressure by Janssen's method (IS approach) (kN/m^2)	Design pressure by Janssen's method (ACI approach) (kN/m^2)	Static pressure by Airy's method (kN/m^2)
4	44.72	34.82	21.73
8	67.08	56.16	43.13
12	78.25	75.17	53.94
16	83.84	83.87	61.15
20	86.64	89.20	66.40
24	88.03	97.33	70.44
28	88.73	99.44	73.67
32	89.08	100.73	76.34
36	87.90	101.52	78.58
41.7	81.90	102.15	81.24
44	77.50	51.15	82.18
48	70.23	51.24	83.65

6.3 ANALYSIS OF COMPARTMENT SILO

6.3.1 ANALYSIS OF INTERNAL FORCES

For analysis of internal forces, analytical method does not take into account axial deformation of members. The matrix method, using computer is the only reasonable way to perform analysis. This method takes into account both flexural and axial force deformation effects.

6.3.1.1 Load cases

Following load cases considered.

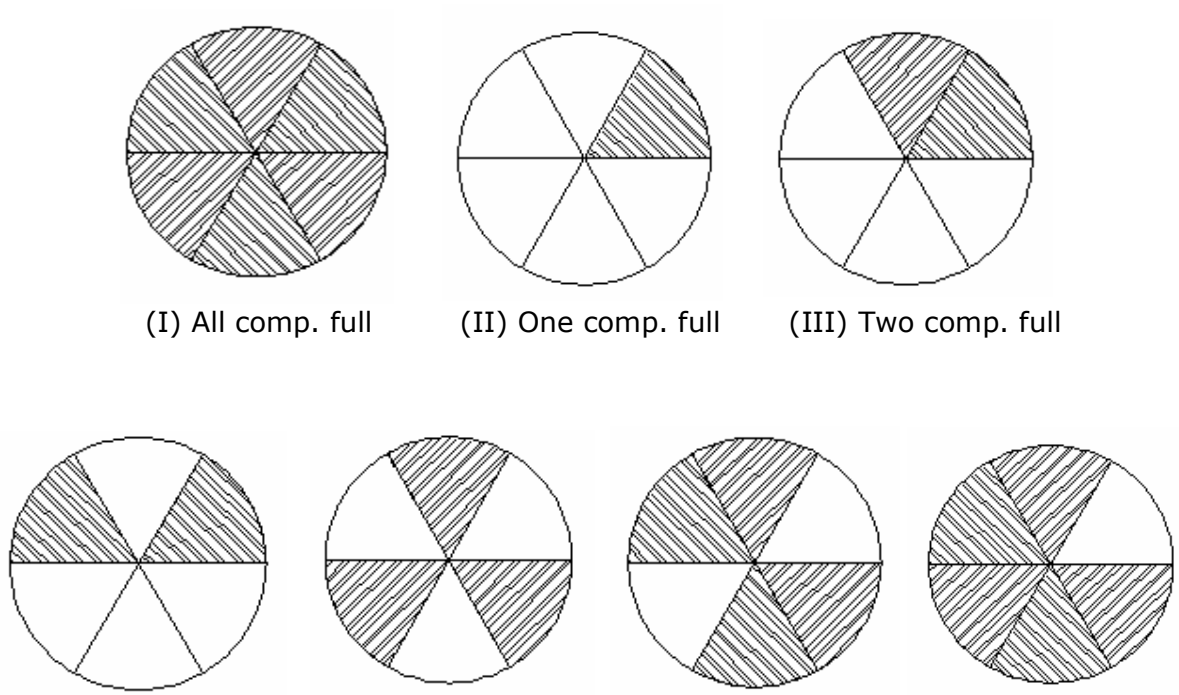


FIGURE: 6.4 LOAD CASES

6.3.1.2 Model for analysis of compartment silo on STAAD.Pro.

Due to fill radial pressure there will be tension, horizontal moment and shear in outer and compartment walls. These are determined by analyzing the system in horizontal plane for a unit height. Analysis is carried out by a 2D-STAAD Pro. Computer plane frame programme.

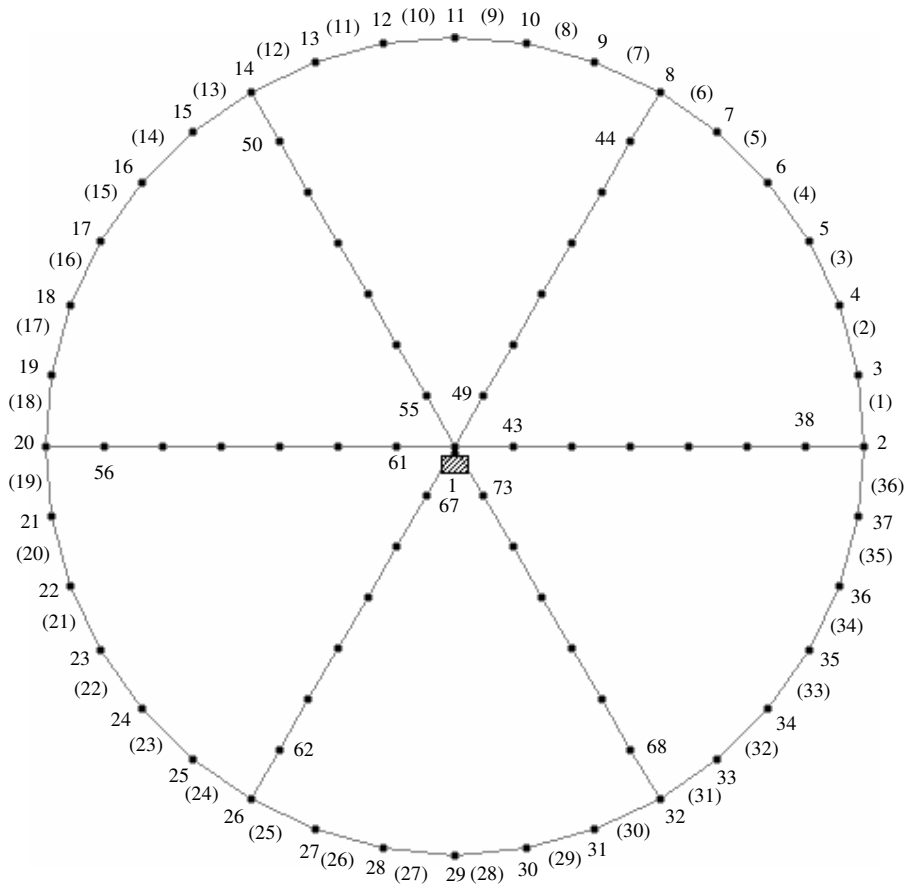


FIGURE: 6.5 NODES AND ELEMENT NUMBERS

[Note: All nodes and beams are not shown for clarity but necessary are shown]
The outer circular wall in plan is assumed to be made of 36 number of straight beam elements. Nodes and element numbers are shown in figure: 6.5. The origin of node coordinates is node1.

For all load cases, pressure of 100kN/m^2 on silo and compartment walls is considered. The inner diameter of silo is 18 m and wall thickness is 450 mm. In

this model node 1 is assumed to be fixed for the support condition for kinematics stability of structure.

Radial and circumferential shears due to change in lateral pressures over silo height are completely ignored. It is possible to analyze without neglecting inter strip shear by analyzing entire structure as a space frame with finite element analysis. However in the zones of maximum lateral pressure, it is probably conservative to ignore inter strip shear.

6.3.1.3 Check on thickness of wall as per IS method [6]

Min.thickness (t) = 10 + 2.5 × (D – 3)/3 ……. (6.5)

= 10 + 2.5 × (18 – 3)/3

= 22.5 cm

Or (t) = 10 + 2.5 × (H-6)/12

= 10 + 2.5 × (60.8 -6)/12

= 21.42 cm

6.3.1.4 Check on thickness of wall as per ACI method [7]

Min.thickness (t) $= \frac{D \times (\max.Pdes.)}{2 \times (0.6 \times fck)}$ ……. (6.6)

$= \frac{18 \times (102.15 \times 10^3)}{2 \times (0.6 \times 30 \times 10^3)}$

= 7.6 cm

Provided thickness is 45 cm. Hence, it is safe.

6.3.2 TEMPERATURE REINFORCEMENT CALCULATION [6]

Wall thickness = 0.45m

Temp. Inside (Ti) = 100°C

Temp. Outside (To) = 5°C

Ca = Surface conductance of concrete to air = 15 kcal/m²h°c

Cs = Surface conductance of concrete to storage materials = 2 kcal/m^2h°C

Cc = Thermal conductivity of reinforced concrete = 2 kcal/m^2h°C

$1/K = 1/Ca + T/Cc + 1/Cs$ ……. (6.7)

$= 1/15 + 0.45/2 + ½$

$= 0.792$

Temperature difference across silo wall

$\Delta_T = (t/Cc) \times (Ti - To) / (1/K)$ ……. (6.8)

$= (0.45/2) \times (100\text{-}5) / 0.792$

$= 27°C$

Moment due to temperature difference across the silo wall

$M_{\Delta T} = \epsilon t\ \Delta_T\ E_c\ I / t$ ……. (6.9)

$= 11 \times 10^{-6} \times 3.12 \times 10^5 \times (1/12\ t^3)/t$

$= 0.286 \times \Delta_T \times t^2$ kg cm / cm

$= 28.6 \times \Delta_T \times t^2$ kg cm / m

$= 28.6 \times 27 \times 45^2$

$= 1563705$ kg cm / m

$Ku = (1.5 \times 1563705) / (100 \times 41^2)$

$= 14$ kg/cm^2

$d = 41$ cm

So, $A_{temp} = 0.411 \times 41 = 16.9$ cm^2

6.3.3 ANALYSIS OF RESULTS

Horizontal moment, axial force and shear for each element for p = 100kN/m² are tabulated in table from computer output file. These values are to be modified for design pressure values.

For the purpose of uniform sign convention is assumed for upper half, i.e node no 2 to 18 and totally opposite sign convention to the upper half is assumed for the lower node no 19 to 36.

So, for upper half portion, node no 2 to 18

- Mz clockwise is +ve.
- Shear is +ve in the direction of radial pressure
- Axial tension in the element is +ve.

And for lower half portion, node no 19 to 36

- Mz is anticlockwise is +ve
- Shear is +ve in the direction opposite to radial pressure.
- Axial tension in the element is +ve.

TABLE : 6.6 [A] RESULTS OF STAAD.PRO-2D MODEL FOR WALL ANALYSIS

			Beam	36		Beam	1		Beam	2		Beam	3		Beam	4		Beam	5
l/c		37		2			3			4			5			6			7
1	moment	30.63		-88.79	-88.79		30.63	-30.63		-5.00	5.00		-16.99	16.99		-4.99	4.99		30.63
	axial		817.07			817.07			811.84			809.19			809.19			811.84	
	shear	41.37		115.51	-115.51		-41.37	-101.15		-55.73	-86.08		-70.80	-70.80		-86.09	-55.74		-101.14
2	moment	-75.22		144.02	-435.44		124.31	-124.31		-66.23	66.23		-130.39	130.39		-66.23	66.23		124.31
	axial		63.13			471.28			443.30			429.10			429.10			443.30	
	shear	43.85		-43.85	-276.76		119.88	-199.90		43.02	-119.34		-37.54	-37.54		-119.34	43.02		-199.90
3	moment	-75.22		192.34	-409.01		129.53	-129.53		-50.06	50.06		-124.28	124.28		-90.89	90.89		49.09
	axial		133.67			545.45			519.84			505.69			503.41			513.08	
	shear	74.66		-74.66	-256.59		99.71	-192.91		36.03	-125.75		-31.13	-57.16		-99.72	10.79		-167.67
4	moment	-87.28		206.55	-370.09		112.68	-112.68		-43.75	43.75		-94.43	94.43		-37.84	37.84		124.32
	axial		132.14			540.22			517.14			505.59			505.92			518.12	
	shear	76.03		-76.03	-242.52		85.64	-178.15		21.27	-110.75		-46.13	-42.37		-114.52	24.92		-181.80
5	moment	-82.06		232.98	-321.77		112.68	-112.68		-15.36	15.36		-58.48	58.48		-15.36	15.36		112.68
	axial		206.30			610.77			591.97			582.42			582.42			591.97	
	shear	96.20		-96.20	-211.71		54.83	-160.06		3.18	-105.92		-50.96	-50.96		-105.93	3.18		-160.06
6	moment	117.48		-298.14	284.11		-81.63	81.63		42.37	-42.37		84.13	-84.13		42.37	-42.37		-81.63
	axial		685.00			276.79			294.99			304.24			304.24			294.99	
	shear	-36.72		193.60	129.07		-129.07	79.04		-79.04	26.62		-26.62	-26.62		26.62	-79.04		79.04
7	moment	105.85		-232.80	346.65		-93.69	93.69		61.23	-61.23		113.40	-113.40		61.23	-61.23		-93.69
	axial		753.95			345.80			368.54			380.09			380.09			368.54	
	shear	-2.49		159.37	161.25		-161.25	98.75		-98.75	33.26		-33.26	-33.25		33.25	-98.75		98.75

TABLE: 6.6 [B] RESULTS OF STAAD.PRO-2D MODEL FOR WALL ANALYSIS

			Beam 6			Beam 7			Beam 8			Beam 9			Beam 10			Beam 11
Vc		7		8			9			10			11			12			13
1	moment	-30.63		88.79	-88.79		30.62	-30.62		-4.99	4.99		-16.99	16.99		-4.99	4.99		30.62
	axial		817.07			817.07			811.84			809.19			809.19			811.84	
	shear	-41.37		-115.51	-115.52		-41.37	-101.14		-55.74	-86.09		-70.80	-70.80		-86.09	-55.74		-101.14
2	moment	-124.31		435.44	144.02		-75.22	75.22		-24.67	24.67		6.11	-6.11		16.17	-16.17		5.22
	axial		471.28			63.13			69.78			74.32			76.59			76.54	
	shear	119.88		-276.76	43.85		-43.85	32.22		-32.22	19.62		-19.62	6.41		-6.41	-6.98		6.98
3	moment	-49.09		291.42	-291.42		49.09	-49.09		-90.89	90.89		-124.28	124.28		-50.06	50.06		129.53
	axial		534.40			534.40			513.08			503.41			505.69			519.84	
	shear	76.03		-232.91	-232.91		76.03	-167.67		10.79	-99.72		-57.16	-31.13		-125.75	36.03		-192.91
4	moment	-124.32		387.11	170.44		-70.00	70.00		-8.49	8.49		12.22	-12.22		-8.49	8.49		-70.00
	axial		541.82			137.29			146.33			150.91			150.91			146.33	
	shear	89.07		-245.95	64.02		-64.02	39.21		-39.21	13.20		-13.20	-13.20		13.20	-39.21		39.21
5	moment	-112.68		321.76	232.98		-82.06	82.06		10.37	-10.37		41.49	-41.49		10.37	-10.37		-82.06
	axial		610.77			206.30			219.88			226.77			226.77			219.88	
	shear	54.83		-211.71	96.20		-96.20	58.92		-58.92	19.84		-19.84	-19.84		19.84	-58.92		58.92
6	moment	81.63		-284.11	-298.14		117.48	-117.48		-2.81	2.81		-59.05	59.05		-49.55	49.55		25.40
	axial		276.79			685.00			668.22			658.37			655.77			660.48	
	shear	-129.07		129.07	-193.60		36.72	-155.11		-1.77	-114.29		-42.59	-72.39		-84.50	-30.66		-126.22
7	moment	93.69		-346.65	-232.80		105.84	-105.84		19.67	-19.67		-23.10	23.10		-21.17	21.17		25.41
	axial		345.80			753.95			742.06			734.87			732.59			735.30	
	shear	-161.25		161.25	-159.37		2.49	-133.37		-23.51	-105.70		-51.18	-77.21		-79.67	-48.76		-108.13

TABLE : 6.6 [C] RESULTS OF STAAD.PRO-2D MODEL FOR WALL ANALYSIS

			Beam 12			Beam 13			Beam 14			Beam 15			Beam 16			Beam 17
l/c		13		14			15			16			17			18		19
1	moment	-30.62		88.79	-88.79		30.63	-30.63		-4.99	4.99		-16.99	16.99		-5.00	5.00		30.63
	axial		817.07			817.07			811.84			809.19			809.19			811.84	
	shear	-41.37		-115.52	-115.51		-41.37	-101.14		-55.74	-86.09		-70.80	-70.80		-86.08	-55.73		-101.15
2	moment	-5.22		-26.43	48.33		0.01	-0.01		28.39	-28.39		35.96	-35.96		22.48	-22.48		-11.63
	axial		74.17			70.55			74.82			76.83			76.50			73.85	
	shear	-20.17		20.17	30.81		-30.81	18.09		-18.09	4.82		-4.82	-8.59		8.59	-21.74		21.74
3	moment	-129.53		409.01	192.34		-75.21	75.21		3.72	-3.72		42.07	-42.07		38.65	-38.65		-6.42
	axial		545.44			133.67			144.60			151.15			153.09			150.39	
	shear	99.71		-256.59	74.66		-74.66	50.32		-50.32	24.44		-24.44	-2.18		2.18	-28.73		28.73
4	moment	70.00		-170.44	-387.11		124.32	-124.32		-37.84	37.84		-94.43	94.43		-43.75	43.75		112.68
	axial		137.29			541.82			518.12			505.92			505.59			517.14	
	shear	-64.02		64.02	-245.95		89.07	-181.80		24.92	-114.52		-42.37	-46.13		-110.75	21.27		-178.15
5	moment	82.06		-232.98	-321.76		112.68	-112.68		-15.36	15.36		-58.48	58.48		-15.36	15.36		112.68
	axial		206.30			610.77			591.97			582.42			582.42			591.97	
	shear	-96.20		96.20	-211.71		54.83	-160.06		3.18	-105.93		-50.96	-50.96		-105.92	3.18		-160.06
6	moment	-25.40		163.54	-163.54		25.40	-25.40		-49.55	49.55		-59.05	59.05		-2.81	2.81		117.48
	axial		672.36			672.36			660.48			655.77			658.37			668.22	
	shear	9.61		-166.49	-166.49		9.61	-126.22		-30.66	-84.50		-72.39	-42.59		-114.29	-1.77		-155.11
7	moment	-25.41		115.21	-137.11		30.62	-30.62		-33.38	33.38		-52.94	52.94		-27.48	27.48		42.26
	axial		742.90			746.53			737.02			732.36			732.69			738.00	
	shear	-21.20		-135.68	-146.32		-10.56	-119.24		-37.64	-90.91		-65.97	-62.21		-94.67	-33.99		-122.89

TABLE : 6.6 [D] RESULTS OF STAAD.PRO-2D MODEL FOR WALL ANALYSIS

			Beam	18		Beam	19		Beam	20		Beam	21		Beam	22		Beam	23
Vc		19		20			21			22			23			24			25
1	moment	-30.63		88.79	88.79		-30.63	30.63		5.00	-5.00		16.99	-16.99		4.99	-4.99		-30.63
	axial		817.07			817.07			811.84			809.19			809.19			811.84	
	shear	-41.37		-115.51	115.51		41.37	101.15		55.73	86.08		70.80	70.80		86.09	55.74		101.14
2	moment	11.63		-65.34	-62.54		12.06	-12.06		-18.86	18.86		-29.27	29.27		-18.86	18.86		12.06
	axial		68.95			69.01			73.55			75.86			75.86			73.55	
	shear	-34.24		34.24	-32.18		32.18	-19.71		19.71	-6.64		6.64	6.64		-6.64	19.71		-19.71
3	moment	6.42		-91.77	-110.87		12.05	-12.05		-47.25	47.25		-65.23	65.23		-41.34	41.34		23.69
	axial		143.12			139.56			148.37			152.68			152.35			147.40	
	shear	-54.41		54.41	-62.99		62.99	-37.80		37.80	-11.46		11.46	15.23		-15.23	41.45		-41.45
4	moment	-112.68		370.09	-206.55		87.28	-87.28		5.81	-5.81		-35.38	35.38		-35.03	35.03		6.84
	axial		540.22			132.14			143.33			150.17			152.45			150.09	
	shear	85.64		-242.52	-76.03		76.03	-51.93		51.93	-26.25		26.25	0.22		-0.22	26.69		-26.69
5	moment	-112.68		321.77	-232.98		82.06	-82.06		-10.37	10.37		-41.49	41.49		-10.37	10.37		82.06
	axial		610.77			206.30			219.88			226.77			226.77			219.88	
	shear	54.83		-211.71	-96.20		96.20	-58.92		58.92	-19.84		19.84	19.84		-19.84	58.92		-58.92
6	moment	-117.48		298.14	-284.11		81.63	-81.63		-42.37	42.37		-84.13	84.13		-42.37	42.37		81.63
	axial		685.00			276.79			294.99			304.24			304.24			294.99	
	shear	36.72		-193.60	-129.07		129.07	-79.04		79.04	-26.62		26.62	26.62		-26.62	79.04		-79.04
7	moment	-42.26		154.13	151.33		-42.68	42.68		23.86	-23.86		46.26	-46.26		23.86	-23.86		-42.68
	axial		748.13			748.06			738.29			733.33			733.33			738.29	
	shear	-7.13		-149.75	147.69		9.19	120.85		36.03	92.72		64.16	64.16		92.72	36.03		120.85

TABLE: 6.6 [E] RESULTS OF STAAD.PRO-2D MODEL FOR WALL ANALYSIS

			Beam	24		Beam	25		Beam	26		Beam	27		Beam	28		Beam	29
l/c		25		26			27			28			29			30			31
1	moment	30.63		-88.79	88.79		-30.62	30.62		4.99	-4.99		16.99	-16.99		4.99	-4.99		-30.62
	axial		817.07			817.07			811.84			809.19			809.19			811.84	
	shear	41.37		115.51	115.52		41.37	101.14		55.74	86.09		70.80	70.80		86.09	55.74		101.14
2	moment	-12.06		62.54	-65.34		11.63	-11.63		-22.48	22.48		-35.96	35.96		-28.39	28.39		-0.01
	axial		69.01			68.95			73.85			76.50			76.83			74.82	
	shear	32.18		-32.18	-34.24		34.24	-21.74		21.74	-8.59		8.59	4.82		-4.82	18.09		-18.09
3	moment	-23.69		127.88	-127.88		23.69	-23.69		-41.34	41.34		-65.23	65.23		-47.25	47.25		12.05
	axial		137.96			137.96			147.40			152.35			152.68			148.37	
	shear	66.42		-66.42	-66.42		66.42	-41.45		41.45	-15.23		15.23	11.46		-11.46	37.80		-37.80
4	moment	-6.84		88.97	-113.67		11.63	-11.63		-50.87	50.87		-71.91	71.91		-50.87	50.87		11.63
	axial		143.18			139.49			148.67			153.33			153.33			148.67	
	shear	52.35		-52.35	-65.05		65.05	-39.84		39.84	-13.41		13.41	13.41		-13.41	39.84		-39.84
5	moment	-82.06		232.98	321.77		-112.68	112.68		15.36	-15.36		58.48	-58.48		15.36	-15.36		-112.68
	axial		206.30			610.77			591.97			582.42			582.42			591.97	
	shear	96.20		-96.20	211.72		-54.84	160.06		-3.18	105.92		50.96	50.96		105.92	-3.18		160.06
6	moment	-81.63		284.11	298.14		-117.48	117.48		2.81	-2.81		59.05	-59.05		49.55	-49.55		-25.40
	axial		276.79			685.00			668.22			658.37			655.77			660.48	
	shear	129.07		-129.07	193.60		-36.72	155.11		1.77	114.29		42.59	72.39		84.50	30.66		126.22
7	moment	42.68		-151.32	154.13		-42.25	42.25		27.47	-27.47		52.94	-52.94		33.38	-33.38		-30.62
	axial		748.06			748.13			738.00			732.69			732.36			737.02	
	shear	9.19		147.69	149.75		7.13	122.89		33.99	94.68		62.21	65.97		90.91	37.65		119.23

TABLE: 6.6 [F] RESULTS OF STAAD.PRO-2D MODEL FOR WALL ANALYSIS

			Beam 30			Beam 31			Beam 32			Beam 33			Beam 34			Beam 35
Vc		31		32			33			34			35			36		37
1	moment	30.62		-88.79	88.79		-30.63	30.63		4.99	-4.99		16.99	-16.99		5.00	-5.00		-30.63
	axial		817.07			817.07			811.84			809.19			809.19			811.84	
	shear	41.37		115.52	115.51		41.37	101.14		55.74	86.09		70.80	70.80		86.08	55.73		101.15
2	moment	0.01		48.33	-26.43		-5.22	5.22		-16.17	16.17		-6.11	6.11		24.67	-24.67		75.22
	axial		70.55			74.17			76.54			76.59			74.32			69.78	
	shear	30.81		-30.81	-20.17		20.17	-6.98		6.98	6.41		-6.41	19.62		-19.62	32.22		-32.22
3	moment	-12.05		110.87	-91.77		6.42	-6.42		-38.65	38.65		-42.07	42.07		-3.72	3.72		75.22
	axial		139.56			143.12			150.39			153.09			151.14			144.60	
	shear	62.99		-62.99	-54.41		54.41	-28.73		28.73	-2.18		2.18	24.44		-24.44	50.32		-50.32
4	moment	-11.63		113.67	-88.97		6.84	-6.84		-35.03	35.03		-35.38	35.38		5.81	-5.81		87.28
	axial		139.49			143.18			150.09			152.45			150.17			143.33	
	shear	65.05		-65.05	-52.35		52.35	-26.69		26.69	-0.22		0.22	26.25		-26.25	51.93		-51.93
5	moment	112.68		-321.77	-232.98		82.06	-82.06		-10.37	10.37		-41.49	41.49		-10.37	10.37		82.06
	axial		610.77			206.30			219.88			226.77			226.77			219.88	
	shear	-54.84		211.72	-96.20		96.20	-58.92		58.92	-19.84		19.84	19.84		-19.84	58.92		-58.92
6	moment	25.40		-163.54	163.54		-25.40	25.40		49.55	-49.55		59.05	-59.05		2.81	-2.81		-117.48
	axial		672.36			672.36			660.48			655.77			658.37			668.22	
	shear	-9.61		166.49	166.49		-9.61	126.22		30.66	84.50		72.39	42.59		114.29	1.77		155.11
7	moment	30.62		-137.12	115.21		-25.41	25.41		21.17	-21.17		23.10	-23.10		-19.67	19.67		-105.85
	axial		746.53			742.90			735.30			732.59			734.87			742.06	
	shear	10.56		146.33	135.68		21.20	108.13		48.75	79.67		77.21	51.18		105.70	23.51		133.37

TABLE: 6.6 [G] RESULTS OF STAAD.PRO-2D MODEL FOR WALL ANALYSIS

			Beam	37		Beam	38		Beam	39		Beam	40		Beam	41		Beam	42		Beam	43
Wt		1		43			42			41			40			39			38			2
1	moment	0.00		0.00	0.00		0.00	0.00		0.00	0.00		0.00	0.00		0.00	0.00		0.00	0.00		0.00
	axial		87.72			87.72			87.72			87.72			87.72			87.72			87.72	
	shear	0.00		0.00	0.00		0.00	0.00		0.00	0.00		0.00	0.00		0.00	0.00		0.00	0.00		0.00
2	moment	718.58		-202.79	202.79		147.70	-147.70		332.89	-332.89		352.76	-352.76		207.33	-207.33		-103.41	103.41		-579.45
	axial		185.45			185.45			185.45			185.45			185.45			185.45			185.45	
	shear	465.46		-336.89	336.89		-208.32	208.32		-79.74	79.74		48.83	-48.83		177.40	-177.40		305.97	-305.97		434.54
3	moment	699.65		-189.69	189.69		154.97	-154.97		334.32	-334.32		348.36	-348.36		197.09	-197.09		-119.48	119.48		-601.36
	axial		122.05			122.05			122.05			122.05			122.05			122.05			122.05	
	shear	460.92		-332.35	332.35		-203.78	203.78		-75.21	75.21		53.37	-53.37		181.94	-181.94		310.51	-310.51		439.08
4	moment	717.95		-201.85	201.85		148.95	-148.95		334.44	-334.44		354.63	-354.63		209.51	-209.51		-100.92	100.92		-576.65
	axial		107.26			107.26			107.26			107.26			107.26			107.26			107.26	
	shear	465.70		-337.13	337.13		-208.56	208.56		-79.99	79.99		48.59	-48.59		177.16	-177.16		305.73	-305.73		434.30
5	moment	736.88		-214.95	214.95		141.69	-141.69		333.01	-333.01		359.03	-359.03		219.75	-219.75		-84.85	84.85		-554.74
	axial		43.86			43.86			43.86			43.86			43.86			43.86			43.86	
	shear	470.24		-341.67	341.67		-213.10	213.10		-84.52	84.52		44.05	-44.05		172.62	-172.62		301.19	-301.19		429.76
6	moment	-719.20		203.72	-203.72		-146.46	146.46		-331.33	331.33		-350.89	350.89		-205.15	205.15		105.90	-105.90		582.25
	axial		19.54			19.54			19.54			19.54			19.54			19.54			19.54	
	shear	-465.22		336.65	-336.65		208.07	-208.07		79.50	-79.50		-49.07	49.07		-177.64	177.64		-306.21	306.21		-434.78
7	moment	-718.58		202.79	-202.79		-147.70	147.70		-332.89	332.89		-352.76	352.76		-207.33	207.33		103.41	-103.41		579.45
	axial		97.72			97.72			97.72			97.72			97.72			97.72			97.72	
	shear	-465.46		336.89	-336.89		208.32	-208.32		79.74	-79.74		-48.83	48.83		-177.40	177.40		-305.97	305.97		-434.54

TABLE: 6.6 [H] RESULTS OF STAAD.PRO-2D MODEL FOR WALL ANALYSIS

			Beam	44		Beam	45		Beam	46		Beam	47		Beam	48		Beam	49		Beam	50
lk		1		49			48			47			46			45			44			8
1	moment	0.00		0.00	0.00		0.00	0.00		0.00	0.00		0.00	0.00		0.00	0.00		0.00	0.00		0.00
	axial		87.72			87.72			87.72			87.72			87.72			87.72			87.72	
	shear	0.00		0.00	0.00		0.00	0.00		0.00	0.00		0.00	0.00		0.00	0.00		0.00	0.00		0.00
2	moment	-718.58		202.78	-202.78		-147.70	147.70		-332.89	332.89		-352.76	352.76		-207.33	207.33		103.41	-103.41		579.45
	axial		185.45			185.45			185.45			185.45			185.45			185.45			185.45	
	shear	-465.46		336.89	-336.89		208.32	-208.32		79.75	-79.74		-48.83	48.83		-177.40	177.40		-305.97	305.97		-434.54
3	moment	0.00		0.00	0.00		0.00	0.00		0.00	0.00		0.00	0.00		0.00	0.00		0.00	0.00		0.00
	axial		370.89			370.89			370.89			370.89			370.89			370.89			370.89	
	shear	0.00		0.00	0.00		0.00	0.00		0.00	0.00		0.00	0.00		0.00	0.00		0.00	0.00		0.00
4	moment	-737.51		215.88	-215.88		-140.44	140.44		-331.46	331.46		-357.16	357.16		-217.57	217.57		87.34	-87.34		557.55
	axial		122.05			122.05			122.05			122.05			122.05			122.05			122.05	
	shear	-470.00		341.42	-341.42		212.85	-212.85		84.28	-84.28		-44.29	44.29		-172.86	172.86		-301.43	301.43		-430.00
5	moment	-736.88		214.95	-214.95		-141.69	141.69		-333.01	333.01		-359.03	359.03		-219.75	219.75		84.85	-84.85		554.74
	axial		43.86			43.86			43.86			43.86			43.86			43.86			43.86	
	shear	-470.24		341.67	-341.67		213.10	-213.10		84.52	-84.52		-44.05	44.05		-172.62	172.62		-301.19	301.19		-429.76
6	moment	719.20		-203.72	203.72		146.46	-146.46		331.33	-331.33		350.89	-350.89		205.15	-205.15		-105.90	105.90		-582.26
	axial		19.54			19.54			19.54			19.54			19.54			19.54			19.54	
	shear	465.22		-336.65	336.65		-208.07	208.07		-79.50	79.50		49.07	-49.07		177.64	-177.64		306.21	-306.21		434.78
7	moment	718.58		-202.78	202.78		147.70	-147.70		332.89	-332.89		352.76	-352.76		207.33	-207.33		-103.41	103.41		-579.45
	axial		97.72			97.73			97.72			97.72			97.72			97.72			97.73	
	shear	465.46		-336.89	336.89		-208.32	208.32		-79.75	79.74		48.83	-48.83		177.40	-177.40		305.97	-305.97		434.54

TABLE: 6.6 [I] RESULTS OF STAAD.PRO-2D MODEL FOR WALL ANALYSIS

			Beam	51		Beam	52		Beam	53		Beam	54		Beam	55		Beam	56		Beam	57
k		1		55			54			53			52			51			50			14
1	moment	0.00		0.00	0.00		0.00	0.00		0.00	0.00		0.00	0.00		0.00	0.00		0.00	0.00		0.00
	axial		87.72			87.72			87.72			87.72			87.72			87.72			87.72	
	shear	0.00		0.00	0.00		0.00	0.00		0.00	0.00		0.00	0.00		0.00	0.00		0.00	0.00		0.00
2	moment	-18.93		13.10	-13.10		7.26	-7.26		1.43	-1.43		-4.40	4.40		-10.24	10.24		-16.07	16.07		-21.91
	axial		63.40			63.40			63.40			63.40			63.40			63.40			63.40	
	shear	-4.54		4.54	-4.54		4.54	-4.54		4.54	-4.54		4.54	-4.54		4.54	-4.54		4.54	-4.54		4.54
3	moment	699.65		-189.69	189.69		154.97	-154.97		334.31	-334.31		348.36	-348.36		197.09	-197.09		-119.48	119.48		-601.36
	axial		122.05			122.05			122.05			122.05			122.05			122.05			122.05	
	shear	460.92		-332.35	332.35		-203.78	203.78		-75.21	75.21		53.37	-53.37		181.94	-181.94		310.51	-310.51		439.08
4	moment	-737.51		215.88	-215.88		-140.44	140.44		-331.46	331.46		-357.16	357.16		-217.57	217.57		87.34	-87.34		557.55
	axial		122.05			122.05			122.05			122.05			122.05			122.05			122.05	
	shear	-470.00		341.42	-341.42		212.85	-212.85		84.28	-84.28		-44.29	44.29		-172.86	172.86		-301.43	301.43		-430.00
5	moment	-736.88		214.95	-214.95		-141.69	141.69		-333.01	333.01		-359.03	359.03		-219.75	219.75		84.85	-84.85		554.74
	axial		43.86			43.86			43.86			43.86			43.86			43.86			43.86	
	shear	-470.24		341.67	-341.67		213.10	-213.10		84.52	-84.52		-44.05	44.05		-172.62	172.62		-301.19	301.19		-429.76
6	moment	0.00		0.00	0.00		0.00	0.00		0.00	0.00		0.00	0.00		0.00	0.00		0.00	0.00		0.00
	axial		214.52			214.52			214.52			214.52			214.52			214.52			214.52	
	shear	0.00		0.00	0.00		0.00	0.00		0.00	0.00		0.00	0.00		0.00	0.00		0.00	0.00		0.00
7	moment	18.93		-13.10	13.10		-7.26	7.26		-1.43	1.43		4.40	-4.40		10.24	-10.24		16.07	-16.07		21.90
	axial		151.12			151.12			151.12			151.12			151.12			151.12			151.12	
	shear	4.54		-4.54	4.54		-4.54	4.54		-4.54	4.54		-4.54	4.54		-4.54	4.54		-4.54	4.54		-4.54

TABLE: 6.6 [J] RESULTS OF STAAD.PRO-2D MODEL FOR WALL ANALYSIS

			Beam	58		Beam	59		Beam	60		Beam	61		Beam	62		Beam	63		Beam	64
k		1		61			60			59			58			57			56			20
1	moment	0.00		0.00	0.00		0.00	0.00		0.00	0.00		0.00	0.00		0.00	0.00		0.00	0.00		0.00
	axial		87.72			87.72			87.72			87.72			87.72			87.72			87.72	
	shear	0.00		0.00	0.00		0.00	0.00		0.00	0.00		0.00	0.00		0.00	0.00		0.00	0.00		0.00
2	moment	-0.62		0.94	-0.94		1.25	-1.25		1.56	-1.56		1.87	-1.87		2.18	-2.18		2.49	-2.49		2.80
	axial		78.19			78.19			78.19			78.19			78.19			78.19			78.19	
	shear	0.24		-0.24	0.24		-0.24	0.24		-0.24	0.24		-0.24	0.24		-0.24	0.24		-0.24	0.24		-0.24
3	moment	-19.55		14.03	-14.03		8.51	-8.51		2.99	-2.99		-2.54	2.54		-8.06	8.06		-13.58	13.58		-19.10
	axial		141.59			141.59			141.59			141.59			141.59			141.59			141.59	
	shear	-4.30		4.30	-4.30		4.30	-4.30		4.30	-4.30		4.30	-4.30		4.30	-4.30		4.30	-4.30		4.30
4	moment	717.95		-201.85	201.85		148.95	-148.95		334.44	-334.44		354.63	-354.63		209.51	-209.51		-100.92	100.92		-576.65
	axial		107.26			107.26			107.26			107.26			107.26			107.26			107.26	
	shear	465.70		-337.13	337.13		-208.56	208.56		-79.99	79.99		48.59	-48.59		177.16	-177.16		305.73	-305.73		434.30
5	moment	736.88		-214.95	214.95		141.69	-141.69		333.01	-333.01		359.03	-359.03		219.75	-219.75		-84.85	84.85		-554.74
	axial		43.86			43.86			43.86			43.86			43.86			43.86			43.86	
	shear	470.24		-341.67	341.67		-213.10	213.10		-84.52	84.52		44.05	-44.05		172.62	-172.62		301.19	-301.19		429.76
6	moment	719.20		-203.72	203.72		146.46	-146.46		331.33	-331.33		350.89	-350.89		205.15	-205.15		-105.90	105.90		-582.25
	axial		19.54			19.54			19.54			19.54			19.54			19.54			19.54	
	shear	465.22		-336.65	336.65		-208.07	208.07		-79.50	79.50		49.07	-49.07		177.64	-177.64		306.21	-306.21		434.78
7	moment	0.62		-0.94	0.94		-1.25	1.25		-1.56	1.56		-1.87	1.87		-2.18	2.18		-2.49	2.49		-2.80
	axial		165.91			165.91			165.91			165.91			165.91			165.91			165.91	
	shear	-0.24		0.24	-0.24		0.24	-0.24		0.24	-0.24		0.24	-0.24		0.24	-0.24		0.24	-0.24		0.24

TABLE: 6.6 [K] RESULTS OF STAAD.PRO-2D MODEL FOR WALL ANALYSIS

			Beam	65		Beam	66		Beam	67		Beam	68		Beam	69		Beam	70		Beam	71
k:		1		67			66			65			64			63			62			26
1	moment	0.00		0.00	0.00		0.00	0.00		0.00	0.00		0.00	0.00		0.00	0.00		0.00	0.00		0.00
	axial		87.72			87.72			87.72			87.72			87.72			87.72			87.72	
	shear	0.00		0.00	0.00		0.00	0.00		0.00	0.00		0.00	0.00		0.00	0.00		0.00	0.00		0.00
2	moment	0.62		-0.94	0.94		-1.25	1.25		-1.56	1.56		-1.87	1.87		-2.18	2.18		-2.49	2.49		-2.80
	axial		78.19			78.19			78.19			78.19			78.19			78.19			78.19	
	shear	-0.24		0.24	-0.24		0.24	-0.24		0.24	-0.24		0.24	-0.24		0.24	-0.24		0.24	-0.24		0.24
3	moment	0.00		0.00	0.00		0.00	0.00		0.00	0.00		0.00	0.00		0.00	0.00		0.00	0.00		0.00
	axial		156.38			156.38			156.38			156.38			156.38			156.38			156.38	
	shear	0.00		0.00	0.00		0.00	0.00		0.00	0.00		0.00	0.00		0.00	0.00		0.00	0.00		0.00
4	moment	-18.31		12.16	-12.16		6.02	-6.02		-0.13	0.13		-6.27	6.27		-12.42	12.42		-18.56	18.56		-24.71
	axial		141.59			141.59			141.59			141.59			141.59			141.59			141.59	
	shear	-4.78		4.78	-4.78		4.78	-4.78		4.78	-4.78		4.78	-4.78		4.78	-4.78		4.78	-4.78		4.78
5	moment	-736.88		214.95	-214.95		-141.69	141.69		-333.01	333.01		-359.03	359.03		-219.75	219.75		84.85	-84.85		554.75
	axial		43.86			43.86			43.86			43.86			43.86			43.86			43.86	
	shear	-470.24		341.67	-341.67		213.10	-213.10		84.52	-84.52		-44.05	44.05		-172.62	172.62		-301.19	301.19		-429.76
6	moment	-719.20		203.72	-203.72		-146.46	146.46		-331.33	331.33		-350.89	350.89		-205.15	205.15		105.90	-105.90		582.26
	axial		19.54			19.54			19.54			19.54			19.54			19.54			19.54	
	shear	-465.22		336.65	-336.65		208.07	-208.07		79.50	-79.50		-49.07	49.07		-177.64	177.64		-306.21	306.21		-434.78
7	moment	-0.62		0.93	-0.93		1.25	-1.25		1.56	-1.56		1.87	-1.87		2.18	-2.18		2.49	-2.49		2.80
	axial		165.91			165.91			165.91			165.91			165.91			165.91			165.91	
	shear	0.24		-0.24	0.24		-0.24	0.24		-0.24	0.24		-0.24	0.24		-0.24	0.24		-0.24	0.24		-0.24

TABLE: 6.6 [L] RESULTS OF STAAD.PRO-2D MODEL FOR WALL ANALYSIS

			Beam	72		Beam	73		Beam	74		Beam	75		Beam	76		Beam	77		Beam	78
lk		1		73			72			71			70			69			68			32
1	moment	0.00		0.00	0.00		0.00	0.00		0.00	0.00		0.00	0.00		0.00	0.00		0.00	0.00		0.00
	axial		87.72			87.72			87.72			87.72			87.72			87.72			87.72	
	shear	0.00		0.00	0.00		0.00	0.00		0.00	0.00		0.00	0.00		0.00	0.00		0.00	0.00		0.00
2	moment	-18.93		13.10	-13.10		7.26	-7.26		1.43	-1.43		-4.40	4.40		-10.24	10.24		-16.07	16.07		-21.90
	axial		63.40			63.40			63.40			63.40			63.40			63.40			63.40	
	shear	-4.54		4.54	-4.54		4.54	-4.54		4.54	-4.54		4.54	-4.54		4.54	-4.54		4.54	-4.54		4.54
3	moment	-19.55		14.03	-14.03		8.51	-8.51		2.99	-2.99		-2.54	2.54		-8.06	8.06		-13.58	13.58		-19.10
	axial		141.59			141.59			141.59			141.59			141.59			141.59			141.59	
	shear	-4.30		4.30	-4.30		4.30	-4.30		4.30	-4.30		4.30	-4.30		4.30	-4.29		4.29	-4.30		4.30
4	moment	-18.31		12.16	-12.16		6.02	-6.02		-0.13	0.13		-6.27	6.27		-12.42	12.42		-18.56	18.56		-24.71
	axial		141.59			141.59			141.59			141.59			141.59			141.59			141.59	
	shear	-4.78		4.78	-4.78		4.78	-4.78		4.78	-4.78		4.78	-4.78		4.78	-4.78		4.78	-4.78		4.78
5	moment	-736.88		214.95	-214.95		-141.69	141.69		-333.01	333.01		-359.03	359.03		-219.75	219.75		84.85	-84.85		554.75
	axial		43.86			43.86			43.86			43.86			43.86			43.86			43.86	
	shear	-470.24		341.67	-341.67		213.10	-213.10		84.52	-84.52		-44.05	44.05		-172.62	172.62		-301.19	301.19		-429.76
6	moment	0.00		0.00	0.00		0.00	0.00		0.00	0.00		0.00	0.00		0.00	0.00		0.00	0.00		0.00
	axial		214.52			214.52			214.52			214.52			214.52			214.52			214.52	
	shear	0.00		0.00	0.00		0.00	0.00		0.00	0.00		0.00	0.00		0.00	0.00		0.00	0.00		0.00
7	moment	18.93		-13.10	13.10		-7.26	7.26		-1.43	1.43		4.40	-4.40		10.24	-10.24		16.07	-16.07		21.91
	axial		151.12			151.12			151.12			151.12			151.12			151.12			151.12	
	shear	4.54		-4.54	4.54		-4.54	4.54		-4.54	4.54		-4.54	4.54		-4.54	4.54		-4.54	4.54		-4.54

Following points to be noted:

- In wall elements, maximum tension, maximum moment and maximum shear do not occur simultaneously under one load case. Maximum tension will occur under one load case; maximum moment under another and maximum shear under yet another case. At various sections, therefore worst combinations of simultaneously occurring tension, moment and shear are to be considered in design.

- Also at a particular section of wall, maximum tension on inner face due to horizontal moment may occur under one load case, whereas tension on outer face of the same section may occur under different load case. This is to be considered in design.

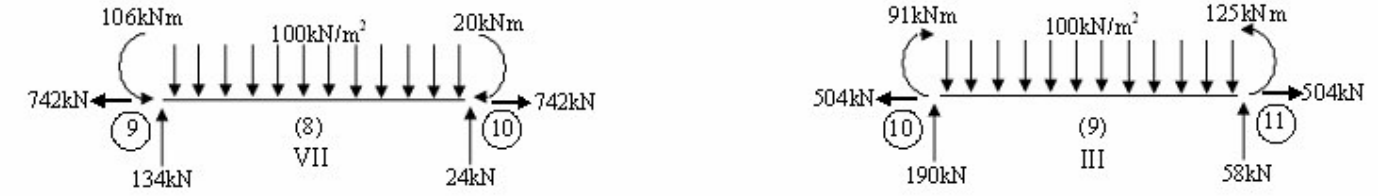
- Symmetrical elements of outer wall can be grouped. In each group worst Combination of forces of the worst affected element will be the basis of design for that group. This is required since under load cases 2 to 7, location of Empty/full compartment can be any.

TABLE: 6.7 GOVERNING CASES FOR OUTER WALL DESIGN

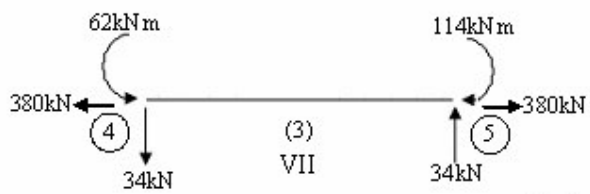
( ) denotes elements, ○ denotes nodes; II, III, VI, VII indicate load cases

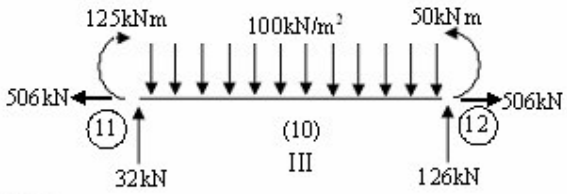
Nodes 2,8,14,20,26,32

Nodes 3,9,15,21,27,33

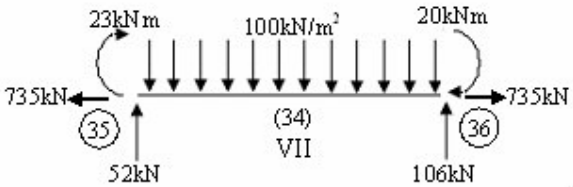


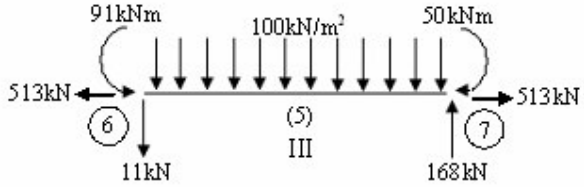
Nodes 4, 10, 16,22,28,34



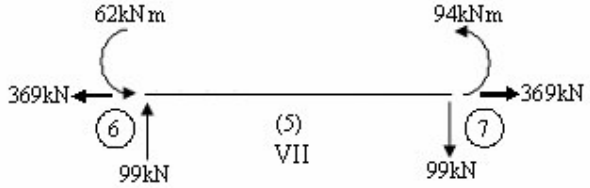


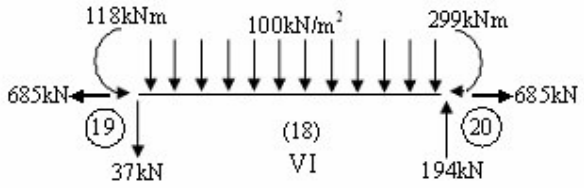
Nodes 5, 11, 17,23,29,35





Nodes 6, 12, 18,24,30,36





Nodes 7, 13, 19,25,31,37

6.4 DESIGN OF OUTER CIRCULAR WALL

In the following tables, design reinforcement is calculated at all heights for groups of elements of outer wall and for inner walls for governing cases of loading. After calculating Ku, %age steel can be provided [2].

Static pressure (Pz) $= \dfrac{WR(1-e^{-\mu\lambda Z/R})}{\mu}$ (kN/m^2)

Mu = 1.5 × M.F × Pdes (kNm)

Ku = Mu / (b × d^2) (N/mm^2)

At = $\dfrac{Pt \times b \times d}{100}$ (mm^2)

Permissible tensile stress in steel restricted to 1900 kg/cm^2 (0.19 kN/mm^2)

Axial (hoop) force = relevant M.F × Pdes kN

Hoop R/F = Hoop force/0.19

in mm^2

TABLE: 6.8 MULTIPLYING FACTOR FOR ALL GROUPS OF NODES FOR DIFFERENT CONDITION

Tables no.	Groups of nodes	Multiplying factors from table:6.7			
		Tension inner face		Tension outer face	
		Moment	Axial	Moment	Axial
6.9 (A)	2,8,14,20,26,32	1.72	4.72	1.50	3.46
6.9 (B)	3,9,15,21,27,33	1.06	7.54	0.94	3.69
6.9 (C)	4,10,16,22,28,34	0.2	7.42	0.91	5.04
6.9 (D)	5,11,17,23,29,35	1.14	3.80	1.25	5.06
6.9 (E)	6,12,18,24,30,36	0.2	7.35	0.91	5.13
6.9 (F)	7,13,19,25,31,37	1.18	6.85	0.94	3.69

6.4.1 SAMPLE CALCULATIONS (OUTERWALL)

Static pressure (Pz) $= \dfrac{WR(1-e^{-\mu\lambda Z/R})}{\mu}$

$$= \frac{15.5\times1.55\times(1-e^{-0.466\times0.406\times Z/1.55})}{0.466}$$

$$= 51.55\times(1-e^{-0.122\times Z})$$

$$= 51.55 \times [1-e^{-0.122\times41.7}]$$

$$= 51.08 \text{ kN/m}^2$$

Pdes = Pz × Cd = 51.08 × 2 = 102.15 kN/m^2

(Z = ht. of fill at cone = 60.2-18.5 = 41.7 m)

6.4.1.1 Element: 1 (Typical for nodes 2, 8,14,20,26 & 32), load Case: 7

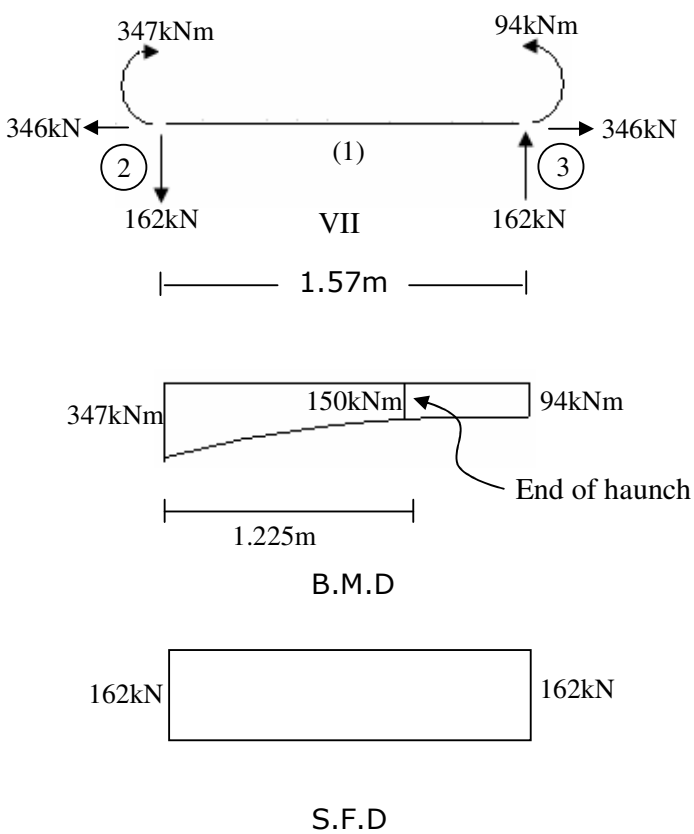


FIGURE: 6.6 DESIGN FORCES (OUTER WALL)

At 1.225m from node 2 (end of haunch), for 100 kN/m^2

$M_{1.225}$ = 347 – 161.25 × 1.225 = 150 kNm

Shear = 161.25 kN

	At node 2	At end of haunch
D (mm)	1450	450
d (mm)	1410	410
M (kNm)	=1.5 × 347 × 102.15/100 =531.7 (Tension outside)	=1.5 × 150 × 102.15/100 =229.84
Hoop or Axial force (kN)	=346 × 102.15/100 = 353.44	= 353.44
Ast for axial force (mm^2)	=353.44/0.19 = 1860.22	=1860.22
Shear (kN)	=161.25 × 102.15/100 =164.72	
Ku = Mu/b×d^2 (N/mm^2)	$= \frac{531.7 \times 10^6}{1000 \times 1410^2}$ = 0.267	$= \frac{229.84 \times 10^6}{1000 \times 410^2}$ = 1.367
Pt	= 0.085	= 0.409
Ast (mm^2)	= 1198.5	= 1677.2

Reinforcement (mm^2) required for load case: 7

	Inside	Outside
Moment	-	1677.2
Axial	930.11	930.11
Temperature	-	1690
Total	930.11	4297.29

6.4.1.2 Element: 6 (Typical for nodes 2, 8,14,20,26 & 32), load Case: 2

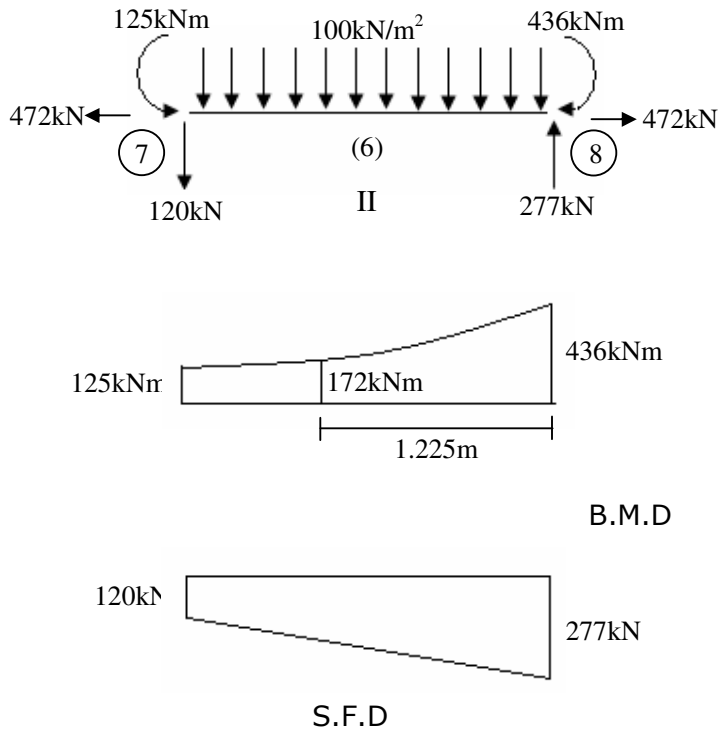


FIGURE: 6.7 DESIGN FORCES (OUTER WALL)

At 1.225m from node 2 (end of haunch), for 100 kN/m^2

$M_{1.225}$= 436-276.76 × 1.225 + 100 × 1.225^2/2 = 172 kNm

Shear = 276.76 – (100 × 1.225) = 154.26 kN

	At node 8	At end of haunch
D (mm)	1450	450
d (mm)	1410	410
M (kNm)	=1.5 × 436 × 02.15/100 =668.06 (Tension inside)	=1.5 × 172 × 102.15/100 =263.55
Hoop or Axial force (kN)	=472 × 102.15/100 =482.15	= 482.15
Ast for axial force (mm^2)	=482.15/0.19 = 2537.64	
Shear (kN)	=276.76 × 102.15/100 =282.71	=2537.64
Ku = Mu/b×d^2 (N/mm^2)	$= \frac{668.06 \times 10^6}{1000 \times 1410^2}$ = 0.336 = 0.094	$= \frac{263.55 \times 10^6}{1000 \times 410^2}$ = 1.568 = 0.472
Pt	=1325.4	
Ast (mm^2)		= 1935.2

Reinforcement (mm^2) required for case: 2

	Inside	Outside
Moment	1935.2	-
Axial	1268.82	1268.82
Temperature		1690
Total	3203.99	2958.82

TABLE: 6.9 [A] DESIGN REINFORCEMENTS FOR OUTER CIRCULAR WALL- GROUP: 1 (NODE NO: 2, 8, 14, 20, 26, 32)

					R/F INNER FACE(Load 2 governs)							R/F OUTER FACE(Load 7 governs)							Temp	TOTAL R/F			
Ht from top	Pz	Cd	Pdes	Mix	Moment				Axial			Moment				Axial			R/F	INNER FACE		OUTER FACE	
of fill																							
					Mu	Ku	Pt	At	Axial force	R/F (mm^2)		Mu	Ku	Pt	At	Axial force	R/F (mm^2)			Req[d]	Prov[d]	Req[d]	Prov[d]
m	kN/m^2		kN/m^2		kNm	N/mm^2	%	mm^2	kN	Total	Inner face	kNm	N/mm^2	%	mm^2	kN	Total	Outer face	mm^2	mm^2	mm^2	mm^2	mm^2
4	19.90	1.75	34.82	20	89.83	0.534	0.159	650.1	164.34	864.9	432.5	78.34	0.466	0.138	567.8	120.47	634.1	317.0	1300	1082.6	1331	2184.8	2337
8	32.09	1.75	56.16	25	144.88	0.862	0.256	1049.6	265.05	1395.0	697.5	126.35	0.752	0.223	914.3	194.30	1022.6	511.3	1500	1747.1	2277	2925.6	3079
12	39.56	1.9	75.17	25	193.93	1.154	0.345	1415.8	354.78	1867.3	933.6	169.12	1.006	0.300	1229.1	260.07	1368.8	684.4	1500	2349.4	2442	3413.5	3656
16	44.14	1.9	83.87	25	216.37	1.287	0.387	1587.5	395.84	2083.4	1041.7	188.70	1.123	0.336	1376.2	290.17	1527.2	763.6	1500	2629.2	2767	3639.8	3830
20	46.95	1.9	89.20	30	230.13	1.369	0.410	1679.4	421.01	2215.8	1107.9	200.69	1.194	0.356	1458.1	308.62	1624.3	812.2	1690	2787.3	3008	3960.3	4371
24	48.67	2	97.33	30	251.11	1.494	0.449	1839.4	459.40	2417.9	1208.9	218.99	1.303	0.389	1595.2	336.76	1772.4	886.2	1690	3048.4	3145	4171.5	4468
28	49.72	2	99.44	30	256.55	1.526	0.459	1881.2	469.35	2470.3	1235.1	223.74	1.331	0.398	1631.0	344.06	1810.8	905.4	1690	3116.3	3388	4226.4	4468
32	50.37	2	100.73	30	259.88	1.546	0.465	1906.9	475.45	2502.3	1251.2	226.64	1.348	0.403	1653.0	348.53	1834.3	917.2	1690	3158.0	3294	4260.1	4468
36	50.76	2	101.52	30	261.93	1.558	0.469	1922.6	479.18	2522.0	1261.0	228.42	1.359	0.406	1666.5	351.26	1848.8	924.4	1690	3183.6	3294	4280.8	4468
41.7	51.08	2	102.15	30	263.55	1.568	0.472	1935.2	482.15	2537.6	1268.8	229.84	1.367	0.409	1677.2	353.44	1860.2	930.1	1690	3204.0	3294	4297.3	4787
44	51.15	1	51.15	30	131.97	0.785	0.233	955.2	241.44	1270.7	635.4	115.09	0.685	0.204	834.4	176.99	931.5	465.8	1690	1590.6	1665	2990.2	3217
48	51.24	1	51.24	30	132.21	0.786	0.233	956.9	241.87	1273.0	636.5	115.30	0.686	0.204	835.9	177.30	933.2	466.6	1690	1593.4	1665	2992.5	3217

TABLE : 6.9 [B] DESIGN REINFORCEMENTS FOR OUTER CIRCULAR WALL-GROUP: 2 (NODE NO: 3, 9, 15, 21, 27, 33)

					R/F INNER FACE(Load 7 governs)							R/F OUTER FACE(Load 7 governs)							Temp	TOTAL R/F			
Ht from top	Pz	Cd	Pdes	Mix	Moment				Axial			Moment				Axial			R/F	INNER FACE		OUTER FACE	
of fill																							
					Mu	Ku	Pt	At	Axial force	R/F (mm^2)		Mu	Ku	Pt	At	Axial force	R/F (mm^2)			Req[d]	Prov[d]	Req[d]	Prov[d]
m	kN/m^2		kN/m^2		kNm	N/mm^2	%	mm^2	kN	Total	Inner	kNm	N/mm^2	%	mm^2	kN	Total	Outer	mm^2	mm^2	mm^2	mm^2	mm^2
											face							face					
4	19.90	1.75	34.82	20	55.36	0.329	0.099	405.3	262.53	1381.7	690.9	49.09	0.292	0.088	361.4	128.48	676.2	338.1	1300	1096.1	1331	1999.5	2337
8	32.09	1.75	56.16	25	89.29	0.531	0.158	648.3	423.41	2228.5	1114.2	79.18	0.471	0.141	576.8	207.21	1090.6	545.3	1500	1762.5	2277	2622.1	3079
12	39.56	1.9	75.17	25	119.51	0.711	0.211	864.7	566.75	2982.9	1491.4	105.98	0.630	0.187	767.3	277.36	1459.8	729.9	1500	2356.2	2442	2997.2	3656
16	44.14	1.9	83.87	25	133.35	0.793	0.235	965.2	632.34	3328.1	1664.1	118.25	0.703	0.209	855.6	309.46	1628.8	814.4	1500	2629.2	2767	3170.0	3830
20	46.95	1.9	89.20	30	141.82	0.844	0.250	1026.2	672.54	3539.7	1769.9	125.77	0.748	0.222	910.7	329.14	1732.3	866.1	1690	2796.0	3008	3466.8	4371
24	48.67	2	97.33	30	154.76	0.921	0.273	1119.9	733.87	3862.5	1931.2	137.24	0.816	0.242	993.1	359.15	1890.3	945.1	1690	3051.2	3145	3628.2	4468
28	49.72	2	99.44	30	158.11	0.941	0.279	1144.3	749.77	3946.1	1973.1	140.21	0.834	0.247	1014.5	366.93	1931.2	965.6	1690	3117.4	3388	3670.1	4468
32	50.37	2	100.73	30	160.16	0.953	0.283	1159.3	759.50	3997.4	1998.7	142.03	0.845	0.251	1027.7	371.69	1956.3	978.1	1690	3158.0	3294	3695.8	4468
36	50.76	2	101.52	30	161.42	0.960	0.285	1168.4	765.47	4028.8	2014.4	143.15	0.852	0.253	1035.7	374.61	1971.7	985.8	1690	3182.8	3294	3711.6	4468
41.7	51.08	2	102.15	30	162.42	0.966	0.287	1175.8	770.22	4053.8	2026.9	144.03	0.857	0.254	1042.1	376.94	1983.9	991.9	1690	3202.6	3294	3724.1	4787
44	51.15	1	51.15	30	81.33	0.484	0.145	595.9	385.69	2029.9	1015.0	72.12	0.429	0.130	531.6	188.75	993.4	496.7	1690	1610.9	1665	2718.3	3217
48	51.24	1	51.24	30	81.48	0.485	0.146	597.0	386.37	2033.5	1016.8	72.25	0.430	0.130	532.5	189.09	995.2	497.6	1690	1613.7	1665	2720.1	3217

TABLE: 6.9 [C] DESIGN REINFORCEMENTS FOR OUTER CIRCULAR WALL-GROUP: 3 (NODE NO: 4, 10, 16, 22, 28, 34)

					R/F INNER FACE(Load 7 governs)							R/F OUTER FACE(Load 3 governs)							Temp	TOTAL R/F			
Ht from top	Pz	Cd	Pdes	Mix	Moment				Axial			Moment				Axial			R/F	INNER FACE		OUTER FACE	
of fill																							
					Mu	Ku	Pt	At	Axial force	R/F (mm^2)		Mu	Ku	Pt	At	Axial force	R/F (mm^2)			Req'd	Prov'd	Req'd	Prov'd
m	kN/m^2		kN/m^2		kNm	N/mm^2	%	mm^2	kN	Total	Inner face	kNm	N/mm^2	%	mm^2	kN	Total	Outer face	mm^2	mm^2	mm^2	mm^2	mm^2
4	19.90	1.75	34.82	20	10.45	0.062	0.023	95.4	258.35	1359.7	679.9	47.53	0.283	0.085	350.5	175.48	923.6	461.8	1300	775.3	1331	2112.3	2337
8	32.09	1.75	56.16	25	16.85	0.100	0.035	145.0	416.67	2193.0	1096.5	76.65	0.456	0.136	559.0	283.02	1489.6	744.8	1500	1241.5	2277	2803.7	3079
12	39.56	1.9	75.17	25	22.55	0.134	0.045	183.9	557.73	2935.4	1467.7	102.60	0.610	0.181	743.1	378.83	1993.9	996.9	1500	1651.6	2442	3240.0	3656
16	44.14	1.9	83.87	25	25.16	0.150	0.049	201.7	622.28	3275.2	1637.6	114.48	0.681	0.202	828.4	422.68	2224.6	1112.3	1500	1839.3	2767	3440.7	3830
20	46.95	1.9	89.20	30	26.76	0.159	0.053	218.6	661.84	3483.4	1741.7	121.75	0.724	0.215	882.0	449.55	2366.1	1183.0	1690	1960.3	3008	3755.0	4371
24	48.67	2	97.33	30	29.20	0.174	0.057	235.3	722.19	3801.0	1900.5	132.86	0.790	0.235	961.6	490.55	2581.8	1290.9	1690	2135.8	3145	3942.5	4468
28	49.72	2	99.44	30	29.83	0.177	0.058	239.6	737.83	3883.3	1941.7	135.73	0.807	0.240	982.3	501.17	2637.7	1318.9	1690	2181.3	3388	3991.1	4468
32	50.37	2	100.73	30	30.22	0.180	0.059	242.2	747.42	3933.8	1966.9	137.50	0.818	0.243	995.0	507.68	2672.0	1336.0	1690	2209.1	3294	4021.0	4468
36	50.76	2	101.52	30	30.46	0.181	0.059	243.9	753.29	3964.7	1982.3	138.58	0.824	0.245	1002.8	511.67	2693.0	1346.5	1690	2226.2	3294	4039.2	4468
41.7	51.08	2	102.15	30	30.65	0.182	0.060	245.2	757.96	3989.3	1994.6	139.44	0.829	0.246	1008.9	514.84	2709.7	1354.8	1690	2239.8	3294	4053.8	4787
44	51.15	1	51.15	30	15.35	0.091	0.034	140.9	379.55	1997.6	998.8	69.82	0.415	0.126	515.5	257.81	1356.9	678.4	1690	1139.7	1665	2884.0	3217
48	51.24	1	51.24	30	15.37	0.091	0.034	141.1	380.22	2001.2	1000.6	69.95	0.416	0.126	516.4	258.26	1359.3	679.6	1690	1141.7	1665	2886.0	3217

TABLE: 6.9 [D] DESIGN REINFORCEMENTS FOR OUTER CIRCULAR WALL-GROUP: 4 (NODE NO: 5, 11, 17, 23, 29, 35)

					R/F INNER FACE(Load 7 governs)							R/F OUTER FACE(Load 3 governs)							Temp	TOTAL R/F			
Ht from top	Pz	Cd	Pdes	Mix	Moment				Axial			Moment				Axial			R/F	INNER FACE		OUTER FACE	
of fill																							
					Mu	Ku	Pt	At	Axial force	R/F (mm^2)		Mu	Ku	Pt	At	Axial force	R/F (mm^2)			Req[d]	Prov[d]	Req[d]	Prov[d]
m	kN/m^2		kN/m^2		kNm	N/mm^2	%	mm^2	kN	Total	Inner face	kNm	N/mm^2	%	mm^2	kN	Total	Outer face	mm^2	mm^2	mm^2	mm^2	mm^2
4	19.90	1.75	34.82	20	59.54	0.354	0.106	434.6	132.31	696.4	348.2	65.28	0.388	0.116	475.1	176.18	927.3	463.6	1300	782.8	957	2238.7	2337
8	32.09	1.75	56.16	25	96.03	0.571	0.170	696.2	213.39	1123.1	561.6	105.29	0.626	0.186	762.4	284.14	1495.5	747.7	1500	1257.7	1272	3010.1	3079
12	39.56	1.9	75.17	25	128.53	0.765	0.227	930.1	285.63	1503.3	751.7	140.93	0.838	0.249	1020.6	380.34	2001.8	1000.9	1500	1681.8	1728	3521.5	3656
16	44.14	1.9	83.87	25	143.41	0.853	0.253	1038.8	318.69	1677.3	838.7	157.25	0.935	0.278	1140.8	424.36	2233.5	1116.7	1500	1877.4	1963	3757.5	3830
20	46.95	1.9	89.20	30	152.53	0.907	0.269	1103.7	338.95	1783.9	892.0	167.24	0.995	0.295	1211.0	451.34	2375.5	1187.7	1690	1995.7	2134	4088.7	4371
24	48.67	2	97.33	30	166.44	0.990	0.294	1205.1	369.86	1946.6	973.3	182.50	1.086	0.323	1323.1	492.49	2592.1	1296.0	1690	2178.4	2231	4309.2	4468
28	49.72	2	99.44	30	170.04	1.012	0.300	1231.5	377.87	1988.8	994.4	186.45	1.109	0.330	1352.3	503.16	2648.2	1324.1	1690	2225.9	2231	4366.4	4468
32	50.37	2	100.73	30	172.25	1.025	0.304	1247.7	382.77	2014.6	1007.3	188.87	1.124	0.334	1370.2	509.69	2682.6	1341.3	1690	2255.0	2337	4401.5	4468
36	50.76	2	101.52	30	173.60	1.033	0.307	1257.6	385.78	2030.4	1015.2	190.35	1.132	0.337	1381.2	513.70	2703.7	1351.8	1690	2272.9	2337	4423.1	4468
41.7	51.08	2	102.15	30	174.68	1.039	0.309	1265.5	388.17	2043.0	1021.5	191.53	1.139	0.339	1390.0	516.88	2720.4	1360.2	1690	2287.1	2337	4440.2	4787
44	51.15	1	51.15	30	87.47	0.520	0.156	639.0	194.38	1023.0	511.5	95.91	0.571	0.170	698.4	258.83	1362.3	681.1	1690	1150.5	1272	3069.6	3217
48	51.24	1	51.24	30	87.63	0.521	0.156	640.1	194.72	1024.9	512.4	96.08	0.572	0.171	699.6	259.29	1364.7	682.3	1690	1152.5	1272	3072.0	3217

TABLE: 6.9 [E] DESIGN REINFORCEMENTS FOR OUTER CIRCULAR WALL-GROUP: 5 (NODE NO: 6, 12, 18, 24, 30, 36)

					R/F INNER FACE(Load 7 governs)							R/F OUTER FACE(Load 3 governs)							Temp	TOTAL R/F			
Ht from top	Pz	Cd	Pdes	Mix	Moment				Axial			Moment				Axial			R/F	INNER FACE		OUTER FACE	
of fill																							
					Mu	Ku	Pt	At	Axial force	R/F (mm^2)		Mu	Ku	Pt	At	Axial force	R/F (mm^2)			Req[d]	Prov[d]	Req[d]	Prov[d]
m	kN/m^2		kN/m^2		kNm	N/mm^2	%	mm^2	kN	Total	Inner face	kNm	N/mm^2	%	mm^2	kN	Total	Outer face	mm^2	mm^2	mm^2	mm^2	mm^2
4	19.90	1.75	34.82	20	10.45	0.062	0.023	95.40	255.91	1346.9	673.5	47.53	0.283	0.085	350.5	178.62	940.1	470.0	1300	768.9	1331	2120.5	2337
8	32.09	1.75	56.16	25	16.85	0.100	0.035	145.02	412.74	2172.3	1086.2	76.65	0.456	0.136	559.0	288.08	1516.2	758.1	1500	1231.2	2277	2817.0	3079
12	39.56	1.9	75.17	25	22.55	0.134	0.045	183.88	552.46	2907.7	1453.9	102.60	0.610	0.181	743.1	385.60	2029.5	1014.7	1500	1637.7	2442	3257.8	3656
16	44.14	1.9	83.87	25	25.16	0.150	0.049	201.71	616.41	3244.3	1622.1	114.48	0.681	0.202	828.4	430.23	2264.4	1132.2	1500	1823.8	2767	3460.6	3830
20	46.95	1.9	89.20	30	26.76	0.159	0.053	218.61	655.60	3450.5	1725.3	121.75	0.724	0.215	882.0	457.58	2408.3	1204.2	1690	1943.9	3008	3776.1	4371
24	48.67	2	97.33	30	29.20	0.174	0.057	235.27	715.38	3765.2	1882.6	132.86	0.790	0.235	961.6	499.31	2627.9	1314.0	1690	2117.9	3145	3965.5	4468
28	49.72	2	99.44	30	29.83	0.177	0.058	239.60	730.87	3846.7	1923.3	135.73	0.807	0.240	982.3	510.12	2684.8	1342.4	1690	2162.9	3388	4014.7	4468
32	50.37	2	100.73	30	30.22	0.180	0.059	242.24	740.37	3896.7	1948.3	137.50	0.818	0.243	995.0	516.75	2719.7	1359.9	1690	2190.6	3294	4044.8	4468
36	50.76	2	101.52	30	30.46	0.181	0.059	243.87	746.18	3927.3	1963.6	138.58	0.824	0.245	1002.8	520.81	2741.1	1370.5	1690	2207.5	3294	4063.3	4468
41.7	51.08	2	102.15	30	30.65	0.182	0.060	245.16	750.81	3951.6	1975.8	139.44	0.829	0.246	1008.9	524.03	2758.1	1379.0	1690	2221.0	3294	4078.0	4787
44	51.15	1	51.15	30	15.35	0.091	0.034	140.92	375.97	1978.8	989.4	69.82	0.415	0.126	515.5	262.41	1381.1	690.6	1690	1130.3	1665	2896.1	3217
48	51.24	1	51.24	30	15.37	0.091	0.034	141.10	376.64	1982.3	991.1	69.95	0.416	0.126	516.4	262.88	1383.6	691.8	1690	1132.2	1665	2898.2	3217

TABLE: 6.9 [F] DESIGN REINFORCEMENTS FOR OUTER CIRCULAR WALL-GROUP: 6 (NODE NO: 7, 13, 19, 25, 31, 37)

					R/F INNER FACE(Load 6 governs)							R/F OUTER FACE(Load 7 governs)							Temp	TOTAL R/F			
Ht from top	Pz	Cd	Pdes	Mix	Moment				Axial			Moment				Axial			R/F	INNER FACE		OUTER FACE	
of fill																							
					Mu	Ku	Pt	At	Axial force	R/F (mm^2)		Mu	Ku	Pt	At	Axial force	R/F (mm^2)			Req'd	Prov'd	Req'd	Prov'd
m	kN/m^2		kN/m^2		kNm	N/mm^2	%	mm^2	kN	Total	Inner face	kNm	N/mm^2	%	mm^2	kN	Total	Outer face	mm^2	mm^2	mm^2	mm^2	mm^2
4	19.90	1.75	34.82	20	61.63	0.367	0.110	449.3	238.50	1255.3	627.6	49.09	0.292	0.088	361.4	128.48	676.2	338.1	1300	1077.0	1331	1999.5	2337
8	32.09	1.75	56.16	25	99.39	0.591	0.176	720.2	384.66	2024.5	1012.3	79.18	0.471	0.141	576.8	207.21	1090.6	545.3	1500	1732.5	2277	2622.1	3079
12	39.56	1.9	75.17	25	133.04	0.791	0.235	963.0	514.88	2709.9	1355.0	105.98	0.630	0.187	767.3	277.36	1459.8	729.9	1500	2317.9	2442	2997.2	3656
16	44.14	1.9	83.87	25	148.44	0.883	0.262	1075.8	574.48	3023.6	1511.8	118.25	0.703	0.209	855.6	309.46	1628.8	814.4	1500	2587.6	2767	3170.0	3830
20	46.95	1.9	89.20	30	157.88	0.939	0.279	1142.6	611.00	3215.8	1607.9	125.77	0.748	0.222	910.7	329.14	1732.3	866.1	1690	2750.5	3008	3466.8	4371
24	48.67	2	97.33	30	172.28	1.025	0.304	1247.9	666.72	3509.0	1754.5	137.24	0.816	0.242	993.1	359.15	1890.3	945.1	1690	3002.4	3145	3628.2	4468
28	49.72	2	99.44	30	176.01	1.047	0.311	1275.3	681.15	3585.0	1792.5	140.21	0.834	0.247	1014.5	366.93	1931.2	965.6	1690	3067.8	3388	3670.1	4468
32	50.37	2	100.73	30	178.29	1.061	0.315	1292.1	690.00	3631.6	1815.8	142.03	0.845	0.251	1027.7	371.69	1956.3	978.1	1690	3107.9	3294	3695.8	4468
36	50.76	2	101.52	30	179.69	1.069	0.318	1302.5	695.42	3660.1	1830.1	143.15	0.852	0.253	1035.7	374.61	1971.7	985.8	1690	3132.5	3294	3711.6	4468
41.7	51.08	2	102.15	30	180.81	1.076	0.320	1310.7	699.73	3682.8	1841.4	144.03	0.857	0.254	1042.1	376.94	1983.9	991.9	1690	3152.1	3294	3724.1	4787
44	51.15	1	51.15	30	90.54	0.539	0.161	660.6	350.39	1844.2	922.1	72.12	0.429	0.130	531.6	188.75	993.4	496.7	1690	1582.7	1665	2718.3	3217
48	51.24	1	51.24	30	90.70	0.540	0.161	661.7	351.01	1847.4	923.7	72.25	0.430	0.130	532.5	189.09	995.2	497.6	1690	1585.4	1665	2720.1	3217

6.4.2 CHECK FOR CRACK WIDTH

Based on design parameters outlined in IS 4995 (Part 1&2)-1974 and ACI 318R-83, crack width is calculated at outer face of wall. The maximum horizontal moments occure on outer face at nodes 2, 8,14,20,26 & 32 (junction of cross walls) under load case 7 of five compartment full. Also the maximum design pressure in the wall is at the cone apex level, which is 41.7m below top of fill. The section is to be checked is at the face of haunch, where wall thickness is 45 cm.

At this level and section:

Hoop force for load case-7 at cone apex level = 353.44kN/m

TABLE: 6.10 REINFORCEMENT DETAIL AT CONE APEX LVL

	Reinforcement (cm2) for			
	Hoop force	**Moment**	**Tempture**	**Total**
Outer face	9.30	16.77	16.9	42.97
Inner face	12.69	19.35	-	32.04

[A] Crack width for direct tension hoop force [6]

$$Wcr = 10^{-6}(4+ \rho\ \Phi' / P_0)\ \sigma_{sa}\ \{1 - (6/ P_0\ \sigma_{sa})^2\} \qquad \text{..... (6.10)}$$

For resisting hoop force, steel available = 42.97+32.04-16.78-16.9

= 41.35 cm^2

σ_{sa} = 353.44 × 10^2 / 41.35 = 854.65 kg/cm^2

Act = Concrete area in tension for flexure member

= 0.5 (Sectional area below neutral axis)

Neutral axis below extreme compression fiber is at level [4] = 0.479d

Act = 0.5 × (1-0.479) × 41 × 100

= 1088.05 cm^2

P_0 = 41.35/1088.05

= 0.038

Φ = 3.2 cm

$Wcr = 10^{-6} \times [4+0.05\times3.2/0.038] \times 854.65 \times \{1-(6/0.038\times854.65)^2\}$

$= 0.0069$ cm < 0.02 cm

It is less than recommended value given in code [6], hence safe.

[B] Crack width for moment [8]

$$W = 0.076 \times \beta \times fs \times \sqrt[3]{d_c A} \qquad \text{..... (6.11)}$$

$\beta = 1.2$

dc = 4.6cm = 1.81 in

Wall height = 100 cm = 39.36in

Assume stress in steel at service load =

$fs = 0.87 \times 415 /1.5 = 240.7 N/mm^2 = 36ksi$ [4]

($415\ N/mm^2$ is yield stress of steel and 1.5 is load factor)

Area of 32mm dia. (largest) bar = $8.04cm^2$

No of bars in 1 m height = 42.97/8.04 = 5.34

$A = (2 \times 1.81 \times 39.36)/7.7 = 26.67in^2$

$w = 0.076 \times 1.2 \times 36 \times \sqrt[3]{1.81\times26.67} = 11.79$ in

w = 11.79 in 0.001"unit = 0.011" = 0.0279 cm

Total crack width = 0.0069 + 0.0279 = 0.03 cm

This is greater than the recommended value of 0.02 cm for silo wall. The alternative is to increase wall thickness to reduce flexural crack width. This is however not considered necessary, since a load case of five compartments full to the top and one compartment absolutely empty is a very remote possibility. The calculated crack width for such extreme case may be considered acceptable. Also the crack width size is only an assessment, based on numbers of assumption and empirical formulae. Till definite recommendations are formed on design method and limitations on crack codal width of silo wall under direct tension and bending, the engineering experience and discretion will have to decide the acceptance limit.

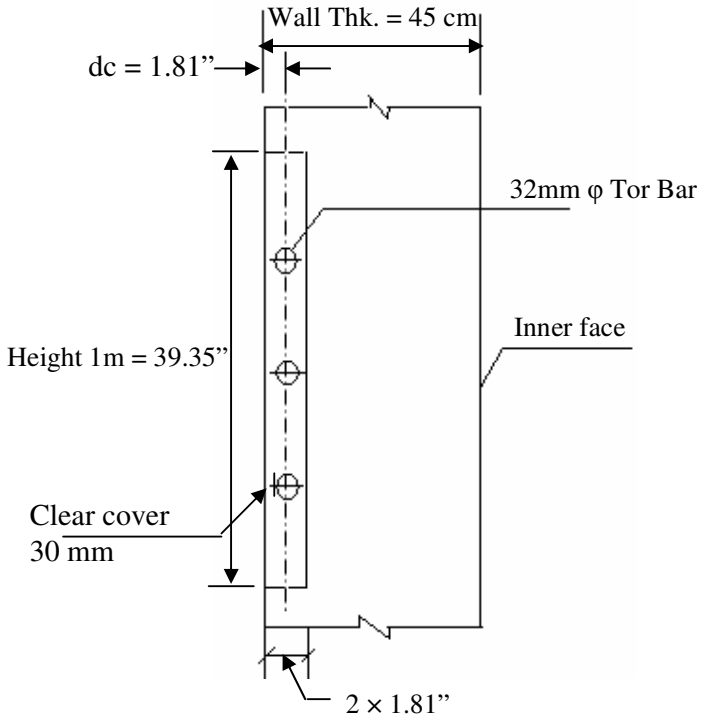
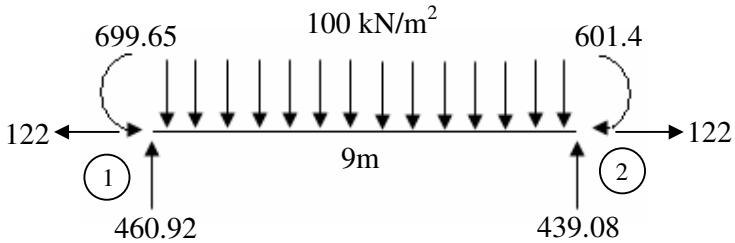


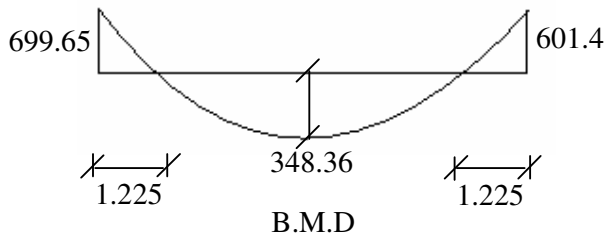
FIGURE: 6.8 CRACK WIDTH (ACI APPROACH)

6.5 DESIGN OF COMPARTMENT WALL

6.5.1 SAMPLE CALCULATION



Case: 3



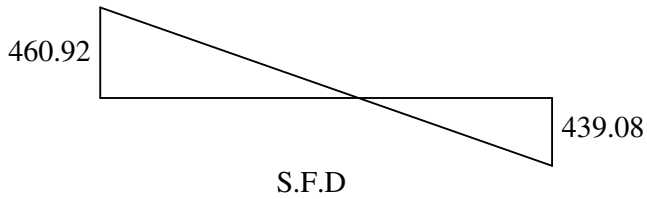


FIGURE: 6.9 GOVERNIND CASE FOR COMPARTMENT WALL DESIGN

Static pressure (Pz) $= \dfrac{WR(1-e^{-\mu\lambda Z/R})}{\mu}$

$$= \frac{15.5\times1.55\times(1-e^{-0.466\times0.406\times Z/1.55})}{0.466}$$

$= 51.55 \times [1-e^{-0.122\times Z}]$

$= 51.55 \times [1-e^{-0.122\times41.7}]$

$= 51.08$ kN/m^2

Pdes = Pz × Cd = 51.08 × 2 = 102.15 kN/m^2

(Z = ht. of fill at cone = 60.2-18.5 = 41.7 m)

At node 2,

D = 1450mm; d = 1410mm

M = 601.4 × 102.15 × 1.5/100 = 921.49 kNm

Ku = $Mu/b\times d^2 = 921.49\times10^6/(1000\times1410^2) = 0.463$ N/mm^2

Pt = 0.131

Ast = 1847.1mm^2

At the end of haunch,

D = 450mm; d = 410mm

$M_{1.225} = (601.4 - 439.08\times1.225 + 100\times1.225^2/2)\times1.5 = 207.84$ kNm

So, moment at 41.7m level

$M_{1.225} = 207.84 \times 102.15/100 = 211.45$ kNm

Ku = $Mu/b\times d^2 = 211.45\times10^6/(1000\times410^2) = 1.258$ N/mm^2

Pt = 0.375

Ast = 1538.56 mm^2

Reinforcement at the end of haunch is critical and it is continued in haunch area also.

Span moment at center = $700-(460.92\times4.5) + (100\times4.5^2/2)$

= $-361.64 \times102.15\times 1.5/100$

= 553.15 kNm

Ku = $553.15\times10^6/(1000\times410^2) = 3.29$ N/mm^2

Pt = 1.076

Ast = 4409.91 mm^2

Reinforcement (mm^2) at node 2 (end of haunch)

	Compartment full face	Compartment empty face
Moment	1538.56	-
Axial (=122×102.15/19)	327.5	327.5
Temperature	-	1690
Total	1866.06	2017.5

Reinforcement (mm^2) in span between nodes 1 & 2

	Compartment full face	Compartment empty face
Moment	-	4409.91
Axial	327.5	327.5
(Same as above)		
Temperature	-	1690
Total	327.5	6427.41

TABLE: 6.11 DESIGN REINFORCEMENTS FOR COMPARTMENT WALL

Ht from	Pz	Cd	Pd	Mix	Axial			Moment R/F end of haunch				Moment in span				Temp	R/F at end of haunch		R/F in span	
top of fill						R/F (mm^2)										R/F	Mom+Axial		Mom+Axial+Temp R/F	
			Cd×Pz		Axial Force	Axial force/0.19		Mu	Ku	Pt	At	Mu	Ku	Pt	At		mm^2		mm^2	
m	kN/m^2		kN/m^2		kN	Total	On face	kNm	N/mm^2	%	mm^2	kNm	N/mm^2	%	mm^2	(mm^2)	Req^d	Prov^d	Req^d	Prov^d
4	19.90	1.75	34.82	20	42.48	223.57	111.78	72.07	0.429	0.128	523.18	188.54	1.122	0.339	1389.93	1300	635	785	2802	3239
8	32.09	1.75	56.16	25	68.51	360.57	180.29	116.24	0.691	0.205	841.11	304.08	1.809	0.558	2286.46	1500	1021	1028	3967	4273
12	39.56	1.9	75.17	25	91.70	482.64	241.32	155.59	0.926	0.275	1128.56	407.02	2.421	0.774	3173.97	1500	1370	1436	4915	4942
16	44.14	1.9	83.87	25	102.32	538.50	269.25	173.60	1.033	0.308	1262.58	454.13	2.702	0.880	3609.95	1500	1532	1828	5379	5604
20	46.95	1.9	89.20	30	108.82	572.74	286.37	184.64	1.098	0.327	1338.94	483.00	2.873	0.917	3759.88	1690	1625	1675	5736	5766
24	48.67	2	97.33	30	118.74	624.97	312.48	201.47	1.199	0.357	1463.94	527.05	3.135	1.015	4163.44	1690	1776	1828	6166	6290
28	49.72	2	99.44	30	121.31	638.50	319.25	205.84	1.224	0.365	1496.52	538.46	3.203	1.042	4270.51	1690	1816	1828	6280	6290
32	50.37	2	100.73	30	122.89	646.79	323.40	208.51	1.240	0.370	1516.52	545.45	3.245	1.058	4336.66	1690	1840	2094	6350	6425
36	50.76	2	101.52	30	123.86	651.88	325.94	210.15	1.250	0.373	1528.79	549.74	3.270	1.068	4377.40	1690	1855	1963	6393	6425
41.7	51.08	2	102.15	30	124.62	655.92	327.96	211.45	1.258	0.375	1538.56	553.15	3.291	1.076	4409.91	1690	1867	1963	6428	6990
44	51.15	1	51.15	30	62.41	328.45	164.22	105.88	0.630	0.188	768.99	276.99	1.648	0.497	2039.46	1690	933	1028	3894	4273
48	51.24	1	51.24	30	62.52	329.03	164.52	106.07	0.631	0.188	770.32	277.48	1.651	0.498	2043.31	1690	935	1028	3898	4273

6.6 DESIGN OF CONE OF 6-COMPARTMENT SILO

6.6.1 ANALYSIS OF INTERNAL FORCES

Internal forces in conical shell are obtained by three dimensional finite element analyses using quadrilateral shell elements. STAAD.Pro programme is used for the purpose of assembling the elements and calculations of displacements and forces.

The shell is discretized into 324 nodes and 288 elements, with all elements being four nodded, i.e. taking shell as a frustum of a cone. The nodes and numbering for this idealization is shown in figure: 6.7

All bottom nodes 1 to 36 are assumed pinned for the analysis.

6.6.2 DIMENSION OF THE STRUCTURE

The diameter of the conical shell is 15.972 m and height 13.83 m to apex, making a 60° cone. For the analysis, thickness of the shell is uniform 750mm and that of compartment walls 450mm.

6.6.3 MATERIAL PROPERTIES

Grade of concrete = M25

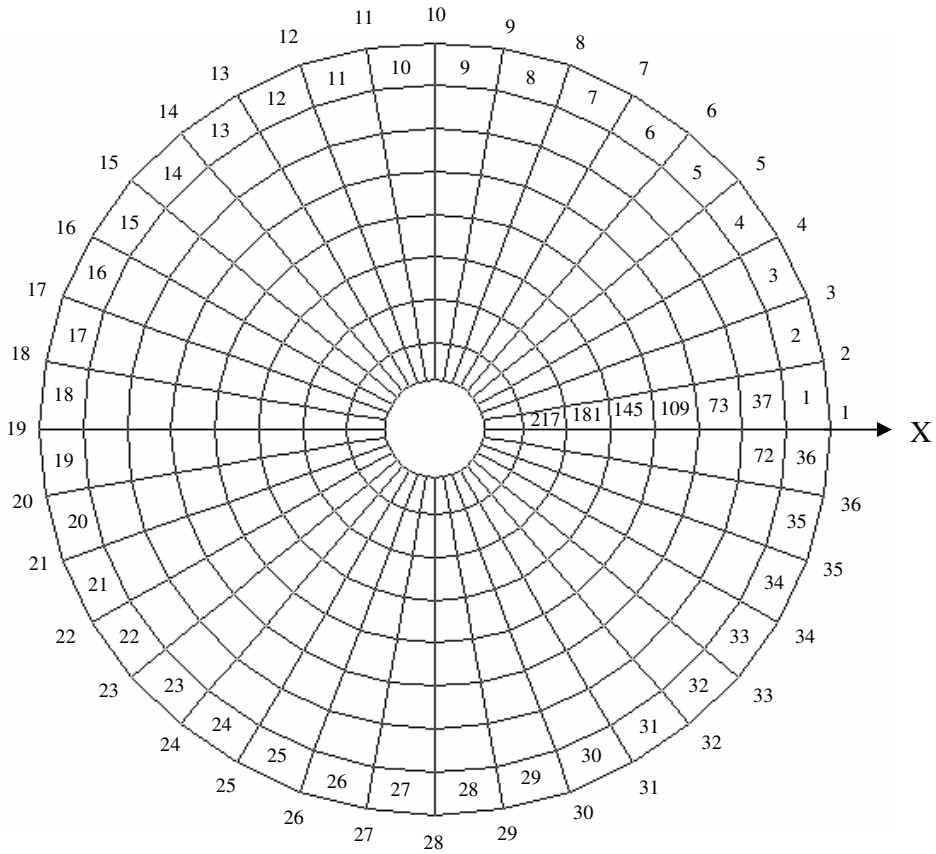
Young's modulus of concrete = 28500 N/mm^2 = 28500000 kN/m^2

Poisson's ratio = 0.15

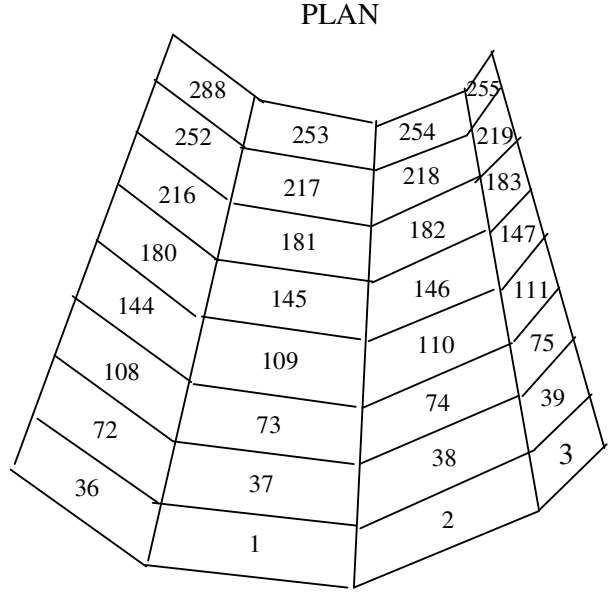
Density of fill material (cement) = 16 kN/m^3 (for stress calculation)

Volume density of fill material (cement) = 13 kN/m^3 (for volume calculation)

Density of concrete = 25 kN/m^3



PLAN



ELEVATION

FIGURE: 6.10 CONE MODEL WITH NODE AND PLATE NUMBER

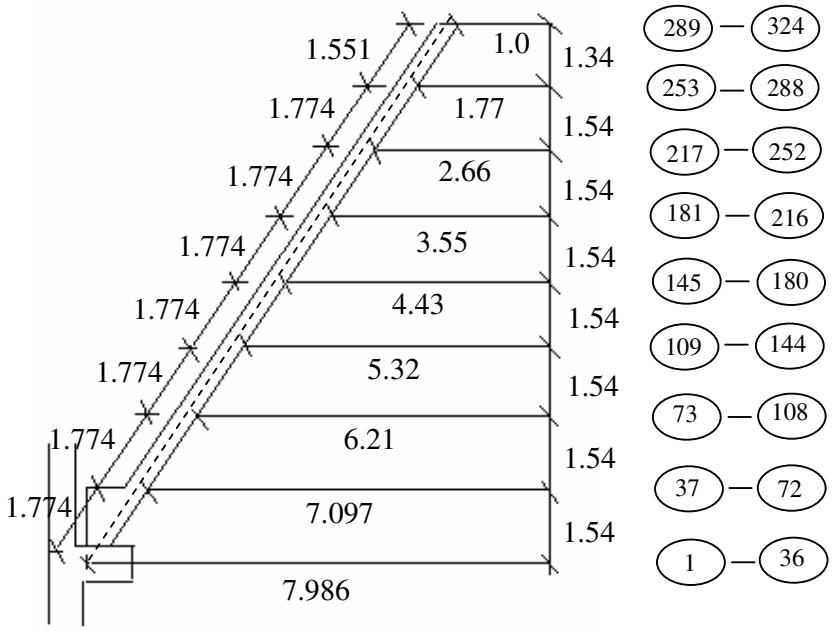


FIGURE: 6.11 CONE DESCRITISATION ALONG HEIGHT

6.6.4 LOAD CASES

1) All compartments full
2) Only one compartment full
3) Two compartments full
4) Alternate two compartments full
5) Alternate three compartments full
6) Four compartments full
7) Five compartments full

The above load cases are considered with two criteria:

Case 1: Considering friction of fill material on walls

Case 2: Not considering wall friction

6.6.5 LOADS FOR COMPUTER INPUT DATA

Loads on conical shell are:

- Self-weight of the compartment walls supported on shell. Silo roof load on compartment walls also should be taken. However, this is not taken in this analysis being small and is compensated by assumed higher load of cement fill.
- Janssen pressure assumes that part of vertical load is carried by walls as friction between granular fill material (cement in this case) and walls. This load is transferred to conical shell through compartment walls.
- Vertical load due to material fills (cement).
- Self-weight of shell. This is built in the computer programme and need not be given separately.

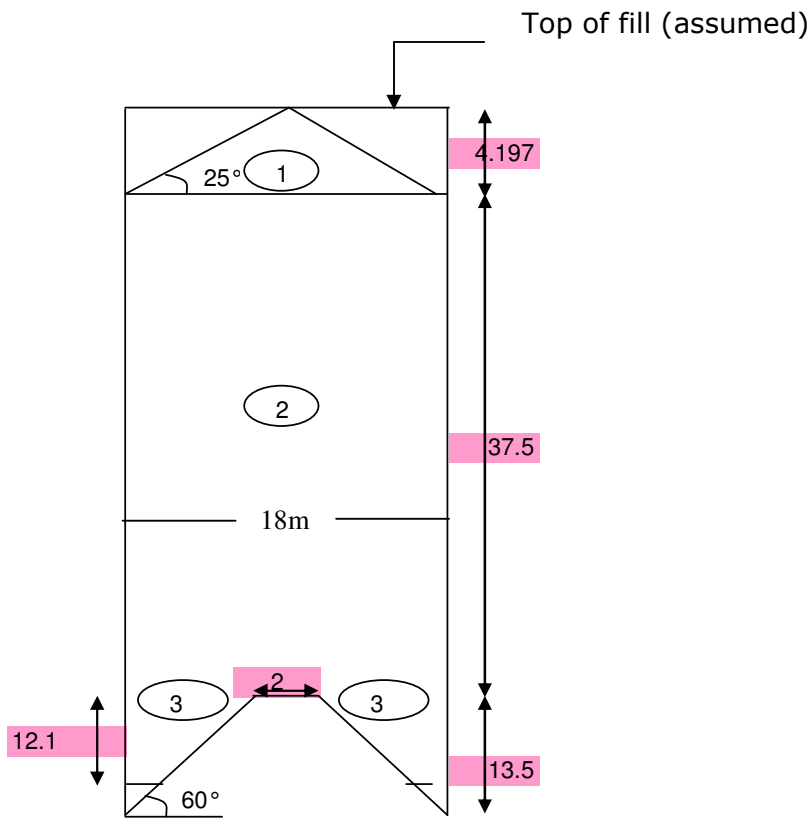


FIGURE: 6.12 STOREGE LEVEL OF SILO

6.6.6 LOAD CALCULATIONS

6.6.6.1 Self weight of compartment walls

Average height of each wall = (53.80 + 41.7)/2 = 47.75 m

Weight of six walls = 6 × 8 × 47.75 × 0.45 × 24 = 24753.6 kN

Self weight of wall / node = 24753.6/54 = 458.4 kN

6.6.6.2 Self weight of central shaft

Self weight of central shaft = Π × 1 × 1 × 41.7 × 25 = 3275 kN

Self weight of central shaft / node = 3275 / 36 = 91.0 kN

6.6.6.3 Friction load

Friction Load $Pw = W \times R \times (1 - e^{-(\mu\lambda z/R)})$

Where $\Phi = 25°$

$\mu = 0.467$

$\lambda = 0.406$

$P = 27.42$

$A = 42.39$

$R = 1.55$

Silo capacity:

Volume of Section 1 = Π × 9 × 9 × 4.197/3 = 356 m^3

Capacity of section 1 = 356 × 13 = 4628 kN

Volume of Section 2 = (Π × 9 × 9 × 37.5) - (Π × 1 × 1 × 37.5) = 9425 m^3

Capacity of section 2 = 9425 × 13 = 122522.11 kN

Volume of Section 3 = $(\Pi \times 9 \times 9 \times 12.1) - (\Pi \times \tan 60 \times (7.986^3 - 1^3))/3$

= 2157 m^3

Capacity of section 3 = 2157 × 13 = 28042.45 kN

Total Volume = 11938 m^3

Total Capacity= 155193 kN

Assuming uniform height, equivalent fill height = Total volume/Area

= 11938 / (Π × 9 × 9)

= 46.91 m

$Pw = W \times R \times (1 - e^{-(\mu\lambda z/R)})$ ..... (6.12)

$= 1300 \times 1.55 \times (1 - e^{-(0.466 \times 0.406 \times Z/1.55)})$

$= 2015\ (1-e^{-0.123 \times Z})$ kN/m^2

$$\int Pwdz = \left[\frac{2015 \times 46.91}{0.123} + 2015(e^{-0.123 \times 46.91} - e^{0})\right]$$

= 94523.65 + (-16329.35)

= 78194 kg/m^2

= 781.94 kN/m^2

Friction load on outer circular wall = (2 × Π × r) × Pw

= 2 × Π × 9 × 781.94

= 44218 kN

Friction load on cone surface from faces of internal walls

= no. of comp. wall × r × Pw

= 12 × 8 × 781.94

= 75067 kN

6.6.6.4 Cement fill load

Fill load = 150000 kN

Less friction on compartment walls = 44218 kN

Less friction on outer wall = 75067 kN

Net fill load on cone = 30716 kN

Distribution of loads on nodes:

Sample calculation:

For Elements 1 – 36 (bottom layer) $= \frac{1}{36}\left[2\pi \times (7.99 + 7.1)/2\right] \times 1.774$ = 2.34 m^2

TABLE: 6.12 AREAS AT DIFFERENT LEVEL ALONG HEIGHT

Elements	Area (m^2)
1 to 36	2.340
37 to 72	2.10
73 to 108	1.785
109 to 144	1.511
145 to 180	1.237
181 to 216	0.961
217 to 252	0.686
253 to 288	0.375
Total	10.996 m^2

Total area = 36 × 10.995

= 395.84 m^2

Check, Total Shell area = Π × 13.969 × (7.986+1)

= 394.23 m^2

Fill loads on nodes when wall friction is considered:

Sample calculation:

For Elements 1 – 36 (bottom layer):

Area contribution = 2.34/2 = 1.17 m^2

% of total = 1.17/10.995 = 11%

Fill node load = (30716 × 0.11)/36 = 93.86 kN

TABLE: 6.13 FILL NODE LOAD WITH WALL FRICTION CASE

Nodes	Area (m^2)	% of total	Fill node load (kN)
1 to 36	1.17	11	93.86
37 to 72	2.22	20	170.65
73 to 108	1.94	18	153.58
109 to 144	1.65	15	127.99
145 to 180	1.37	12	102.39
181 to 216	1.10	10	85.33
217 to 252	0.82	7	59.73
253 to 288	0.53	5	42.67
289 to 324	0.19	2	17.07
Total	10.996 m^2	100	

For no wall friction case, material fill load is 150000 kN

So, multiply fill loads of friction case by 150000/30716 = 4.88

TABLE: 6.14 FILL NODE LOADS WITH NO WALL FRICTION CASE

Nodes	Fill load (kN)
1 to 36	458.1
37 to 72	832.77
73 to 108	749.47
109 to 144	624.59
145 to 180	499.66
181 to 216	416.41
217 to 252	291.48
253 to 288	208.23
289 to 324	83.30

TABLE: 6.15 NODE LOADS WITH BOTH CASES

No Wall Friction Loads (kN)				Wall Friction Loads		
Nodes	Fill	Wall Self-wt.	Total	Fill	Friction loads	Total
1,7,13,19,25,31	458.1	458.4	916.5	93.87	1390.1	1942.37
37,43,49,55,61,67	832.97	458.4	1291.37	170.67	1390.1	2019.17
73,79,85,91,97,103	749.57	458.4	1207.97	153.6	1390.1	2002.1
109,115,121,127,133,139	624.64	458.4	1083.04	128	1390.1	1976.5
145,151,157,163,169,175	499.71	458.4	958.11	102.4	1390.1	1950.9
181,187,193,199,205,211	416.41	458.4	874.81	85.33	1390.1	1933.83
217,223,229,235,241,247	291.48	458.4	749.88	59.73	1390.1	1908.23
253,259,265,271,277,283	208.23	458.4	666.63	42.67	1390.1	1891.17
289,295,301,307,313,319	83.30	458.4	541.7	17.07	1390.1	1865.57
2 - 6, 8 -12, 14 -18, 20 -24,26 - 30, 32 - 36	458.1		458.1	93.87		93.87
38 - 42, 44 - 48,50- 54, 56 - 60, 62 -66, 68 - 72	832.97		832.97	170.67		170.67
74 - 78, 80 - 84, 86 - 90, 92 - 96, 98 -102,104 - 108	749.57		749.57	153.6		153.6
110 - 114,116 - 120, 122 -126,128 – 132, 134 – 138,140 – 144	624.64		624.64	128		128
146 – 150, 152 – 156, 158 – 162,164 – 168, 170 – 174,176 – 180	499.71		499.71	102.4		102.4
182 – 186, 188 – 192, 194 – 198, 200 – 204, 206 – 210, 212 – 216	416.41		416.41	85.33		85.33
218 – 222, 224 – 228, 230 – 234,236 – 240, 242 – 246, 248 – 252	291.48		291.48	59.73		59.73
254 – 258,260 – 264, 266 – 270,272 – 276, 278 – 282,284 – 288	208.23		208.23	42.67		42.67
290 – 294,296 – 300, 302 – 306, 308 – 312, 314 – 318,320 - 324.	83.30		83.30	17.07		17.07

On nodes 289 – 324 layer, loads of central shaft is acting = 90.97 kN/node.

TABLE: 6.16 CHECK ON TOTAL LOAD (KN) OF TABLE 6.15

	Wall Friction	No wall Friction
Wall self	54 × 458.4 = 24753.6	24753.6
Wall Friction (Comp. walls)	54 × 1390.1 = 75065.4	Nil
Fill	30720	4.88 × 30720 = 149913.6
Central shaft	3275.11	3275.11
Total (kN)	133814.11	177942.31

The difference of 44128 kN between two cases is because of friction in outer circular wall. In the Wall friction case, this load is directly on wall below.

6.6.7 Design Reinforcement

Conical shell elements are in compression in meridional and circumferential direction, causing thereby principal stresses and shears. The failure envelop based on experimental investigations by kupfer can be adopted. For devising the safe design criteria for different stress combinations, the same envelope is assumed to be valid for the working state.

For COMPRESSION – COMPRESSION state,

$F = (\sigma_1+\sigma_2)^2 / (\sigma_1+3.65\sigma_2) < \sigma_0$ ……. (6.13)

Where, F = Reference stress

σ_1 & σ_2 = principal stresses in direction 1 and 2 respectively

σ_0 = Uniaxial permissible stress in concrete

From STAAD.Pro. Output of cone model, maximum values of Principal stresses are

σ_1 = -6.41 N/mm^2

σ_2 = -2 N/mm^2

for M25 grade concrete[3]

σ_0 = 6 N/mm^2

f t = Permissible tensile stress in concrete = 3.2 N/mm^2

f c = Permissible stress in concrete in direct compression for static + wind load

= 1.33 × 6 = 7.98 N/m

So, Reference stress,

$F = (\sigma_1+\sigma_2)^2 / (\sigma_1+3.65\sigma_2)$

$= -5.15\ N/mm^2 < 7.98\ N/mm^2$.....OK...Stresses are with in the limit.

Provide nominal reinforcement 0.25 % on each face meridional and 0.2 % on each face hoop.

So, Meridional steel = $0.25 \times 1000 \times 750 / 100$

$= 1825\ mm^2$

Hoop steel = $0.2 \times 1000 \times 750 / 100$

$= 1500\ mm^2$

6.7 DESIGN OF CONE RING BEAM OF COMPARTMENT SILO

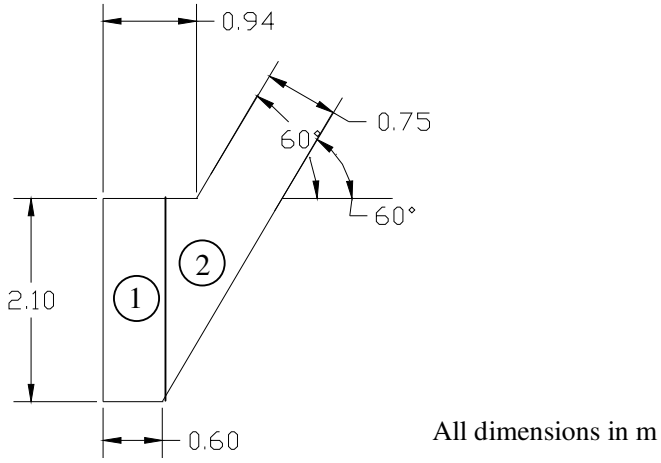


FIGURE: 6.13 SECTION OF THE CONE RING BEAM

Area of portion 1 and 2

A_1 = 2.1 x 0.6

= 1.26m²

A_2 = ½ x (0.8626+0.3464) x 2.1

=1.269m²

Total area A = A_1 + A_2 = 1.26 + 1.269 = 2.529 m^2

X = (A_1X_1+A_2X_2) / (A_1+A_2)

= (1.26 x 0.3) + (1.269 x 1.003) / (1.26 + 1.269)

= 0.653

Y = (A_1Y_1 + A_2Y_2) / (A_1+A_2)

= (1.26 x 1.05) + (1.269 x 0.7) / (1.26+1.269)

= 0.874

C.G (0.653,0.874)

Ixx = (bd³/12+Ah²) + (bd³/36+Ah²)

= [(0.6 x 2.1³/12 + 1.26 x 0.176²) + (1.209 x 2.1³/36+ 1.269 x 0.174²)]

= 0.85 m^4

$I = b \times d^3/12$

$0.85 = b \times 2.1^3 / 12$

So, b = 1.10m

Equivalent width = b = 1.10 m

Equivalent depth = d = 2.1 m

From STAAD.Pro Output of cone analysis

Fy = 5400 kN / unit thickness

Equivalent thickness = 0.750 m

Therefore, Fy = 5400 x 0.75

= 4050 kN/m width

Vertical force due to Fy = Fy sin 60˚

= 3507 kN/m

Horizontal force due to Fy = Fy cos 60˚

= 2025 kN/m

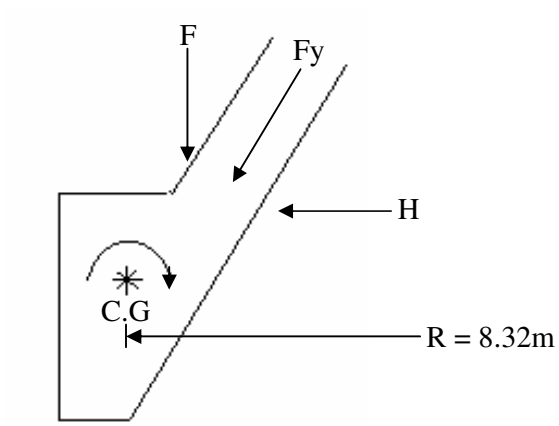


FIGURE: 6.14GEOMETRY SHOWN FORCES

These forces act at an offset from the C.G. of the ring beam in x and y direction.

Offset in x direction = X = 0.2920 m

Offset in y direction = Y = 0.1240 m

Net moment @ C.G. of ring beam due to Fy force = T = 772.94 kN

This moment causes torsion in the ring beam.

Hence,

Equivalent bending moment Me = T x (1+b/d)/1.7 ……. (6.14)

= 772.94 x (1 + 1.1/2.1) / 1.7

= 1322.68 kNm/m

Equivalent shear Ve = 1.6 x T / b ……. (6.15)

= 1.6 x 772.94 / 1.10

= 1124.28 kN/m

Longitudinal steel required in the ring beam due to bending moment,

Bending stress Fb = Me x (b/2) x ($bd^3/12$) ……. (6.16)

= 1322.68 x 1.10 / 2 x ($1.1 \times 2.1^3/12$)

= 617.57 kN/m²

Allowable stress in the steel in bending [6] (Fa) = 2100 kg/cm²

Area of steel required to resist bending

Ast_1 = Fb x b x d / Fa

= 617.57 x 1.1 x 2.1 / 210000

= 6.793×10^{-3} m²

= 67.93 cm^2

Ring beam is supported all over on periphery. If it is assumed that its displacement in the radial direction is zero, theoretically there is no hoop tension due to meridional force. However, it is considered that there will be hoop tension due to this force and hoop compression due to frictional resistance available on ledge support.

Longitudinal steel required in the ring beam due to ring tension

Ast_2 = (1-0.4) x Fy cos 60˚ x R / Fa

= 0.6 x 2025 x 8.32 / 210000

= 481.37 cm²

Where friction co-efficient between concrete- concrete = 0.4

Total Ast = Ast_1 + Ast_2

= 67.93 + 481.37

= 549.3 cm²

Ast $_{prov.}$ = 78 – 32 T bars = 627.3 cm²= 2.71 %

6.7.1 CHECK FOR SHEAR [5]

Total shear = V + Ve

= 3507 + 1124.28

= 4631.28 kN/m

Nominal shear Tv = 4631.28 / 1.1 x 2.1

= 2 N/mm²

Tc $_{max}$ = 2.8 N/mm² For M: 20

Tc $_{max}$> Tv …….Ok.

Pt provided = 2.71%

Therefore, design shear stress Tc = 0.82 N/mm²

Tv > Tc.

Hence shear reinforcement is required.

Vc = Tc x b x d = 0.82 x 1.1 x 2.1 = 1894.2 kN/m

Vus = 4631.28 – 1894.2 = 2737.08 kN

$$Sv = \frac{0.87\,fyAsvd}{Vus}$$

$$= \frac{0.87 \times 415 \times (\pi / 4) \times 16^2 \times 4 \times 2100}{2737.08 \times 10^3}$$

= 222.79 mm

Provide stirrups 16φ-200 mm c/c

Ast $_{prov.}$ = 165.73 mm c/c

6.8 Design of bottom ring beam

Total load on bottom ring beam = 230338.31 kN

Load /m length = 230338.31/Π × 17.35

= 4225.88 kN/m

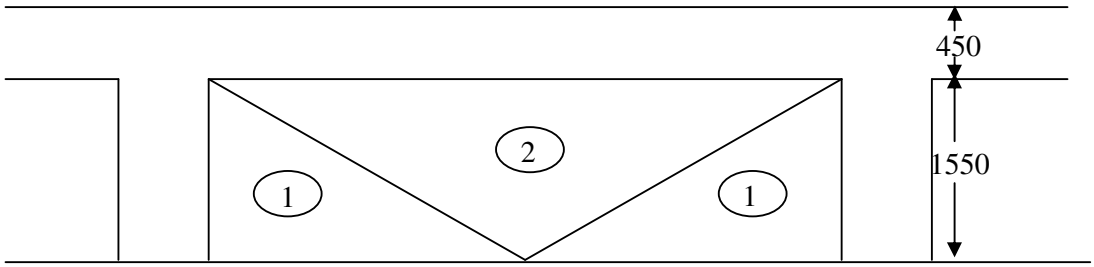


FIGURE: 6.15 PLAN VIEW OF BOTTOM RING BEAM

Area of triangle no.1 = 1.3592 m²

Total area = 5.9 m²

Load /m² = 230338.31 / 5.9 = 39040.39kN /m²

Load per triangle no.1 = 39040.39 × 1.3592 = 53063.7 kN

Load per triangle no.2 = 39040.39 × 3.1816 = 124210.9 kN

So, from the load distribution, 54% load is taken by wall and remaining 46% load is taken by columns.

6.8.1 MOMENT FOR CANTILEVER PROJECTION.

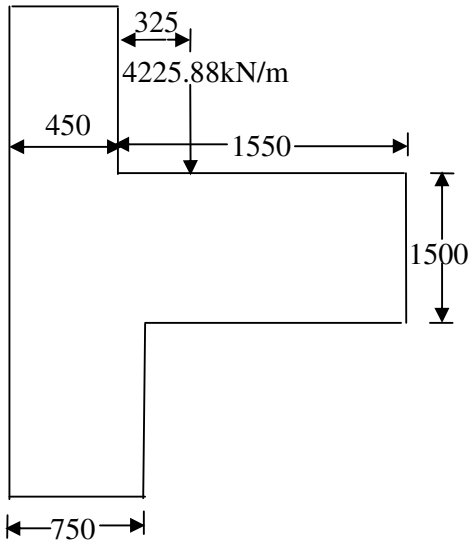


FIGURE: 6.16 SECTION OF BOTTOM RING BEAM

Cantilever moment

M = [(0.54 × 4225.88) × 0.325]

= 741.64 kNm

Cantilever R/F

$Mu/bd^2 = \frac{1.5 \times 741.64 \times 10^6}{1000 \times 1450^2}$

= 0.52 N/mm^2

Pt = 0.147

Ast = 0.147 × 1000 × 1500/100

= 2205 mm²

Provide 20T – 200mm c/c bars

Ast $_{prov.}$ = 2454 mm²

6.8.2 FOR LONG SPAN SUPPORTED ON COLUMN [9]

As per the constants given in Reinforced concrete design By H.J.Shah [9] (Volume: 2) for the continuous ring beam design supported on 12 columns.

Support moment = 0.045×4225.88×0.46×8.95²×Π/6 = 3668.87 kNm

Mid Span moment = 0.023×4225.88×0.46×8.95²× Π/6 = 1875.20 kNm

Torque = 0.0022×4225.88×8.95²×0.46× Π/6 = 179.37 kNm

Shear at support = 4225.88×8.95×Π/12×0.46 = 4554.77 kN

At support R/F

Support moment = 3668.87 kNm

$Mu/bd^2 = \frac{1.5 \times 3668.87 \times 10^6}{2000 \times 1450^2}$

= 1.31 N/mm^2

Pt = 0.382

Ast = 0.084× 1000 × 1500/100

= 11460 mm²

Provide 16-32T, 8- Thru + 8 – extra as support reinforcement.

Ast $_{prov.}$ = 12864 mm²

At span R/F

Mid Span moment = 1875.20 kNm

$$Mu/bd^2 = \frac{1.5\times1875.20\times10^6}{2000\times1450^2} = 0.67$$

Pt = 0.191

Ast = 0.191×2000×1500/100

= 5730 mm²

Provide 6-32T & 2-28T bars as span reinforcement.

Ast $_{prov}$. = (48.24 + 12.32)

= 60.56 mm²

Moment due to torque [5]

Mt = τ_u (1 + D/b)/1.7

$$= (1.5\times179.37)\left[\frac{1+1500/2000}{1.7}\right]$$

= 279.97kNm < 1875.20 kNm

6.8.3 CHECK FOR SHEAR [5]

Total shear = 4554.77 kN

$$\tau_v = \frac{4554.77\times10^3}{2000\times1450}$$

= 1.57 N/mm² < 3.5 N/mm²

Pt = 0.44

τ_c = 0.468 N/mm^2

Vuc = 0.468 × 2000× 1450 × 10^{-3}

= 1359.52 kN

Vus = 4554.77 – 1359.52

= 3195.25 kN

$$Sv = \frac{0.87\,fyAsvd}{Vus}$$

$$= \frac{0.87\times415\times(\pi/4)\times16^2\times6\times1450}{3195.25\times10^3}$$

= 200 mm

Provide as stirrups, 6 – legged, 16φ – 200mm c/c bars.

6.9 DESIGN OF SHAFT WALL

6.9.1 CHECK IN WIND LOADING
(As per IS: 875 (Part 3) – 1987)[10]

Diameter of the shell = 18.9 m

Height of the structure = 60.8 m

Silo is situated in Malkhed, Karnataka.

Vb = 33 m/sec

K_1 = 1.0

For the value of the K_2, consider category – 2 and class – B

K_3 = 1.0

Wind force, $F = C_f \times Ae \times P_d$ ……. (6.17)

Where, P_d = Design pressure

$= 0.6 \times Vz^2$ Where $Vz = Vb \times K_1 \times K_2 \times K_3$

Ae = Effective frontal area

C_f = Force coefficient

For sample calculation:

Up to 10 m LVL:

$Vz = K_1 \times K_2 \times K_3 \times Vb$ ……. (6.18)

$= 1 \times 0.93 \times 1 \times 33$

= 30.69 m/sec

$Pd = 0.6 \times Vz^2$ ……. (6.19)

$= 0.6 \times 30.69^2$

$= 565.13$ N\m²

Table-23 Pg: 40

For circular structure,

Force coefficient $C_f = 0.7$

Wind force, $F = C_f \times Ae \times P_d$

$= 0.7 \times 189 \times 565.13$

= 74.76 Kn

TABLE: 6.17 FORCES AND MOMENTS DUE TO WIND

Height (m)	K_2	Vz (m/sec)	Pd (N/m²) (0.6 x Vz^2)	Ht. increment H (m)	A (H*D) m²	F (kN)	Ht.from bottom (m)	M (kNm)
10	0.93	30.69	565.13	10	189	74.76	5	373.83
10-15	0.97	32.01	614.78	5	94.5	40.66	12.5	508.34
15-20	1	33	653.4	5	94.5	43.22	17.5	756.38
20-30	1.04	34.32	706.72	10	189	93.49	25	2337.4
30-50	1.10	36.3	790.61	20	378	209.1	40	8367.8
50-60	1.11	36.76	810.78	10	189	107.2	55	5899.6

Total moment due to wind force = 18243.46 kNm

Wind base shear = 568.43 kN

6.9.2 CHECK IN SEISMIC LOADING

(As per IS: 1893-1984) [11]

Period of free vibration, T, of such structure when fix at base,shall be calculated from the following formula :

$$T = C_T \times \sqrt{\frac{Wt \times h'}{Es \times Ag}} \qquad \text{..... (6.20)}$$

Where C_T = coefficient depends on slenderness ratio = 16.44

Wt = total weight of the structure = 233172.86 kN

h' = height of structure above the base = 60.8 m

Es = modulus of elasticity of material of the structural shell

= 5000 x $\sqrt{f_{ck}}$ = 27386.13 N/mm^2

A = area of cross section at the base of the structural shell

= 2 x Π x 9.075 x 0.75 = 42.76 m^2

g = acceleration due to gravity = 9.81 m/sec^2

$$T = 20.74 \times \sqrt{\frac{233172.86 \times 60.8}{27386.13 \times 42.76 \times 9.81}}$$

= 0.724 Sec

For C_T:

r = $\sqrt{\frac{I}{A}}$

So, I = (π/64) × (18.9^4 – 17.4^4)

= 1763.97 m^4

A = 2 × Π × r × t = 2 × Π × 9.075 × 0.75 = 42.76 m^2

So, r= 6.423 m

From Table: 6,

C_T = 20.47

Assume damping 5%

And for T =0.724

Sa/g = 0.138

$\dot{a}_h$ = β × I × Fo × Sa / g

= 1 × 1 × 0.1 × 0.138

$\dot{a}_h$ = 0.0138

Base shear, Vb = K × C × a_h × W

= 1 × 0.68 × 0.0138 × 236325.35

= 2188.09 kN > 568.43 kN (Wind base shear)

So, the governing case is seismic force.

Calculation of Horizontal seismic loads:

1. Total load on the roof slab.

Equipment load/live load = $10 \times \pi/4 \times 18^2 = 2544.7$ kN

Self weight of the slab = $25 \times \pi/4 \times 18^2 \times 0.25 = 1590.4$ kN

Dust load = $6 \times \pi/4 \times 18^2 = 1526.8$ kN

Plaster load = $1.25 \times \pi/4 \times 18^2 = 318$ kN

Total vertical load = 5980 kN

Horizontal seismic force due to slab loading (H_1)

H_1 = vertical load × $\dot{a}_h$

= 5980 × 0.013

= 77.74 kN

2. Total load of 450mm thick wall up to cone bottom

Wall load = D × Π × t × h × W

= 18 × Π × 0.45 × 56.25 × 25

= 35784.7 kN

H_2 = 35784.7 × 0.013

= 465.20 kN

3. Total load of 750mm thick wall up to G.L

Wall load = (18.9 – 0.75) × Π × 0.75 × 2.8 × 25

= 2993.54 kN

H_3 = 2993.54 × 0.013

= 38.92 kN

4. Total load of hopper and ring beam

Volume $V_1 = 1/3 \times h \times D^2 \times \pi/4$

$= 14.2/3 \times 18^2 \times \pi/4$

$= 1204.5$ m³

Volume V_2 = [A_1+A_2+sqrt (A_1 × A_2)] × h/3

= 1035.83 m³

Where, A_1 = π/4 × 17²

= 226.98 m²

A_2 = π/4 × 2²

= 3.14 m²

Net volume = V_1-V_2

= 168.7 m³

Total load of hopper = 25 × 168.7

= 4217.5 kN

H_4 (a) = 4217.5 × 0.013

= 54.83 kN

Assuming equivalent size of ring beam 1.5 m × 2.0 m

Self weight of ring beam = 1.5 x 2.0 x Π × 17 × 25

= 4005.53 kN

H_4 (b) = 4005.53 × 0.013

= 52.07 kN

5. Total load due to stored materials

Storage capacity of silo = 150000 kN

Density of cement = 13 kN/m^3

As per the provision of the ACI 313 only 80% of the actual material weight in the silo is used as an "efficient weight"

= 150000 × 0.8

= 120000 kN

Cement stored up to top of the cone = 13 × π/4 × 18² × 41.7 = 137947.6 kN

H_5 (a) = 137947.6 × 0.8 × 0.013

= 1434.65 kN

Cement stored in the cone area = 150000 - 137947.6

= 12052.4 kN

H_5 (b) = 12052.4 × 0.8 × 0.013

= 125.34 kN

6. Total load due to Compartment walls

Load of Compartment wall up to apex of the cone

Wall load = 6 × 8 × 42.05 × 0.45 × 24

= 22707 kN

H_6 (a) = 22707 × 0.013

= 295.19 kN

Load of compartment wall in the inverted cone portion

Wall load = 6 × ½ × 12.1 × 8 × 0.45 × 25

= 3267 kN

H_6 (b) = 3267 × 0.013

= 42.47 kN

7. Load of bottom ring beam

Load = Π × (18.9 – 1) × 2 × 1.5 × 25

= 4217.59 kN

H_7 = 4217.59 × 0.013

= 54.83 kN

TABLE: 6.18 SEISMIC FORCES AND MOMENT

Name	Horizontal Force (kN)	C.G. From -3.5 m LVL (m)	Moment @ Top of Foundation (kNm)
H_1	77.74	64.17	4988.6
H_2	465.20	35.92	16710.0
H_3	38.92	4.9	190.7
H_4 (a)	54.83	12.81	702.4
H_4 (b)	52.07	9.026	470.0
H_5 (a)	1434.65	43.02	61718.6
H_5 (b)	125.34	17.96	2251.1
H_6 (a)	295.19	43.02	12699.1
H_6 (b)	42.47	17.96	762.8
H_7	54.83	7.05	386.6

Total load of the structure = 233172.86 kN

Total moment due to horizontal seismic force = 100879.8 kNm

Live load due to fill load = 150000 kN

Dead load excluding fill load = 83172.86 kN

Horizontal seismic moment due to dead load = 36910.2 kNm

Here Base shear and moment is much higher in the case of seismic force than the wind force. So, governing case is seismic force and the shaft wall is designed for the same.

6.9.2.1 Check for bending stress

There are two cases 1] When silo is full

2] When silo is empty

1] When silo is full = M × y/I

$= 100879.8 \times 10^6 \times 9.45 \times 10^3 / 1167.87 \times 10^{12}$

$= 0.816\ N/mm^2$

2] When silo is empty = M × y/I

$= 36910.2 \times 10^6 \times 9.45 \times 10^3 / 1167.87 \times 10^{12}$

$= 0.299\ N/mm^2$

Min. stress when silo is full = 5.45 – 0.816 = 4.634 N/mm^2

Max. stress when silo is full = 5.45 + 0.816 = 6.266 N/mm^2

$< 1.33 \times 10\ N/mm^2$

Shaft wall design [12]:

Assume 0.2% of steel in shaft wall.

Modular ratio m = Es/Ec = $2 \times 10^5 / 5000 \times \sqrt{30} = 7.30$

From the chart 7.6

For two openings,

$\beta = 15°$ & $\mu = 15°$

np = 7.3 × 0.002/1-0.002

= 0.015

For case: 1

e/r = 100879.8/233172.86×9.075 = 0.0477

For case: 2

e/r = 36910.2/83172.86×9.075 = 0.049

As per the charts given in Tall Chimney By S.N.Manohar [12],

The value of e/r starts from 0.2, so by taking minimum value for e/r as 0.2, it does not create any difference in both the cases.

So for both the cases from chart, C = 0.25

So permissible stress fc = C × W/r × t × (1-p)

$= 0.25 \times 233172.86 \times 10^{-3}/9.075 \times 0.75 \times (1-0.002)$

$= 8.5\ N/mm^2 \quad < \quad 0.4\ \sigma_{cv}$

Where $\sigma_{cv} = 30\ N/mm^2$

(As per [13], cl: 8.2.6)

6.10 DESIGN OF SOLID RAFT FOUNDATION [12]

Total load on foundation including self weight = 287837 kN

Take SBC = 550 kN/m^2 due to weathered rock

Hence area of foundation required = 287837 / 550

= 523.34 m^2

Total diameter of raft = $\sqrt{(523.34 \times 4)/\pi}$

= 25.8 m ≈ 26 m

So, outer diameter of raft = 26 m

Therefore, outside radius of raft (a) = 13 m

C/c diameter of shaft = 18.15 m

Thickness of Shaft = 0.750 m

Therefore, mean radius of shaft (βa) = 9.075m

So, β = 0.698 m

Area of raft footing, A = 530.93 m^2

Moment of Inertia of annular raft, I = 22431.58 m^4

Section modulus, Z = I / outer radius = 1725.51 m^3

Total load from superstructure, P = 287837 kN

Total moment at base of raft due to, M = 100879.8 kN m

P / A = p = 542.14 kN/m^2

M / Z = q = 58.46 kN/m^2

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

Hence, permissible pressure = 756.25 kN/m^2

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

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

= 600.60 kN/m2 < 756.25 kN/m^2

Therefore OK.

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

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

= 483.67 kN/m^2 > 0

Therefore footing is in compression.

6.10.1 ANALYSIS OF SOLID RAFT

6.10.1.1 Radial and tangential moment due to direct load

Constants:

$Y_1 = 1.96\beta^4 + 8\beta^2 \ln \beta + 2.52\ \beta^2 = 0.2923$

$Y_2 = -2.52 - 8 \ln \beta - 2.96\ \beta^2 = -1.087$

$Y_3 = Y_4 = 0$

$Y_5 = 16\ \beta^2 \ln \beta - 5.48\ \beta^2 + 1.96\ \beta^4 = -5.0075$

$Y_6 = 5.48 - 2.96\ \beta^2 = 4.0376$

$Y_7 = -8\ \beta^2 = -3.8985$

$Y_8 = -8$

Moments:

$f < \beta$

$$M_{rI} = \frac{pa^2}{64}\left[-12.6f^2 - 2.3Y_2 + \frac{0.85}{f^2}Y_3 - Y_4(3.15 + 2.3\ln f)\right] \quad \text{..... (6.21)}$$

$$M_{ti} = \frac{pa^2}{64}\left[-5.8f^2 - 2.3Y_2 + \frac{0.85}{f^2}Y_3 - Y_4(1.45 + 2.3\ln f)\right] \quad \text{..... (6.22)}$$

$f > \beta$

$$M_{re} = \frac{pa^2}{64}\left[-12.6f^2 - 2.3Y_6 + \frac{0.85}{f^2}Y_7 - Y_8(3.15 + 2.3\ln f)\right] \quad \text{..... (6.23)}$$

$$M_{te} = \frac{pa^2}{64}\left[-5.8f^2 - 2.3Y_6 + \frac{0.85}{f^2}Y_7 - Y_8(1.45 + 2.3\ln f)\right] \quad \text{..... (6.24)}$$

6.10.1.2 Radial and tangential moment due to external moment

Constants:

$Y_1 = -0.19\beta^4 + 5.46\ \beta^2 - 3 = -0.3844$

$Y_2 = 3/\ \beta^2 - 5.46 - 0.81\ \beta^2 = 0.3015$

$Y_3 = Y_4 = 0$

$Y_5 = -0.19\ \beta^4 + 5.46\ \beta^2 - 3 - 12 \ln \beta = 3.9287$

$Y_6 = -5.46 - 0.81\ \beta^2 = -5.8547$

$Y_7 = 3\ \beta^2 = 1.4619$

$Y_8 = 12$

Moments:

f < β

$$M_{ri} = \frac{qa^2}{192}\left[-20.6f^3 - 6.3fY_2 - \frac{1.7}{f^3}Y_3 - \frac{1.15}{f}Y_4\right]\cos\theta \qquad \text{..... (6.25)}$$

$$M_{ti} = \frac{qa^2}{192}\left[-7f^3 - 2.9fY_2 - \frac{1.7}{f^3}Y_3 - \frac{1.15}{f}Y_4\right]\cos\theta \qquad \text{..... (6.26)}$$

f > β

$$M_{re} = \frac{qa^2}{192}\left[-20.6f^3 - 6.3fY_6 - \frac{1.7}{f^3}Y_7 - \frac{1.15}{f}Y_8\right]\cos\theta \qquad \text{..... (6.27)}$$

$$M_{te} = \frac{qa^2}{192}\left[-7f^3 - 2.9fY_6 - \frac{1.7}{f^3}Y_7 - \frac{1.15}{f}Y_8\right]\cos\theta \qquad \text{..... (6.28)}$$

TABLE: 6.19 GOVERNING MOMENT OF SOLID RAFT FOUNDATION

Radial Distance (m)	Diameter d1 (m)	f d1/D1	Moment due to external loads		Moment due to external moments		Combined Moment	
			Mr (kN.m)	Mt (kN.m)	Mr (kN.m)	Mt (kN.m)	Mr (kN.m)	Mt (kN.m)
13.00	**26.00**	**1.000**	**-0.15**	**-247.19**	**-0.03**	**-68.75**	**0.17**	**315.94**
12.65	25.29	0.973	-27.87	-259.12	1.62	-72.76	29.49	331.88
12.29	24.58	0.945	-127.00	-280.46	-3.81	-78.17	130.82	358.63
11.94	23.87	0.918	-301.64	-309.56	-16.50	-84.84	318.14	394.40
11.58	23.16	0.891	-556.41	-344.44	-36.68	-92.60	593.09	437.04
11.23	22.45	0.863	-896.62	-382.70	-64.62	-101.25	961.24	483.95
10.87	21.74	0.836	-1328.33	-421.46	-100.70	-110.55	1429.03	532.01
10.52	21.03	0.809	-1858.56	-457.22	-145.36	-120.20	2003.92	577.42
10.16	20.32	0.782	-2495.41	-485.71	-199.18	-129.85	2694.60	615.56
9.81	19.61	0.754	-3248.36	-501.76	-262.87	-139.04	3511.23	640.80
9.45	18.90	0.727	-4128.49	-499.01	-337.31	-147.21	4465.80	646.22
9.08	**18.15**	**0.698**	**-5210.89**	**-467.02**	**-428.86**	**-153.95**	**5639.74**	**620.97**
8.70	17.40	0.669	-4499.44	-139.53	-383.15	-138.08	4882.60	277.61
7.83	15.66	0.602	-2964.50	567.03	-290.50	-105.81	3255.00	672.84
6.96	13.92	0.535	-1591.13	1199.22	-215.01	-79.37	1806.14	1278.59
6.09	12.18	0.468	-379.33	1757.03	-154.78	-58.11	534.10	1815.14
5.22	10.44	0.402	670.90	2240.47	-107.88	-41.39	778.78	2281.85
4.35	8.70	0.335	1559.55	2649.53	-72.43	-28.55	1631.98	2678.08
3.48	6.96	0.268	2286.63	2984.22	-46.50	-18.95	2333.13	3003.17
2.61	5.22	0.201	2852.14	3244.53	-28.20	-11.95	2880.34	3256.48
1.74	3.48	0.134	3256.07	3430.47	-15.63	-6.89	3271.69	3437.35
0.87	1.74	0.067	3498.43	3542.03	-6.86	-3.12	3505.29	3545.15
0.00	**0.00**	**0.000**	**3579.11**	**3556.00**	**0.00**	**0.00**	**3579.11**	**3556.00**

6.10.2 DESIGN OF SOLID RAFT

Depth required = $\frac{Mu}{Qbd^2}$

Assume width b = 1 m

d = $\sqrt{\frac{5639.74 \times 10^3}{0.719 \times 1.33 \times 10^6 \times 1}}$ = 2.53 m ≈ 2.6 m

So, take depth at raft wall section = 2.6 m

& depth at outer face = 1.5 m

Concrete Grade (f_{ck}) = 25 N/mm^2

Steel Grade (f_y) = 415 N/mm^2

Effective Cover = 75 mm

Here, Ast_{min} = 0.12 × 1000 × d / 100

& Ast_{req} = $\frac{M}{\sigma st \times j \times d}$ ..... (6.29)

Where σ_{st} = 2300 kg/cm^2

j = 0.904

TABLE: 6.20 DESIGN REINFORCEMENTS OF SOLID RAFT FOUNDATION

Radius	Diameter	Effective Depth	Mr	Ast min	Ast	Ast req	Dia of Bars	Spacing	Ast pro	Mt	Ast min	Ast	Ast req	Dia of Bars	Spacing	Ast pro
(m)	(m)	(m)	(kN m)	(mm^2)	(mm^2)	(mm^2)/m	(mm)	(mm)	(mm^2)/m	(kN m)	(mm^2)	(mm^2)	(mm^2)/m	(mm)	(mm)	(mm^2)/m
13.00	26.00	1.425	0.17	1710	0.44	1710.00	25	180	2727	315.94	1710	801.75	1710.00	25	200	2454
12.65	25.29	1.535	29.49	1842	69.47	1842.00	25	180	2727	331.88	1842	781.84	1842.00	25	200	2454
12.29	24.58	1.645	130.82	1974	287.57	1974.00	25	180	2727	358.63	1974	788.37	1974.00	25	200	2454
11.94	23.87	1.755	318.14	2106	655.53	2106.00	25	180	2727	394.40	2106	812.66	2106.00	25	200	2454
11.58	23.16	1.865	593.09	2238	1149.98	2238.00	25	180	2727	437.04	2238	847.41	2238.00	25	200	2454
11.23	22.45	1.975	961.24	2370	1760.01	2370.00	25	180	2727	483.95	2370	886.11	2370.00	25	200	2454
10.87	21.74	2.085	1429.03	2502	2478.49	2502.00	25	90	5464	532.01	2502	922.72	2502.00	25	150	3272
10.52	21.03	2.195	2003.92	2634	3301.40	3301.40	25	90	5464	577.42	2634	951.28	2634.00	25	150	3272
10.16	20.32	2.305	2694.60	2766	4227.42	4227.42	25	90	5464	615.56	2766	965.72	2766.00	25	150	3272
9.81	19.61	2.415	3511.23	2898	5257.68	5257.68	32	110	7311	640.80	2898	959.53	2898.00	25	150	3272
9.45	18.90	2.525	4465.80	3030	6395.73	6395.73	32	110	7311	646.22	3030	925.50	3030.00	25	150	3272
9.08	18.15	-	-	-	-	-	32	110	7311	-	-	-	-	25	150	3272
8.70	17.40	2.525	4882.60	3030	6992.65	6992.65	32	110	7311	277.61	3030	397.59	3030.00	25	150	3272
7.83	15.66	2.525	3255.00	3030	4661.68	4661.68	32	110	7311	672.84	3030	963.61	3030.00	32	150	5362
6.96	13.92	2.525	1806.14	3030	2586.68	3030.00	32	150	5362	1278.59	3030	1831.14	3030.00	32	150	5362
6.09	12.18	2.525	534.10	3030	764.92	3030.00	32	150	5362	1815.14	3030	2599.57	3030.00	32	150	5362
5.22	10.44	2.525	778.78	3030	1115.33	3030.00	32	150	5362	2281.85	3030	3267.97	3267.97	32	150	5362
4.35	8.70	2.525	1631.98	3030	2337.25	3030.00	32	150	5362	2678.08	3030	3835.43	3835.43	32	150	5362
3.48	6.96	2.525	2333.13	3030	3341.41	3341.41	32	150	5362	3003.17	3030	4301.01	4301.01	32	150	5362
2.61	5.22	2.525	2880.34	3030	4125.10	4125.10	32	150	5362	3256.48	3030	4663.79	4663.79	32	150	5362
1.74	3.48	2.525	3271.69	3030	4685.58	4685.58	32	150	5362	3437.35	3030	4922.83	4922.83	32	150	5362
0.87	1.74	2.525	3505.29	3030	5020.12	5020.12	32	150	5362	3545.15	3030	5077.21	5077.21	32	150	5362
0.00	0.00	2.525	3579.11	3030	5125.85	5125.85	32	150	5362	3556.00	3030	5092.75	5092.75	32	150	5362

7. PARAMETRIC STUDY

In designing compartment silo, different parameters are the governing factors for designing. Out of them the most important factor is the pressure exerted due to fill material.

In compartment silo the static material pressure is computed by Janssen's and Airy's methods. Here material pressure to be considered should include not only static pressure, but also possible pressure increases caused by filling, withdrawal of material, arching, collapse of arched material, aeration and eccentric discharge. This pressure increases is usually referred to as "overpressure". Now this computed static material pressure is multiplied by overpressure factor which ultimately gives estimate design a total pressure. This modified pressure is computed in chapter: 6.

Our parametric study mainly based on variation of actual pressure with change in hydraulic mean radius which ultimately depends upon the nos of compartments.

This fact is further clarified on the basis of……

7.1 STATIC PRESSURE BY DIFFERENT THEORIES

(a) Janssen's theory by (I) IS Approach (chart: 7.1)
(II) ACI Approach (chart: 7.2)

(b) Airy's theories (chart:7.3)

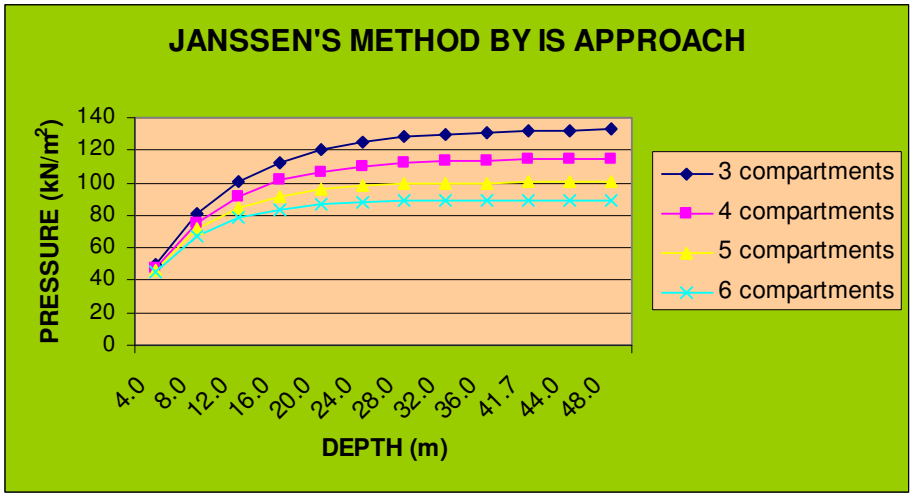


CHART: 7.1 STATIC PRESSURE COMPARISION BY JANSSENS METHOD (IS APPROACH)

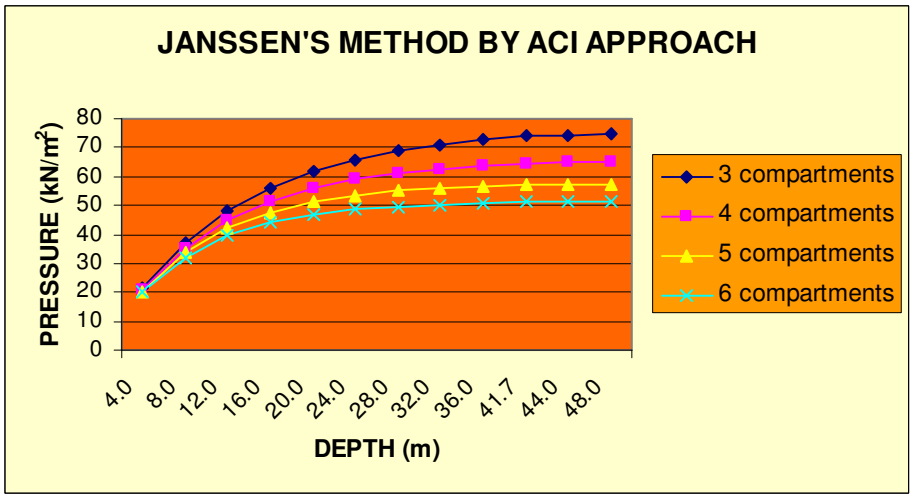


CHART: 7.2 STATIC PRESSURE COMPARISION BY JANSSENS METHOD (ACI APPROACH)

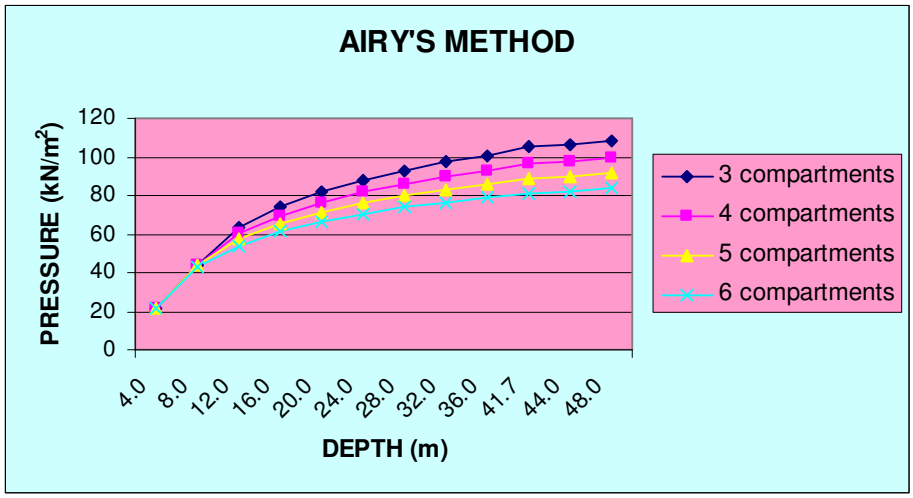


CHART: 7.3 STATIC PRESSURE COMPARISION BY AIRYS METHOD

7.2 DESIGN PRESSURE BY DIFFERENT STANDARDS

(a) Janssen's method by IS Approach

(b) Janssen's method by ACI Approach

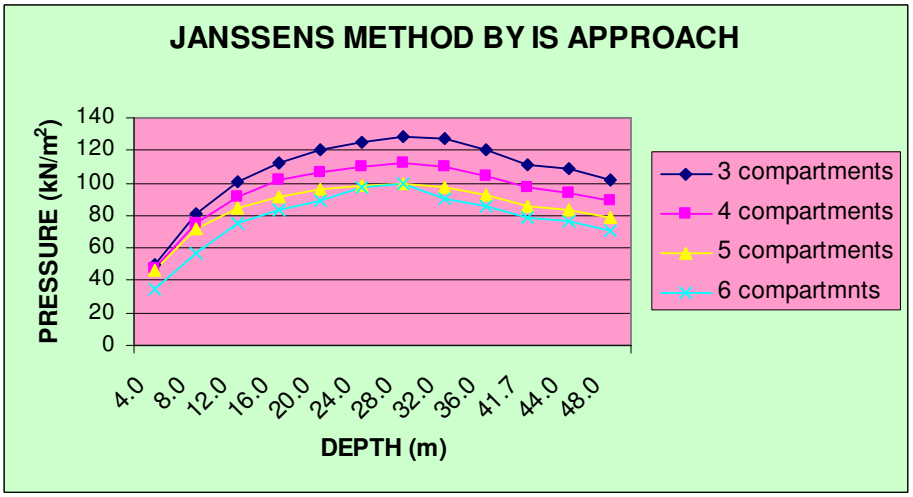
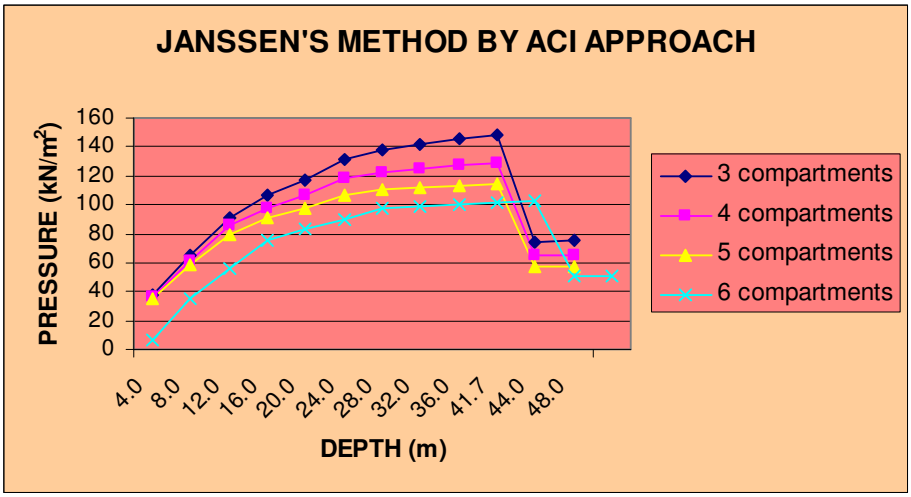


CHART: 7.4 DESIGN PRESSURES COMPARISION BY JANSSEN'S METHOD (IS APPROACH)



GRAPH: 7.5 DESIGN PRESSURE COMPARISION BY JANSSEN'S METHOD (ACI APPROACH)

Above discussion clearly leads us to arrive at the following graph which shows that the pressure variation directly proportional to the Hydraulic mean radius which ultimately dependant on nos of compartments.

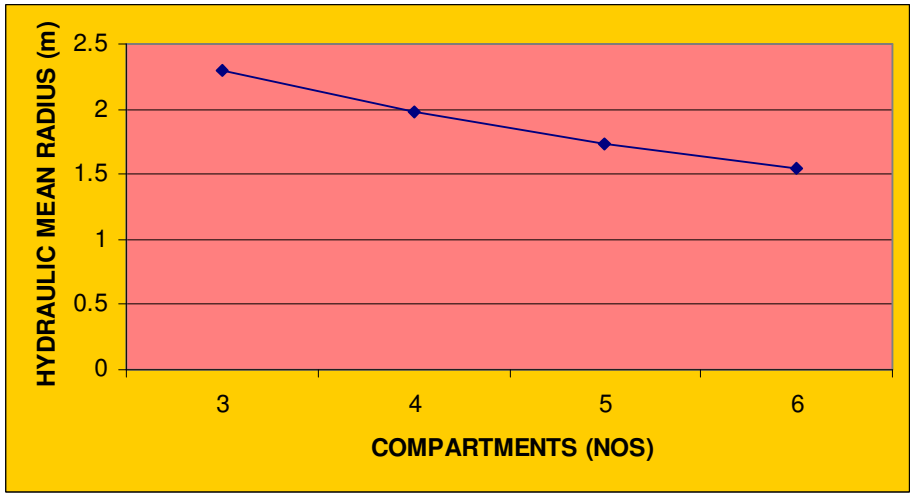


CHART: 7.6 CONCLUSION OF PARAMETRIC STUDY

8. CONCLUSION AND FUTURE SCOPE

8.1 SUMMARY AND CONCLUSIONS

In winding up of my thesis intellectual & creative work of this dissertation i.e. In ***Analysis and Design of compartment silo***, I arrived at certain facts which are as follows:

- In designing top slab of silo, a normal practice is to provide nos. of openings with steel beam framing but the supplier data regarding this is not available, my work is based on theoretical approach without openings.
- Lateral pressure by ACI approach gives more conservative results which I have used in my designed calculations but ACI Code does not give any idea regarding the overpressure factor especially for compartment silo and hence for computing design pressure, I have taken overpressure factor from advanced structural design By J.D.Buch. Which is the Data based on field experience.
- For analysis of outer circular wall of silo, I have created 2D-STAAD.Pro. model as entire structure analysis is very time consuming and is not necessary for this work. In finding the shear force and bending moments at various level of silo height I have considered pressure of 100kN/m^2.

If outer circular wall is represented by Infinite number sided regular polygon, the general observations are as under:

1. For all compartments full, hoop force in outer circular wall = p × r and horizontal moment in outer wall is zero but as outer circular wall is considered as 36 sided polygon, hoop force (when all compartments full) is lower and there are marginal horizontal moments.
2. If considered 32 sided or 108 sided polygon, In case all compartments are not full, there is only marginal reduction in horizontal moment in outer circular wall and marginal change in hoop force.

- As the normal adopted procedure the cone ring beam is designed on the basis of inclined thrust applied by the cone wall.
- The bottom ring beam is designed by proportionally divided reaction of cone ring beam on shaft wall and 12 columns giving due weight age to cantilever action and continuous circular beam loading respectively. This is purely based on real structural behaviour.
- In designing a shaft wall, I have assumed 0.2% steel which is safer side as the stresses developed in the wall are with in the permissible limit mentioned in Tall chimney by S.N.Manohar also In shaft wall maximum ultimate compressive stresses < 0.25fck and hence it is safe against buckling.
- By critically examining the graphs of the parametric study, we conclude that as the numbers of compartment in silo increases corresponding Hydraulic mean radius (R) will decreases and subsequently pressure also decreases.
- I have prepared programmes for designing the different components of the compartment silo as per IS Approach.

8.2 FUTURE SCOPE

- There are no code recommendations on pressure in compartment silos. With this transmutation on existing such silos, values of actual pressures are available in developed countries (Germany), when codes are formulated on this bases, design can be further optimized.
- Raft foundation can be analyse for this sort of huge work by using FEM technique.
- This work is extended to prepare a programme which optimizes the result between the different nos of compartments.
- Design of roof slab and effects of openings on slab by FEM.
- For wind or Earthquake dynamic analysis can be studied.

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