UNIT – I
FOOTINGS

Objective:

To learn the types and design procedure of Isolated and combined footings.

Syllabus:

Different types of footings – Design of isolated, and combined footings. Rectangular and circular footings subjected to axial loads.

Introduction:

Footings are structural elements that transmit column or wall loads to the underlying soil below the structure. Footings are designed to transmit these loads to the soil without exceeding its safe bearing capacity, to prevent excessive settlement of the structure to a tolerable limit, to minimize differential settlement, and to prevent sliding and overturning.

The settlement depends upon the intensity of the load, type of soil, and foundation level. Where possibility of differential settlement occurs, the different footings should be designed in such away to settle independently of each other.

Factors affecting types of footing:

The type of footing chosen for a particular structure is affected by the following:

1. The bearing capacity of the underlying soil.

2. The magnitude of the column loads.

3. The position of the water table.

4. The depth of foundations of adjacent buildings.

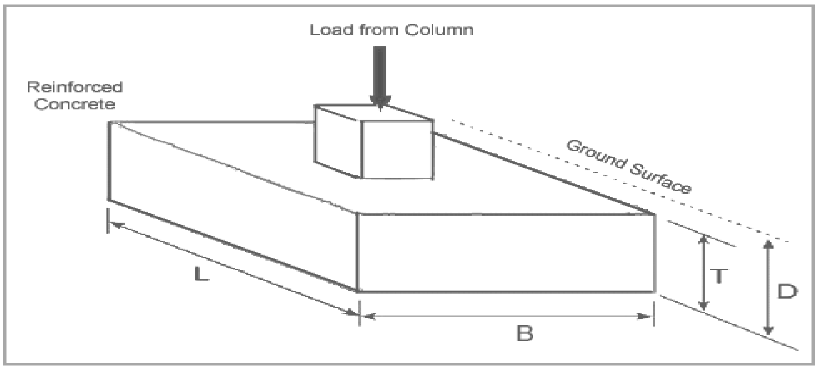
Footings may be classified as deep or shallow. If depth of the footing is equal to or greater than its width, it is called deep footing, otherwise it is called shallow footing. Shallow footings comprise the following types:

Types of shallow footing

- Isolated footing
- Strap footing
- Strip or continuous footing
- Combined footing
- Mat or raft footing

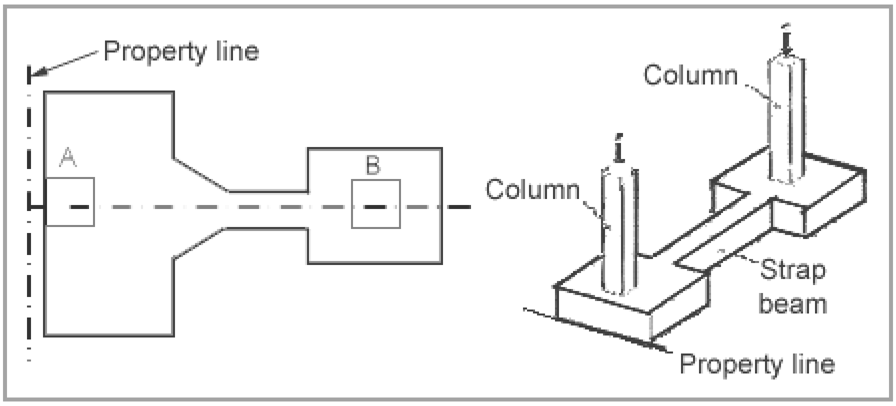
Isolated Footing:

It is circular, square or rectangular slab of uniform thickness. Sometimes, it is stepped or haunched to spread the load over a larger area. When spread footing is provided to support an individual column, it is called "Isolated footing" as shown in fig.

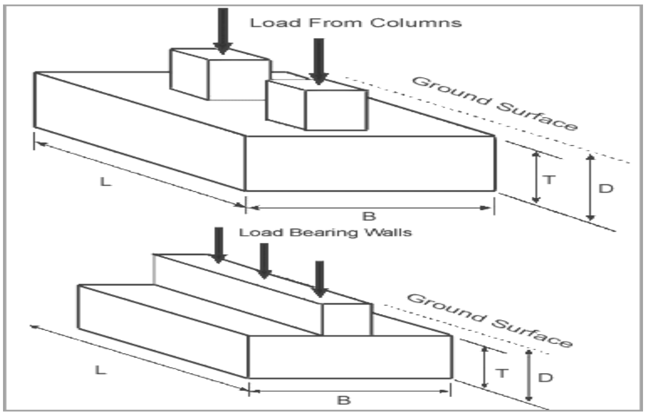


Strap Footing:

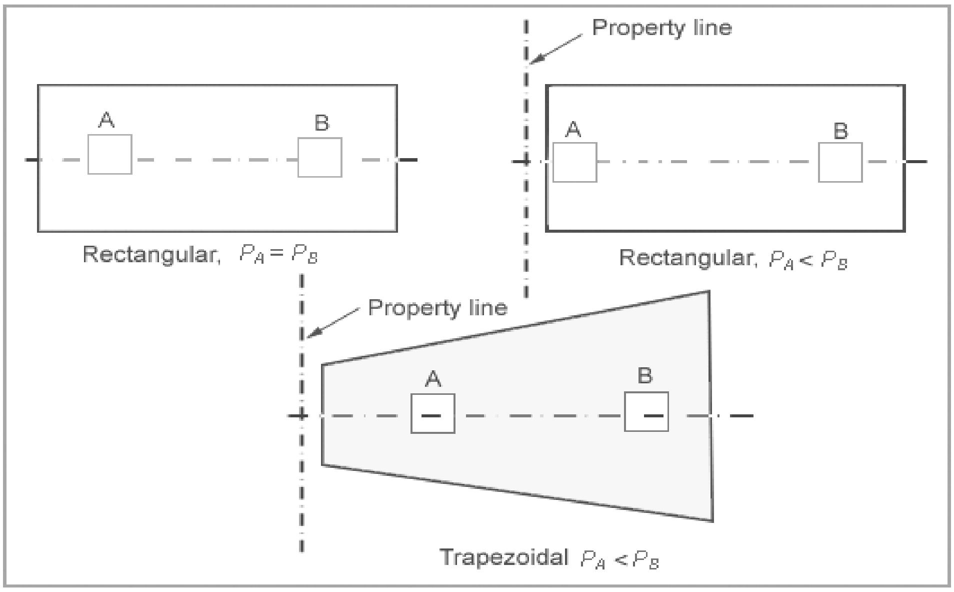
It consists of two isolated footings connected with a structural strap or a lever, as shown in fig.The strap connects the footing such that they behave as one unit. The strap simply acts as a connecting beam. A strap footing is more economical than a combined footing when the allowable soil pressure is relatively high and distance between the columns is large.



Strip or continuous footing :- A strip footing is another type of spread footing which is provided for a load bearing wall. A strip footing can also be provided for a row of columns which are so closely spaced that their spread footings overlap or nearly touch each other.



Combined footing:- When the spacing of the adjacent columns is so close that separate isolated footings are not possible due to the overlapping areas of the footings or inadequate clear space between the two areas of the footings, combined footings are the solution combining two or more columns. Combined footing normally means a footing combining two columns. A combine footing may be rectangular or trapezoidal in plan. Trapezoidal footing is provided when the load on one of the columns is larger than the other column.



Mat or Raft footing:- It is a large slab supporting a number of columns and walls under entire structure or a large part of the structure. A mat is required when the allowable soil pressure is low or where the columns and walls are so close that individual footings would overlap or nearly touch each other. Mat foundations are useful in reducing the differential settlements on non-homogeneous soils or where there is large variation in the loads on individual columns.

Design considerations

(a) **Minimum nominal cover** (cl. 26.4.2.2 of IS 456) :-

The minimum nominal cover for the footings should be more than that of other structural elements of the superstructure as the footings are in direct contact with the soil. Clause 26.4.2.2 of IS 456 prescribes a minimum cover of 50 mm for footings. However, the actual cover may be even more depending on the presence of harmful chemicals or minerals, water table etc.

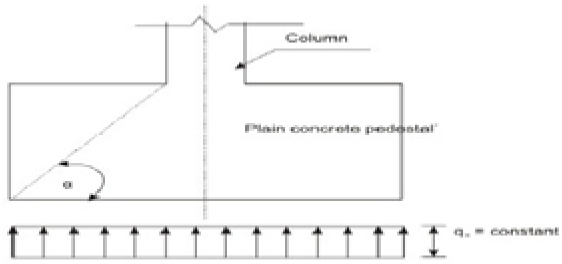
(b) **Thickness at the edge of footings** (cl. 34.1.2 and 34.1.3 of IS 456-2000):-

The minimum thickness at the edge of reinforced and plain concrete footings shall be at least 150 mm for footings on soils and at least 300 mm above the top of piles for footings on piles, as per the stipulation in cl.34.1.2 of IS 456-2000. For plain concrete pedestals, the angle α (fig.6) between the plane passing through the bottom edge of the pedestal and the corresponding junction edge of the column with pedestal and the horizontal plane shall be determined from the following expression (cl.34.1.3 of IS 456-2000).

$\tan \alpha \leq 0.9\{(100\, q_a/f_{ck}) + 1\}^{1/2}$

Where q_a = calculated maximum bearing pressure at the base of pedestal in N/mm^2, and

f_{ck} = characteristic strength of concrete at 28 days in N/mm^2



(c) **Bending moments** (cl. 34.2 of IS 456-2000):-

The critical section of maximum bending moment for the purpose of designing an isolated concrete footing which supports a column, pedestal or wall shall be at the face of the column, pedestal or wall for footing supporting a concrete column, pedestal or reinforced concrete wall.

(d) **Shear force** (cl. 31.6 and 34.2.4 of IS 456-2000):-

Footing slabs shall be checked in one-way or two-way shears depending on the nature of bending. If the slab bends primarily in one-way, the footing slab shall be checked in one-way vertical shear. On the other hand, when the bending is primarily two-way, the footing slab shall be checked in two-way shear or punching shear.

1. One-way shear (cl. 34.2.4 of IS 456-2000):-

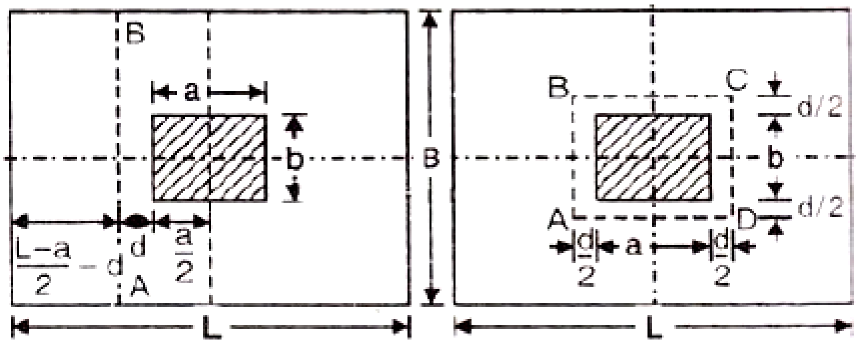
One-way shear has to be checked across the full width of the base slab on a vertical section located from the face of the column, pedestal or wall at a distance equal to effective depth of the footing slab. The design shear strength of concrete without shear reinforcement is given in Table 19 of cl.40.2 of IS 456-2000.

2. Two-way or punching shear (cls.31.6 and 34.2.4):-

Two-way or punching shear shall be checked around the column on a perimeter half the effective depth of the footing slab away from the face of the column or pedestal.

The permissible shear stress, when shear reinforcement is not provided, shall not exceed $k_s\tau_c$, where $k_s = (0.5 + \beta_c)$, but not greater than one, β_c being the ratio of short side to long side of the column, and $\tau_c = 0.25(f_{ck})^{1/2}$ in limit state method of design, as stipulated in cl.31.6.3 of IS 456-2000.

Normally, the thickness of the base slab is governed by shear. Hence, the necessary thickness of the slab has to be provided to avoid shear reinforcement.

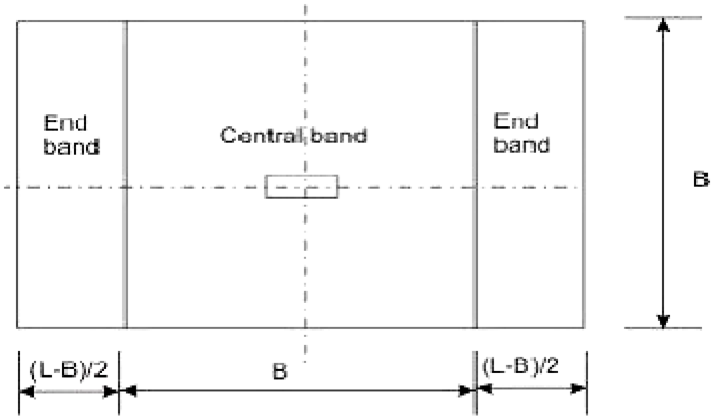


(e) **Tensile reinforcement** (cl.34.3 of IS 456-2000):-

The distribution of the total tensile reinforcement, calculated in accordance with the moment at critical sections shall be done as given below

(i) In two-way reinforced square footing slabs, the reinforcement extending in each direction shall be distributed uniformly across the full width/length of the footing.

(ii) In two-way reinforced rectangular footing slabs, the reinforcement in the long direction shall be distributed uniformly across the full width of the footing slab. In the short direction, a central band equal to the width of the footing shall be marked along the length of the footing, where the portion of the reinforcement shall be determined as given in the equation below. This portion of the reinforcement shall be distributed across the central band.



Bands for reinforcement in a rectangular footing

Reinforcement in the central band = {2/(β+1)} (Total reinforcement in the short direction).

Where β is the ratio of longer dimension to shorter dimension of the footing slab

Each of the two end bands shall be provided with half of the remaining reinforcement, distributed uniformly across the respective end band.

(f) **Transfer of load at the base of column** (cl.34.4 of IS 456-2000):-

All forces and moments acting at the base of the column must be transferred to the pedestal, if any, and then from the base of the pedestal to the footing, (or directly from the base of the column to the footing if there is no pedestal) by compression in concrete and steel and tension in steel. Compression forces are transferred through direct bearing while tension forces are transferred through developed reinforcement. The permissible bearing stresses on full area of concrete shall be taken as given below from cl.34.4 of IS 456-2000

$\sigma_{br} = 0.45 f_{ck} \ (A_1/A_2)^{1/2}$

with a condition that $(A_1/A_2)^{1/2} \leq 2.0$

where A_1 = maximum supporting area of footing for bearing.

A_2 = loaded area at the base of the column.

(g) **Nominal reinforcement** (cl. 34.5 of IS 456-2000):-

1. Clause 34.5.1 of IS 456 stipulates the minimum reinforcement and spacing of the bars in footing slabs as per the requirements of solid slab (cls.26.5.2.1 and 26.3.3b(2) of IS 456-2000, respectively).

(h) Depth of foundation:

The minimum, depth of foundation according to Rankine's theory is given by

$$h = \frac{p}{w}\left[\frac{1 - sin\emptyset}{1 + sin\emptyset}\right]^2$$

where h= depth of foundation

w= unit weight of soil

p = safe bearing capacity of soil under the footing

∅ = angle of repose

Design of Isolated footing:

Procedure:

1. Size of footing

To find out the size of footing and also find the net soil pressure on the footing.

Area of the footing $= \frac{self\ weight\ of\ the\ footing + load\ on\ column}{S.B.C\ of\ the\ soil}$

Net soil pressure on footing at ultimate loads with a factor of safety is given by

$$q_u = \frac{ultimate\ load}{size\ of\ the\ footing}$$

2. Depth of footing from the B.M considerations:

The critical section of the B.M will be at the face of the column

$$M_u = \frac{q_u B(B-b)^2}{8}$$

Where B= size of the footing

b= size of the column

3. Check for one way shear:

One way shear is checked at a critical section distant d from the column face.

The factored shear force is $V_{u1} = q_u B\left[\frac{(B-b)}{2} - d\right]$

Assume the percentage of reinforcement in the footing is 0.25 percent.

The permissible shear stress is taken from the table19 of IS: 456 code i.e

$$\tau_c = 0.36\ N/mm^2$$

One way shear resistance =$V_{c1} = \tau_c B d$

Then the design ultimate shear force (V_{u1}) is limited to the shear resistance of concrete (V_{c1}) by providing the necessary depth.

4. Check for Two way shear:

The critical section for two way shear is considered at a distance (d/2) from the periphery of the column.

Factored shear force is V_{u2}= $q_u \times Area\ of\ the\ shaded\ portion$

Two way shear resistance =$V_{c2} = k_s \tau_c [4(column\ size + d)d] Bd$

Where $k_s = 1.0$ and $\tau_c = 1.118\ N/mm^2$

Then the design ultimate shear force (V_{u2}) is limited to the shear resistance of concrete (V_{c2}) by providing the necessary depth.

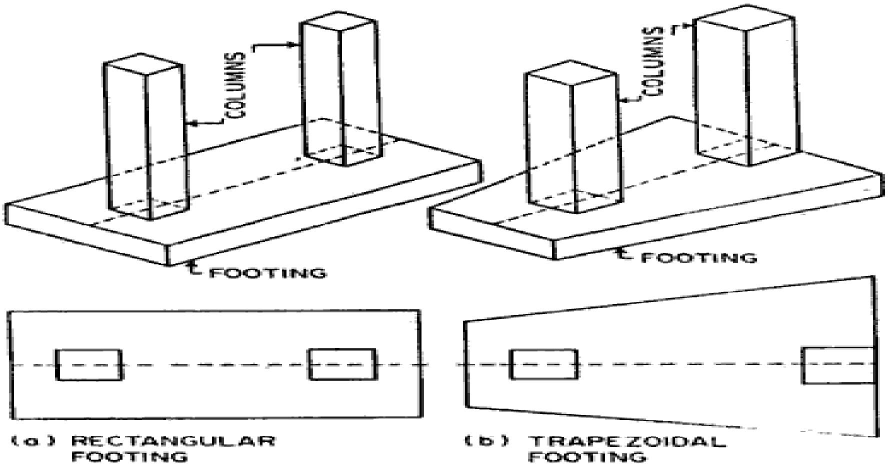
5. Design of Reinforcements:

$$M_u = 0.87\ f_y A_{st} d \left(1 - \frac{f_y A_{st}}{f_{ck} bd}\right)$$

Spacing of the bars = $\frac{a_{st}}{A_{st}} \times B$

Combined Footing:

- A spread footing which supports two or more columns is termed as combined footing. The combined footing may be rectangular, trapezoidal or Tee-shaped in plan.
- The geometric proportions and shape are so fixed that the centroid of the footing area coincides with the resultant of the column loads. This results in uniform pressure below the entire area of footing.
- Trapezoidal footing is provided when one column load is much more than the other. As a result, the both projections of footing beyond the faces of the columns will be restricted.
- Rectangular footing is provided when one of the projections of the footing is restricted or the width of the footing is restricted.



Steps for Design of Combined Footing

- Locate the point of application of the column loads on the footing.
- Proportion the footing such that the resultant of loads passes through the center of footing.
- Compute the area of footing such that the allowable soil pressure is not exceeded.
- Calculate the shear forces and bending moments at the salient points and hence draw SFD and BMD.
- Fix the depth of footing from the maximum bending moment.
- Calculate the transverse bending moment and design the transverse section for depth and reinforcement. Check for anchorage and shear.
- Check the footing for longitudinal shear and hence design the longitudinal steel
- Design the reinforcement for the longitudinal moment and place them in the appropriate positions.
- Check the development length for longitudinal steel
- Curtail the longitudinal bars for economy
- Draw and detail the reinforcement.

UNIT – II
FLAT SLABS

Flat slab is a reinforced concrete slab supported directly by concrete columns without the use of beams. Flat slab is defined as one sided or two-sided support system with sheer load of the slab being concentrated on the supporting columns and a square slab called ‘drop panels’.

Drop panels play a significant role here as they augment the overall capacity and sturdiness of the flooring system beneath the vertical loads thereby boosting cost effectiveness of the construction. Usually the height of drop panels is about two times the height of slab.

Flat Slabs are considered suitable for most of the construction and for asymmetrical column layouts like floors with curved shapes and ramps etc. The advantages of applying flat slabs are many like depth solution, flat soffit and flexibility in design layout. Even though building flat slabs can be an expensive affair but gives immense freedom to architects and engineers the luxury of designing

Benefit of using flat slabs are manifold not only in terms of prospective design and layout efficacy but is also helpful for total construction process especially for easing off installation procedures and saving on construction time. If possible, try to do away with drop panels as much as possible and try to make the best use of thickness of flat slabs. The reason is to permit the benefits of flat soffits for the floor surface to be maintained, ensure drop panels are cast as part of the column.

To utilize the slab thickness to optimum level, the essential aspects that should be kept in mind are:

1. Procedure related to design
2. Presence or absence of holes
3. Significance of deflections
4. Previous layout application experience

Types of Flat Slab Construction

Following are the types of flab slab construction:

- Simple flat slab
- Flat slab with drop panels

- Flat slab with column heads
- Flat slab with both drop panels and column heads

Uses of Column Heads

- It increase shear strength of slab
- It reduce the moment in the slab by reducing the clear or effective span

Uses of Drop Panels

- It increase shear strength of slab
- It increase negative moment capacity of slab
- It stiffen the slab and hence reduce deflection

Advantages of Flat Slabs

It is recognized that Flat Slabs without drop panels can be built at a very **fast pace** as the framework of structure is simplified and diminished. Also, speedy turn-around can be achieved using an arrangement using early striking and flying systems.

Flat slab construction can deeply **reduce floor-to –floor height** especially in the absence of false ceiling as flat slab construction does act as limiting factor on the placement of horizontal services and partitions. This can prove gainful in case of lower building height, decreased cladding expense and pre-fabricated services.

In case the client plans changes in the interior and wants to use the accommodation to suit the need, flat slab construction is the perfect choice as it offers that **flexibility** to the owner. This flexibility is possible due to the use of square lattice and absence of beam that makes channelling of services and allocation of partitions difficult.

Thickness of flat slab

Thickness of flat slab is another very attractive benefit because thin slab provides the advantage of increased floor to ceiling height and lower cladding cost for the owner. However, there is profound lower limit to thickness of slab because extra reinforcements are needed to tackle design issues. Besides this, added margin must be provided to facilitate architectural alterations at later stages.

Types of Flat Slab Design

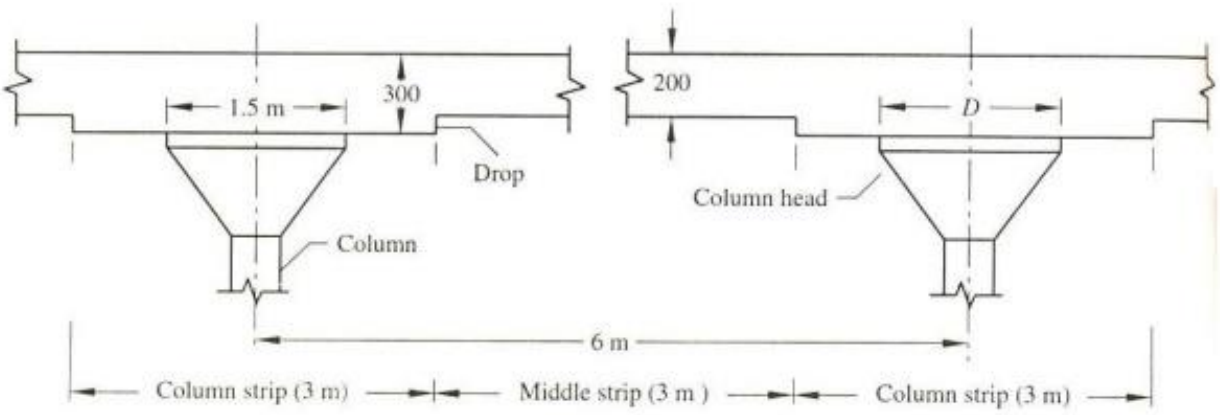
Multitudes of process and methods are involved in designing flat slabs and evaluating these slabs in flexures. Some of these methods are as following:

- The empirical method
- The sub-frame method
- The yield line method
- Finite –element analysis

For smaller frames, **empirical methods** are used but **sub-frame method** is used in case of more irregular frames. The designs are conceptualized by employing appropriate software but the fact is using sub-frame methods for very complicated design can be very expensive. The most cost effective and homogenous installation of reinforcements can be achieved by applying the **yield line method**. A thorough visualization in terms of complete examination of separate cracking and deflection is required since this procedure utilises only collapse mechanism.

Structures having floors with irregular supports, large openings or bears heavy loads, application of **finite- element analysis** is supposed to be very advantageous. Great thought is put into choosing material properties or installing loads on the structures. Deflections and cracked width can also be calculated using Finite- element analysis.

Design of Flat Slab



DESIGN OF FLAT SLAB

a) Interior panel

b) Exterior panel

Various components of flat slab:

i) Without drop and head

ii) With drop and without head

iii) With drop and head

Column strip : It is the design strip having a width of l2/4, where l2 is the span transverse to l1. l2 -longer span, moment is considered along the span l1

Middle strip : It is the design strip bounded by a column strip on its opposite sides

Proportioning of flat slabs:

As per cl.31 of IS456-2000, the span by depth ratio of two way slab is applicable for flat slabs and the values can be (l/d)modified by 0.9 for flat slabs with drops.
Take l/d as 32 for HYSD bars

As per ACI -The drop thickness should not be less than 100mm or (Thickness of slab)/4. While calculating span by depth ratio, longer span is used.

The thickness of slab should not be less than 125mm. The purpose of column drop is to reduce the shear stress and also reduce the reinforcement in the column strip.

The increase in column diameter at the head flaring of column head takes care of punching shear developed at a distance of d/2 all around the junction between the slab and column head.

Two methods of design are available for flat slabs:

1. Direct design method

2. Equivalent frame method

Direct design method: (Cl.31.4.1, IS456-2000)

Requirements for direct design methods are,

1. There must be atleast three continuous spans in each direction

The panels should be rectangular with ly/lx = l2/l1 ratio < 2

3. The columns must not offset by more than 10% of the span from either of the successive columns

4. Successive span length in each direction must not differ by more than one third of longer span.

5. Design live load must not exceed 3 times the designed dead load

Design procedure:

As per Cl.31.4.2.2, IS456-2000, the total moment for a span bounded by columns laterally is Mo = Wlo/2, where Mo is the sum of positive and negative moment in each direction. W is the total design load covered on an area L2L1

W = w x L2 x Ln

This moment is distributed for the column strip and middle strip

Moment distribution for Interior Panel:

Moment distribution for Interior Panel:

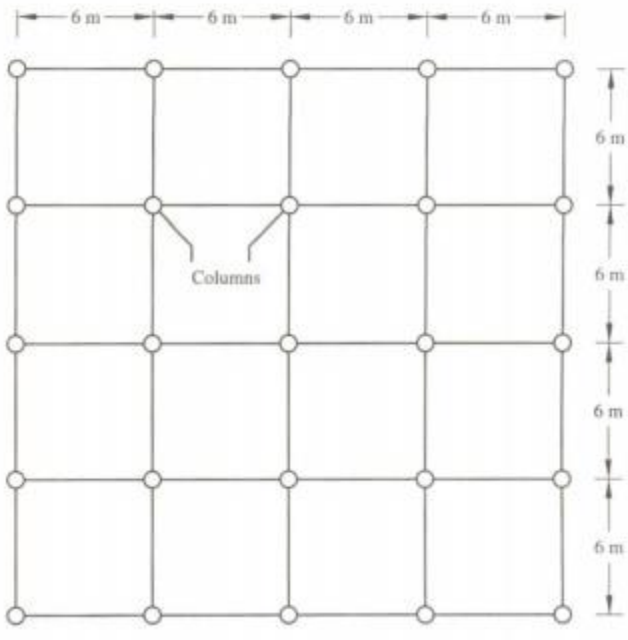
	Column strip	Middle strip
Negative moment (65%)	65 x 0.75 = 49%	65 x 0.2 = 15%
Positive moment (35%)	35 x 0.6 = 21%	35 x 0.4= 15%

$M_o = \frac{WL_o}{8}$

Also L_o should not be less than 0.65 times of L_1 [$L_n > 0.65L_1$)

1) Design a flat slab system (interior panel) to suit the following data: Size of the floor = 20 x 30m

Column interval = 5m c/c Live load on slab = 5kN/m^2
Materials used are Fe415 HYSD bars and M20 concrete



Proportioning of flat slab:

Assume l/d as 32, d = 5000/32 � d = 156.25mm
d = 175mm (assume), D = 175 + 20 + 10/2 = 200mm

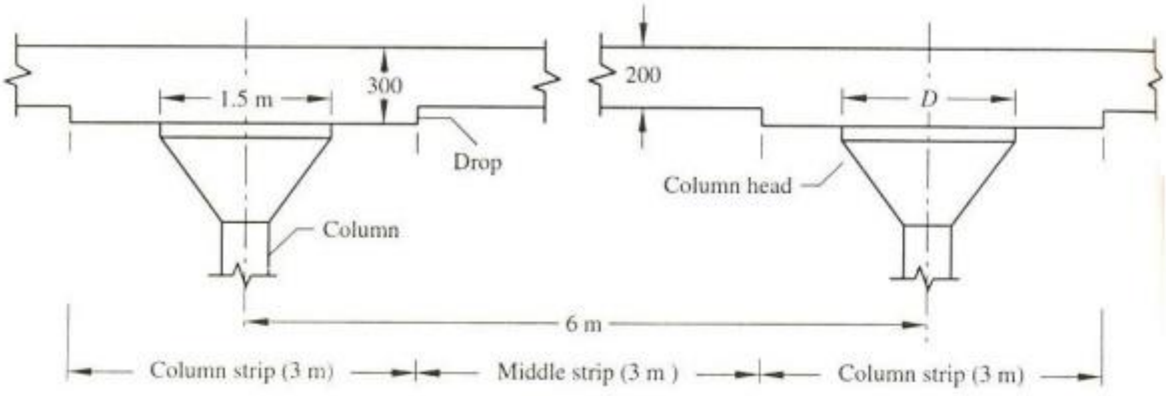
As per ACI code, the thickness of drop > 100mm and > (Thickness of slab)/4 Therefore, 100mm or 200/4=50mm

Provide a column drop of 100mm

Overall depth of slab at drop = 200 + 100 = 300mm Length of the drop > L/3 = 5/3 = 1.67m

Provide length of drop as 2.5m. For the panel, 1.25m is the contribution of drop. Column head = L/4 = 5/4 = 1.25m
$L_1 = L_2$ = 5m

$L_n = L_2$ -D = 5 -1.25 = 3.75m
As per code, Mo = *WLo/8*



Loading on slab:

(Average thickness = (300 + 200)/2 = 250mm)

Self weight of slab = 25 x 0.25 = 6.25 kN/m^2

Live load = 5 kN/m^2

Floor finish = 0.75 kN/m^2

Total = 12 kN/m^2

Factored load = 1.5 x 12 = 18 kN/m^2

W = w_u x L_2 x L_n = 18 x 5 x 3.75 = 337.5 kN

Total moment on slab panel = (337.5 x 3.75)/8 = 158.203 kNm

Distribution of moment:

Distribution of moment:

	Column strip	Middle strip
Negative moment (65%)	65 x 0.75 = 49% 0.49 x 158.2 = 77.52kNm	65 x 0.2 = 15% 0.15 x 158.2 = 23.73kNm
Positive moment (35%)	35 x 0.6 = 21% 0.21 x 158.2 = 33.22 kNm	35 x 0.4= 15% 0.15 x 158.2 = 23.73kNm

Check for depth adopted:

Column strip:

$M_u = 0.138.f_{ck}.b.d^2$

b = 2.5m

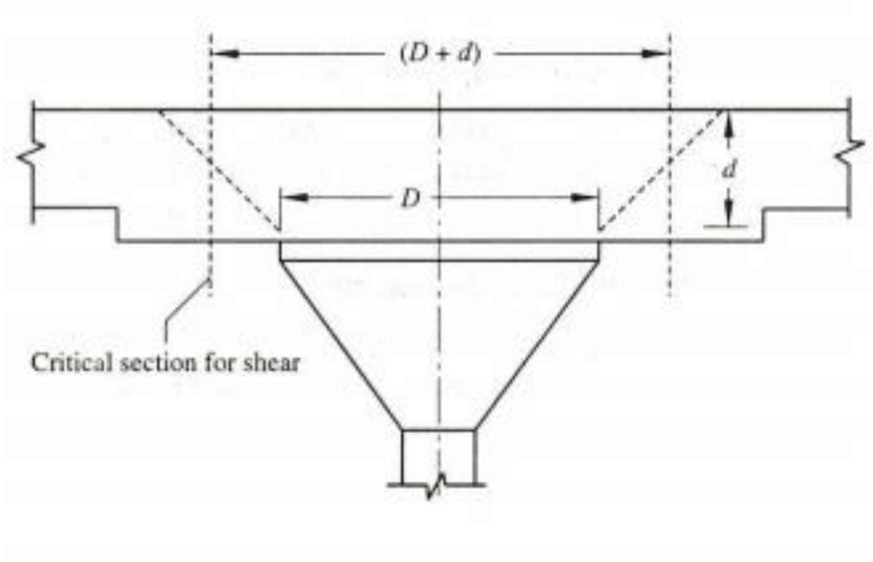
$77.5 \times 10^6 = 0.138 \times 20 \times 2.5 \times 1000 \times d^2$

d = 105.98mm ~ 106mm < 275mm Middle strip:

$M_u = 0.138.f_{ck}.b.d^2$ b = 2.5m

$23.73 \times 10^6 = 0.138 \times 20 \times 2.5 \times 1000 \times d^2$

d = 58.68mm ~ 59mm < 175mm



Critical section for shear in a flat slab

Check for punching shear:

The slab is checked for punching shear at a distance of d/2 all around the face of the column head. The load on the slab panel excluding the circular area of diameter (D + d) is the punching shear force.

Shear force = Total Load -(Load on circular area)

= 18 x 5 x 5 -(?(D ^{2}/4)+x wd)$_{n}$

= 417.12 kN

Shear force along the perimeter of the circular area = *Shear Force / p* (*D* +*d*) = 87.06 kN

Nominal shear stress: (b = 1m)

Nominal shear stress: (b = 1m)

$$\varsigma_v = \frac{V_u}{b.D} = \frac{87.06x10^3}{1000x275} = 0.317 \text{ N/mm}^2$$

Design shear stress: $\varsigma_c = K.\varsigma_c'$

Where, $K = (0.5 + \beta) \leq 1$

$= 1.5 \leq 1$

$\varsigma_c' = 0.25.\sqrt{f_{ck}} = 1.118 \text{ N/mm}^2$

$\varsigma_c = 1 \times 1.118 = 1.118 \text{ N/mm}^2$

$\varsigma_v < \varsigma_c$

Safe in shear.

Reinforcement:

Column strip: (b=2.5m), (d = 275mm)

Negative moment = 77.5×10^6 Nmm

$$M_u = 0.87.f_y.A_{st}.\left(d - 0.42\left(\frac{0.87.f_y.A_{st}}{0.36.f_{ck}.b}\right)\right) \quad \text{[OR]} \quad K = \frac{M_u}{bd^2} \rightarrow \text{Take } p_t \text{ from SP16}$$

$77.6 \times 10^6 = 99.29 \times 10^3.A_{st} - 3.04.A_{st}^2$

➔ $A_{st} = 800.16 \text{ mm}^2$

Required 10mm @ 240mm c/c

Min A_{st}: 0.12% of c/s = 0.12/100 x 1000 x 275 = 825 mm^2

Provide 10mm @ 230mm c/c

Positive moment = 33.2 kNm

A_{st} = 337.876 mm^2

Provide 8mm @ 370mm c/c

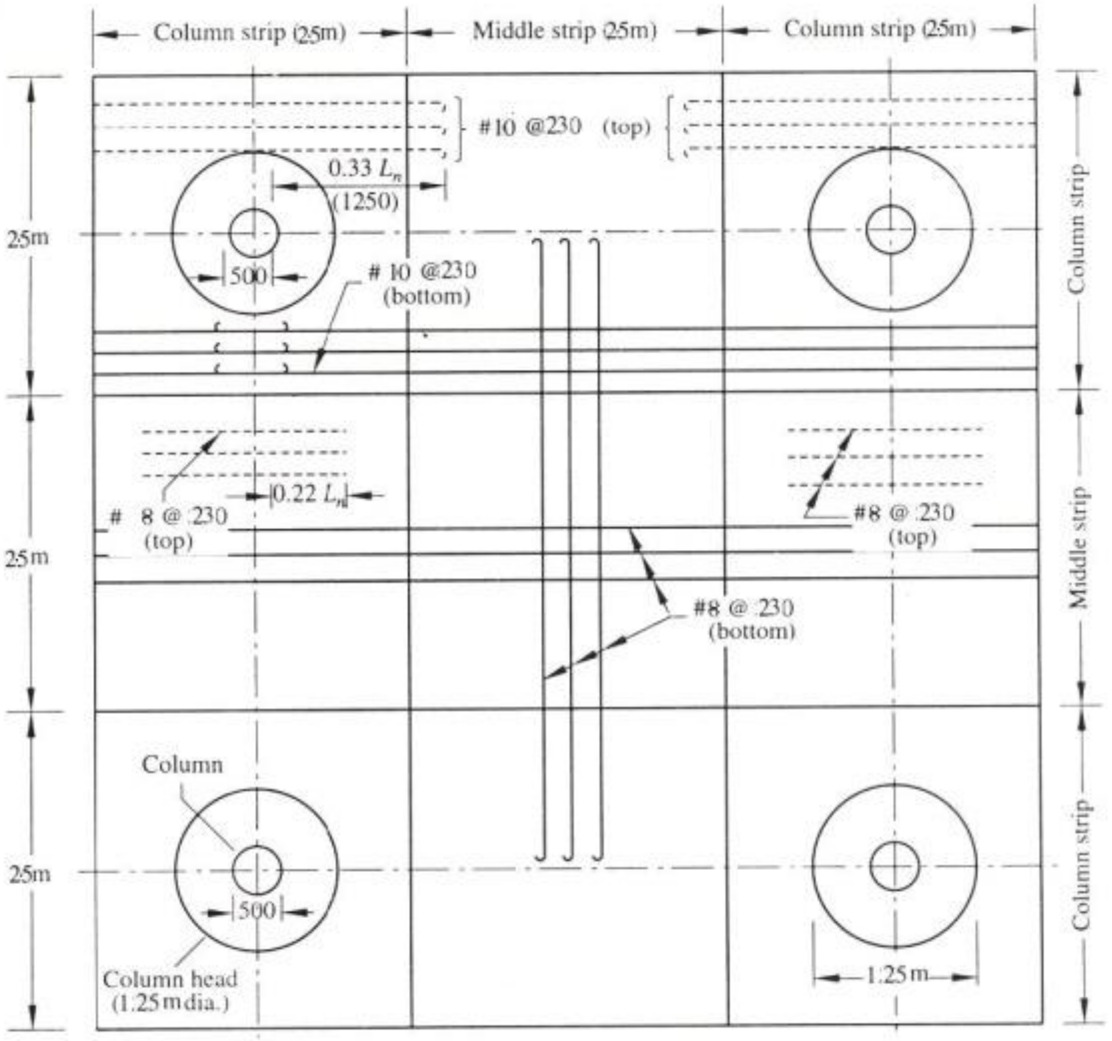
Min. steel: Provide 10mm @ 230mm c/c

Middle strip: (b = 2.5m), (d = 175mm)

Negative and positive moment: 23.7 kNm A_{st} = 382.6 mm^2

$A_{st\ min.}$ = (0.12/100 x 1000 x 2500 x 175) = 525 mm^2

Provide 8mm @ 230mm c/c.



Plan (interior panel)

FLAT SLAB [EXTERIOR PANEL] (Cl31.4.3.3, IS456-2000)

Stiffness of slab and column = $\frac{4EI}{L}$, where, I = $bd^3/12$ (or) $\pi d^4/64$, E = $5000\sqrt{f_{ck}}$

α_c is checked with $\alpha_{c\ min}$ given in Table 17 of IS456. From Cl.31.4.3.3, the interior and exterior negative moments and the positive moments are found.

Interior negative design moment is,

$$0.75 - \frac{0.10}{1+\frac{1}{\alpha_c}} \qquad \text{where, } \alpha_c = \frac{\sum K_c}{K_s}$$

Interior positive design moment is,

$$0.63 - \frac{0.28}{1+\frac{1}{\alpha_c}}$$

Exterior negative design moment is,

$$\frac{0.65}{1+\frac{1}{\alpha_c}}$$

The distribution of interior negative moment for column strip and middle strip is in the ratio 3:1 (0.75 : 0.25)

The exterior negative moment is fully taken by the column strip. The distribution of positive moment in column strip and middle strip is in the ratio 1.5 : 1 (0.6 : 0.4).

Design an exterior panel of a flat slab floor system of size 24m x 24m, divided into panels 6m x 6m size. The live load on the slab is 5 kN/m^2 and the columns at top and bottom are at diameter 400mm. Height of each storey is 3m. Use M20 concrete and Fe415 steel.

l/d = 32, d = 6000/32 = 187.5 mm

Length of drop = 3m

Length of drop = Column strip = 3m

Assume effective depth, d = 175mm, D = 200mm As per ACI, Assume a drop of 100mm

Depth of slab at the drop is 300mm Diameter of column head = l/4 = 6/4 = 1.5m

Loading on slab:

Self weight of slab = (0.2 + 0.3)/2 x 25 = 0.25 x 25 = 6.25 kN/m^2

Live load = 5 kN/m^2

Floor finish = 0.75 kN/m^2

Total = 12 kN/m^2

Factored load = 1.5 x 12 = 18 kN/m^2

To find the value of $\alpha_c = \frac{\sum K_c}{K_s}$

as per Cl.3.4.6.,

α_c = flexural stiffness of column and slab

ΣK_c = summation of flexural stiffness of columns above and below

ΣK_s = summation of flexural stiffness of slab

$\Sigma K_c = 2\left(\frac{4EI}{L}\right) = 2\left(\frac{4xExI_c}{L_c}\right) = \frac{2x4x22.3606x10^3 x1.25x10^9}{3000}$

Where, I = $\pi d^4/64 = \pi \times 400^4 /64$, E = $5000\sqrt{f_{ck}}$ = 22.3606×10^3

$\Sigma K_s = \frac{4xEx6000x(250)^3}{12x6000}$ = 5.208×10^6E

α_c = 0.644

From Table 17 of IS456-2000

$\alpha_{c\,min} = L_2/L_1$ = 6/6 = 1

$L_L / D_L = \frac{5}{(6.25+0.75)}$ = 0.71 ~ 1

$\alpha_{c\,min}$ = 0.7

$\alpha_{c\,min}$ should be < $\alpha_{c\,min}$

α_c = 0.7

Total moment on slab = $\frac{W.L_n}{8}$ = 273.375 kNm

W = $w_u \times L_2 \times L_n$ = 18 x 6 x 4.5 = 486 kN

L_n = 6 – 1.5 = 4.5m

As per Cl.31.4.3.3 of IS456-2000,

Exterior negative design moment is,

$\frac{0.65}{1+\frac{1}{\alpha_c}} \times M_o$ = 73.168 kNm where, α_c = 0.7

Interior negative design moment is,

$0.75 - \frac{0.10}{1+\frac{1}{\alpha_c}} \times M_o$ where, $\alpha_c = \frac{\sum K_c}{K_s}$

= 193.775 kNm

Interior positive design moment is,

$$0.63 - \frac{0.28}{1+\frac{1}{\alpha_c}} \text{ x } M_o$$

= 140.708 kNm

For column strip (60%),

= 0.6 x 140.708 = <u>84.43</u> kNm

For middle strip (40%),

= 0.4 x 140.708 = <u>56.28</u> kNm

Check for depth:

M_{ulim} = 0.138 fck.b.d^2

➔ 145.331 x 106 = 0.138 x 20 x 3 x d^2

➔ d_{cs} = 132.484 mm < 275 mm

M_{ms} = 82.4 kNm

➔ d_{ms} = 82.4 mm < 175 mm

Check for punching shear:

SF = TL – (Load on circular area)

= 18 x 6 x 6 – [π$(1.775)^2$/4] x 18 [w_n = 18]

= 648 – 44.54 = 603.45 kN [D + d = 1.5 + 0.275 = 1.775m]

Shear force/m along the perimeter of the circular area = $\frac{SF}{\pi(D+d)}$ = 108.216 kN/m

Nominal shear stress = ς_v = $\frac{V_u}{b.d}$ = $\frac{108.216x10^3}{1000x275}$ = 0.394 N/mm2

Design shear stress: $\varsigma_c = K.\varsigma_c'$.

Design shear stress: $\varsigma_c = K.\varsigma_c'$

where, $K = (0.5 + \beta) \leq 1$

$= (0.5 + 6/6) \leq 1$

$= 1.5 \leq 1 \quad \rightarrow K = 1$

$\varsigma_c' = 0.25 \sqrt{f_{ck}} = 1.118$ N/mm^2

$\varsigma_v < \varsigma_c$

Section is safe in shear.

A_{st} for exterior negative moment (73.168 kNm), b = 3000mm, d = 275mm,

$$M_u = 0.87.f_y.A_{st}\left(d - 0.42\left(\frac{0.87.f_y A_{st}}{0.36.f_{ck}.b}\right)\right) \qquad \text{[OR]} \qquad K = \frac{M_u}{bd^2} \rightarrow \text{Take } p_t \text{ from SP16}$$

$73.168 \times 10^6 = 99.28 \times 10^3.A_{st} - 2.53.A_{st}^2$

$\rightarrow A_{st} = 751.373$ mm^2

Required 10mm @ 310mm c/c

Min A_{st}: 0.12% of c/s = 0.12/100 x 3000 x 275 = 990 mm^2

Provide 10mm @ 230mm c/c

Similarly the reinforcement required in CS and MS for –ve and +ve moments are found and listed below:

Location	A_{st} Req.	Min. A_{st}	A_{st} Provided	Rein. Provided
Ext. –ve Mom. CS	751	990	990	10 @ 230 c/c
Int. –ve Mom. CS	1522	990	1522	10 @ 150 c/c
Int. –ve Mom. MS	791	630	791	10 @ 290 c/c
+ve Mom. CS	869	990	990	10 @ 230 c/c
+ve Mom. MS	925	630	925	10 @ 250 c/c

UNIT-3

RETAINING WALLS

Retaining walls are structures used to retain earth or water or other materials such as coal, ore, etc; where conditions do not permit the mass to assume its natural slope. The retaining material is usually termed as backfill. The main function of retaining walls is to stabilize hillsides and control erosion. In general, retaining walls can be divided into two major categories:

(a) Conventional retaining walls and

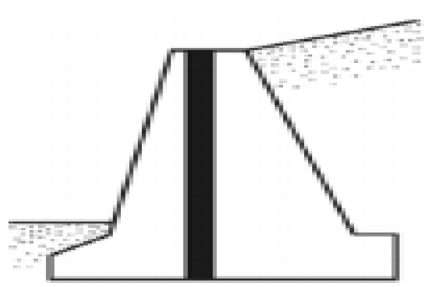
(b) Mechanically stabilized earth walls

Conventional retaining walls can generally be classified as

1. Gravity retaining walls
2. Semi gravity retaining walls
3. Cantilever retaining walls
4. Counter fort retaining walls

Gravity walls:

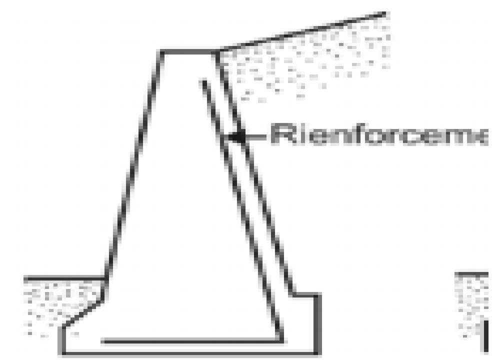
Gravity walls are stabilized by their mass. They are constructed of dense, heavy materials such as concrete and stone masonry and are usually reinforced. Some gravity walls do use mortar, relying solely on their weight to stay in place, as in the case of dry stone walls. They are economical for only small heights, not economical for high walls.



(a) Gravity wall

Semi Gravity Retaining Walls:

These walls generally are trapezoidal in section. This type of wall is constructed in concrete and derives its stability from its weight. A small amount of reinforcement is provided for reducing the mass of the concrete. In many cases, a small amount of steel may be used for the construction of gravity walls, thereby minimizing the size of wall sections. Such walls are generally referred to as semi-gravity walls. This can be classified into two: Cantilever retaining wall Counter fort retaining wall.

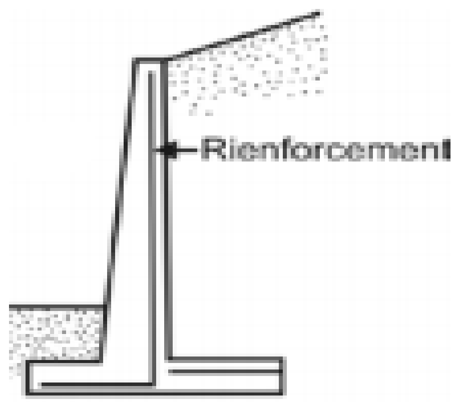


(b) Semigravity wall

Cantilever retaining walls :

This is a reinforced concrete wall which utilizes cantilever action to retain the backfill. This type is suitable for retaining backfill to moderate heights (4m-7m). In cross section most cantilevered walls look like “L”s or inverted “T”s. To ensure stability, they are built on solid foundations with the base tied to the vertical portion of the wall with reinforcement rods. The base is then backfilled to counteract forward pressure on the vertical portion of the wall. The cantilevered base is reinforced and is designed to prevent uplifting at the heel of the base, making the wall strong and stable. They can be faced with stone, brick, or simulated veneers. Reinforced Concrete Cantilevered Walls are built using forms. When the use of forms is not desired, Reinforced Concrete Block Cantilevered Walls are another option. Where foundation soils are poor, Earth

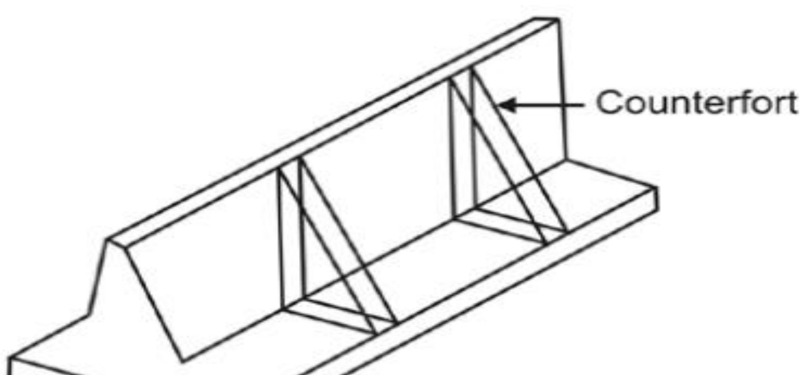
Tieback Retaining Walls are another choice. These walls are counterbalanced not only by a large base but also by a series of horizontal bars or strips extending out perpendicularly from the vertical surface into the slope. The bars or strips, sometimes called “deadmen” are made of wood, metal, or synthetic materials such as geo-textiles. Once an earth tieback retaining wall is backfilled, the weight and friction of the fill against the horizontal members anchors the structure.



(c) Cantilever wall

Counter-fort retaining walls:

When the height of the cantilever retaining wall is more than about 7m, it is economical to provide vertical bracing system known as counter forts. In this case, both base slab and face of wall span horizontally between the counter forts.



(d) Counterfort wall

Forces acting on Retaining walls:

The various forces acting on retaining wall are detailed as below

a) **Lateral earth pressure:**

The lateral forces due to earth pressure are the major force acting on the retaining wall. The magnitude of the force is expressed by

$$P_a = \frac{C_a \gamma_e (h')^2}{2}$$

Where C_a = Coefficient of active earth pressure

γ_e= Density of soil

h'= height of the back fill measured vertically above the heel

The coefficient of earth pressure depends on angle of repose $\emptyset$ and the inclination of the back fill to the horizontal θ

The general relation for the coefficient of active earth pressure based on Rankines theory is given by

$$C_a = \left[\frac{cos\theta - \sqrt{cos^2\theta - cos^2\emptyset}}{cos\theta + \sqrt{cos^2\theta - cos^2\emptyset}}\right] cos\theta$$

For the case of a level backfill, $\theta = 0$ and h'=h

$$C_a = \left\{\frac{1 - sin\emptyset}{1 + sin\emptyset}\right\}$$

The coefficient of passive earth pressure is given by the relation,

$$C_P = \left\{\frac{1 + sin\emptyset}{1 - sin\emptyset}\right\}$$

The total force due to active earth pressure is expressed as

$$P_a = P_{a1} + P_{a2}$$

$P_{a1} = C_a h_s \gamma_e h$ and $P_{a2} = \frac{(C_a \gamma_e h^2)}{2}$

The forces P_{a1} and P_{a2} act at a height of h/2 and h/3 respectively above the heel.

b) The vertical forces include the weight of soil, weight of stem, heel, toe slab and the fill above toe slab.

c) The soil pressure developed to resist the earth pressure and other vertical forces acting upwards from heel to toe. The pressure distribution at base is obtained by stability calculations comprising the equilibrium condition of vertical forces and moments.

Stability requirements:

The design of retaining walls should conform to the stability requirements specified in clause-20 of Is: 456 which includes overturning and sliding.

a) Overturning:

The factor of safety against overturning is expressed as

$(F.S)_{overturning} = \left(\frac{0.9M_s}{M_o}\right) \geq 1.4$

Where $Stabilizing\ moment\ M_s = W(B-x) + (P_a sin\theta)B$

$$overturning\ moment\ M_o = \left[\frac{C_a \gamma_e (h)^3}{6}\right] cos\theta$$

$W = W_1 + W_2 + W_3 + W_4$

And W_1= weight of earth fill

W_2= weight of stem

W_3= weight of heel and toe slab

W_4 = weight of earthfill over toe slab

X= distance of W from the heel

B= Base width of slab

b) Sliding:

The factor of safety against sliding is

$(F.S)_{sliding} = \left(\frac{0.9\mu W}{P_a cos\theta}\right) \geq 1.4$

Where μ = coefficient of friction between concrete and soil

W = Resultant soil pressure acting on the base slab

c) Shear key:

In the case of back fills with surcharge, the active pressures are relatively high and consequently the required factor of safety against sliding by the frictional forces above will not be sufficient. In such cases, it is advantageous to provide a shear key projecting below the base slab.

The factor of safety against sliding by the use of the shear key can be expressed as

$(F.S)_{sliding} = \left(\frac{0.9\mu W + P_p}{P_a cos\theta}\right) \geq 1.4$

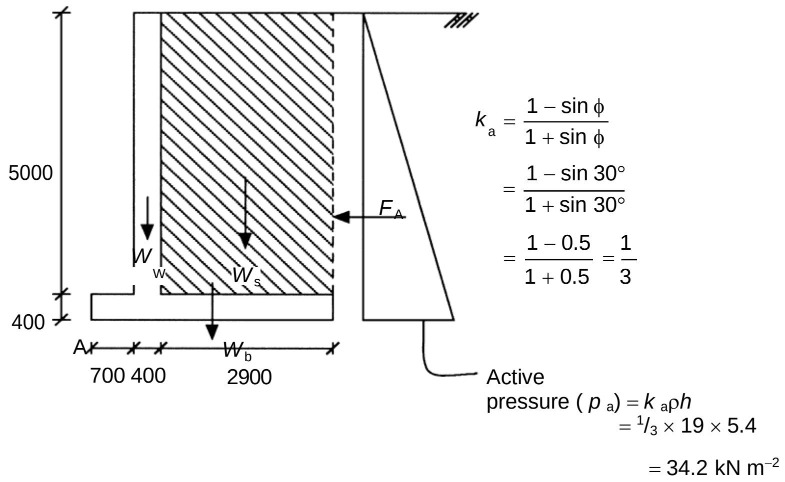
The passive resistance developed against sliding is

$$P_p = \frac{C_P \gamma_e (h_2^2 - h_1^2)}{2}$$

Design of a cantilever retaining wall

The cantilever retaining wall shown below is backfilled with granular material having a unit weight, ρ, of 19 kNm^{-3} and an internal angle of friction, of 30. Assuming that the allowable bearing pressure of the soil is 120 kNm^{-2}, the coefficient of friction is 0.4 and the unit weight of reinforced concrete is 24 kNm^{-3}

1. Determine the factors of safety against sliding and overturning.
2. Calculate ground bearing pressures.
3. Design the wall and base reinforcement assuming f_{cu} = 35 N/mm^2, f_y = 500 N/mm^2 and the cover to reinforcement in the wall and base are, respectively, 35 mm and 50 mm.



SLIDING

Consider the forces acting on a 1 m length of wall. Horizontal force on wall due to backfill, F_A, is

$$F_A = 0.5 p_a h = 0.5 \times 34.2 \times 5.4 = 92.34 \text{ kN}$$

and

Weight of wall (W_w)	$= 0.4 \times 5 \times 24 = 48.0$ kN
Weight of base (W_b)	$= 0.4 \times 4 \times 24 = 38.4$ kN
Weight of soil (W_s)	$= 2.9 \times 5 \times 19 = 275.5$ kN
Total vertical force (W_t)	$= 361.9$ kN

Friction force, F_F, is

$$F_F = \mu W_t = 0.4 \times 361.9 = 144.76 \text{ kN}$$

Assume passive pressure force (F_P) = 0. Hence factor of safety against sliding is

144.76/92.34 = 1.56 >1.50, OK

OVERTURNING MOMENTS

Taking moments about point A (see above), sum of overturning moments (Mover) is

FA x 5.4/3 = 92.34 x 5/3 = 166.2 kN

Example 3.16 *continued*

Sum of restoring moments (M_{res}) is

$$M_{res} = W_w \times 0.9 + W_b \times 2 + W_s \times 2.55$$
$$= 48 \times 0.9 + 38.4 \times 2 + 275.5 \times 2.55 = 822.5 \text{ kNm}$$

Factor of safety against overturning is

$$\frac{822.5}{166.2} = 4.9 > 2.0 \quad \text{OK}$$

GROUND BEARING PRESSURE

Moment about centre line of base (M) is

$$M = \frac{F_A \times 5.4}{3} + W_W \times 1.1 - W_S \times 0.55$$
$$= \frac{92.34 \times 5.4}{3} + 48 \times 1.1 - 275.5 \times 0.55 = 67.5 \text{ kNm}$$
$$N = 361.9 \text{ kN}$$
$$\frac{M}{N} = \frac{67.5}{361.9} = 0.187\ m < \frac{D}{6} = \frac{4}{6} = 0.666 \text{ m}$$

Therefore, the maximum ground pressure occurs at the toe, p_{toe}, which is given by

$$p_{toe} = \frac{361.9}{4} + \frac{6 \times 67.5}{4^2} = 116 \text{ kNm}^{-2} < \text{allowable } (120 \text{ kNm}^{-2})$$

Ground bearing pressure at the heel, p_{heel}, is

$$p_{heel} = \frac{361.9}{4} - \frac{6 \times 67.5}{4^2} = 65 \text{ kNm}^{-2}$$

BENDING REINFORCEMENT

Wall

Height of stem of wall, $h_s = 5$ m. Horizontal force on stem due to backfill, F_s, is

$$F_s = 0.5 k_a \rho h_s^2$$
$$= 0.5 \times {}^1/_3 \times 19 \times 5^2$$
$$= 79.17 \text{ kNm}^{-1} \text{ width}$$

Design moment at base of wall, M, is

$$M = \frac{\gamma_f F_s h_s}{3} = \frac{1.4 \times 79.17 \times 5}{3} = 184.7 \text{ kNm}$$

Effective depth

Assume diameter of main steel (Φ) = 20 mm.
Hence effective depth, d, is

$$d = 400 - \text{cover} - \Phi/2 = 400 - 35 - 20/2 = 355 \text{ mm}$$

Ultimate moment of resistance

$$M_u = 0.156 f_{cu} b d^2 = 0.156 \times 35 \times 10^3 \times 355^2 \times 10^{-6} = 688 \text{ kNm}$$

Since $M_u > M$, no compression reinforcement is required.

Example 3.16 *continued*

Steel area

$$K = \frac{M}{f_{cu}bd^2} = \frac{184.7 \times 10^6}{35 \times 10^3 \times 355^2} = 0.0419$$

$$z = d[0.5 + \sqrt{(0.25 - K/0.9)}]$$

$$= 355[0.5 + \sqrt{(0.25 - 0.0419/0.9)}] = 337 \text{ mm}$$

$$A_s = \frac{M}{0.87f_y z} = \frac{184.7 \times 10^6}{0.87 \times 500 \times 337} = 1260 \text{ mm}^2/\text{m}$$

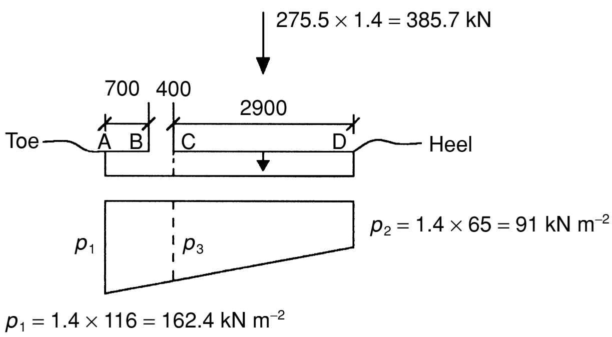
Hence from *Table 3.22*, provide H20 at 200 mm centres (A_s = 1570 mm^2/m) in near face (NF) of wall. Steel is also required in the front face (FF) of wall in order to prevent excessive cracking. This is based on the minimum steel area, i.e.

$$= 0.13\%bh = 0.13\% \times 10^3 \times 400 = 520 \text{ mm}^2/\text{m}$$

Hence, provide H12 at 200 centres (A_s = 566 mm^2)

Base

Heel



$$p_3 = 91 + \frac{2.9(162.4 - 91)}{4} = 142.8 \text{ kNm}^{-2}$$

Design moment at point C, M_c, is

$$\frac{385.7 \times 2.9}{2} + \frac{2.9 \times 38.4 \times 1.4 \times 1.45}{4} - \frac{91 \times 2.9^2}{2} - \frac{51.8 \times 2.9 \times 2.9}{2 \times 3} = 160.5 \text{ kNm}$$

Assuming diameter of main steel (Φ) = 20 mm and cover to reinforcement is 50 mm, effective depth, d, is

$$d = 400 - 50 - 20/2 = 340 \text{ mm}$$

$$K = \frac{160.5 \times 10^6}{35 \times 10^3 \times 340^2} = 0.0397$$

$$z = 340[0.5 + \sqrt{(0.25 - 0.0397/0.9)}] \leq 0.95d = 323 \text{ mm}$$

$$A_s = \frac{M}{0.87f_y z} = \frac{160.5 \times 10^6}{0.87 \times 500 \times 323} = 1142 \text{ mm}^2/\text{m}$$

Hence from *Table 3.22*, provide H20 at 200 mm centres (A_s = 1570 mm^2/m) in top face (T) of base.

Example 3.16 *continued*

Toe

Design moment at point *B*, M_B, is given by

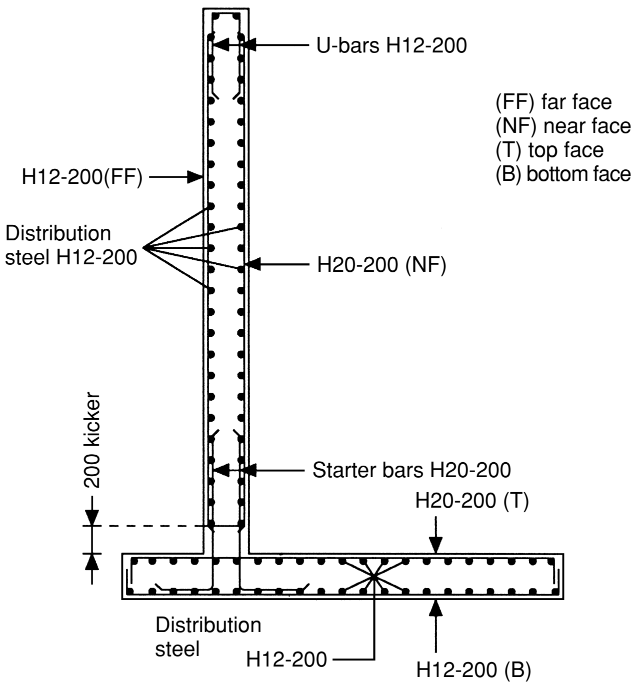
$$M_B \approx \frac{162.4 \times 0.7^2}{2} - \frac{0.7 \times 38.4 \times 1.4 \times 0.7}{4 \times 2} = 36.5 \text{ kNm}$$

$$A_s = \frac{36.5 \times 1142}{160.5} = 260 \text{ mm}^2/\text{m} < \text{minimum steel area} = 520 \text{ mm}^2/\text{m}$$

Hence provide H12 at 200 mm centres ($A_s = 566$ mm^2/m), in bottom face (B) of base and as distribution steel in base and stem of wall.

REINFORCEMENT DETAILS

The sketch below shows the main reinforcement requirements for the retaining wall. For reasons of buildability the actual reinforcement details may well be slightly different.



ASDIP Retain 3.0.0 **CANTILEVER RETAINING WALL DESIGN** *www.asdipsoft.com*

GEOMETRY

Conc. Stem Height	**5.00**	m	
Stem Thickness Top	**40.0**	cm	
Stem Thickness Bot	**40.0**	cm	
Footing Thickness	**40.0**	m	
Toe Length	**0.70**	m	
Heel Length	**2.90**	m	
Soil Cover @ Toe	**0.00**	m	
Backfill Height	**5.00**	m	OK
Backfill Slope Angle	**0.0**	deg	

SEISMIC EARTH FORCES

Hor. Seismic Coeff. kh	**0.00**	
Ver. Seismic Coeff kv	**0.00**	
Seismic Active Coeff. Kae	0.30	
Seismic Force Pae-Pa	-8.8	KN/m

SOIL BEARING PRESSURES

Allow. Bearing Pressure ..	**120.0**	KPa	
Max. Pressure @ Toe	115.0	KPa	OK
Min. Pressure @ Heel	65.1	KPa	
Total Footing Length	4.00	m	
Footing Length / 6	0.67	m	
Resultant Eccentricity e ...	0.18	m	

Resultant is Within the Middle Third

APPLIED LOADS

Uniform Surcharge	**0.0**	KPa
Strip Pressure	**0.0**	KPa

Strip 0.6 m deep, 1.2 m wide @ 0.9 m from Stem

Stem Vertical (Dead)	**0.0**	KN/m
Stem Vertical (Live)	**0.0**	KN/m
Vertical Load Eccentricity	**15.2**	cm
Wind Load on Stem	**0.0**	KPa

BACKFILL PROPERTIES

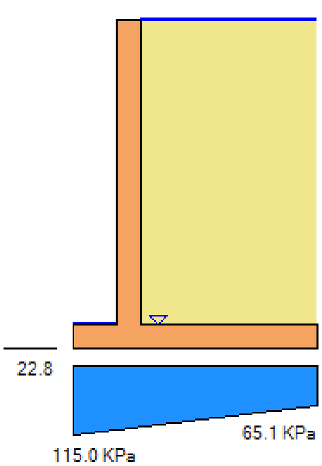
Backfill Density	**19.0**	KN/m³
Earth Pressure Theory	**Rankine Active**	
Internal Friction Angle	**30.0**	deg
Active Pressure Coeff. Ka	0.33	
Active Pressure @ Wall	6.3	KPa/m
Active Force @ Wall Pa	92.3	KN/m
Water Table Height	**0.00**	m

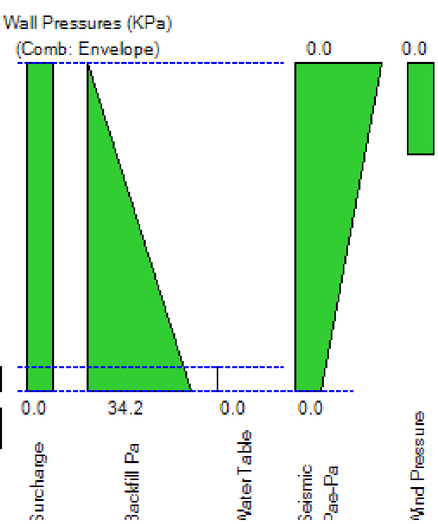
SHEAR KEY DESIGN

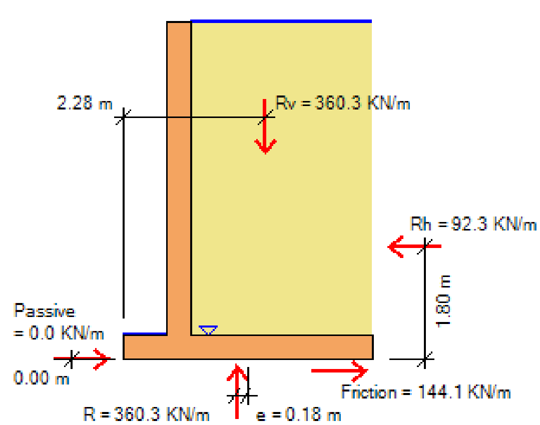
Shear Key Depth	**0.0**	cm
Shear Key Thickness	**0.0**	cm
Max. Shear Force @ Key ..	0.0	KN/m
Shear Capacity Ratio	0.00	OK

No shear key has been specified

Moment Capacity Ratio	0.00	OK







CANTILEVER RETAINING WALL DESIGN

OVERTURNING CALCULATIONS (Comb. D+H+W)

OVERTURNING	Force KN/m	Arm m	Moment KN-m/m
Backfill Pa	92.34	1.80	166.2
Water Table	0.00	0.13	0.0
Surcharge Hor	0.00	2.70	0.0
Strip Load Hor	0.00	2.50	0.0
Wind Load	0.00	4.64	0.0
Seismic Pae-Pa ...	0.00	3.24	0.0
Seismic Water	0.00	0.13	0.0
Seismic Selfweight	0.00	0.00	0.0
	Rh = 92.34		OTM = 166.2

RESISTING	Force KN/m	Arm m	Moment KN-m/m
Stem Top	47.12	0.90	42.4
Stem Taper	0.00	1.10	0.0
CMU Stem at Top ..	0.00	0.00	0.0
Footing Weight	37.70	2.00	75.4
Shear Key	0.00	0.70	0.0
Soil Cover @ Toe .	0.00	0.35	0.0
Stem Wedge	0.00	1.10	0.0
Backfill Weight	275.50	2.55	702.5
Backfill Slope	0.00	3.03	0.0
Water Weight	0.00	2.55	0.0
Seismic Pae-Pa	0.00	4.00	0.0
Pa Vert @ Heel	0.00	4.00	0.0
Vertical Load	0.00	0.95	0.0
Surcharge Ver	0.00	2.55	0.0
Strip Load Ver	0.00	2.55	0.0
	Rv = 360.32		RM = 820.3

Arm of Horizontal Resultant = $\frac{166.2}{92.34}$ = 1.80 m

Arm of Vertical Resultant = $\frac{820.3}{360.32}$ = 2.28 m

Overturning Safety Factor = $\frac{820.3}{166.2}$ = 4.94 > 2 OK

STEM DESIGN (Comb. 0.9D+1.6H+E)

Height m	d cm	Mu KN-m/m	ϕMn KN-m/m	Ratio	
5.00	35.5	0.0	0.0	0.00	
4.50	35.5	0.2	112.8	0.00	
4.00	35.5	1.7	123.1	0.01	
3.50	35.5	5.7	123.1	0.05	
3.00	35.5	13.5	123.1	0.11	
2.50	35.5	26.4	123.1	0.21	
2.00	35.5	45.6	151.4	0.30	
1.50	35.5	72.4	241.6	0.30	
1.00	35.5	108.1	241.6	0.45	
0.50	35.5	153.9	241.6	0.64	
0.00	35.5	211.1	241.6	0.87	OK

Shear Force @ Crit. Height .. 117.7 KN/m OK

Resisting Shear ϕVc .... 261.5 KN/m

Use vertical bars D20 @ 20 cm at backfill side

Cut off alternate bars. Cut off length = 2.13 m

Vert. Bars Embed. Ldh Reqd .. 28.4 cm OK

Vert. Bars Splice Length Ld .... 54.6 cm

SLIDING CALCS (Comb. D+H+W)

Footing-Soil Friction Coeff. ..	0.40	
Friction Force at Base	144.1	KN/m
Passive Pressure Coeff. Kp .	3.00	
Depth to Neglect Passive	0.40	m
Passive Pressure @ Wall	Infinity	KPa/m
Passive Force @ Wall Pp	0.0	KN/m
Horiz. Resisting Force	144.1	KN/m
Horiz. Sliding Force	92.3	KN/m

Sliding Safety Factor = $\frac{144.1}{92.3}$ = 1.56 > 1.5 OK

LOAD COMBINATIONS (ASCE 7)

STABILITY	STRENGTH
① D+H+W	① 1.4D
② D+L+H+W	② 1.2D+1.6(L+H)
③ D+H+0.7E	③ 1.2D+0.8W
④ D+L+H+0.7E	④ 1.2D+L+1.6W
	⑤ 1.2D+L+E
	⑥ 0.9D+1.6H+1.6W
	⑦ 0.9D+1.6H+E

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HEEL DESIGN (Comb. 0.9D+1.6H+E)

	Force KN/m	Arm m	Moment KN-m/m
Upward Pressure .	-182.4	1.08	197.3
Concrete Weight ..	24.6	1.45	35.7
Backfill Weight	248.0	1.45	359.5
Backfill Slope	0.0	1.93	0.0
Water Weight	0.0	1.45	0.0
Surcharge Ver.	0.0	1.45	0.0
Strip Load Ver.	0.0	1.45	0.0
	90.1	Mu =	197.9

Shear Force @ Crit. Sect. ..	93.7	KN/m	OK
Resisting Shear ϕVc	249.7	KN/m	

Use top bars D20 @ 20 cm , Transv. D12 @ 20 cm

Resisting Moment ϕMn	230.3	KN-m/m	OK
Develop. Length Ratio at End	0.19		OK
Develop. Length Ratio at Toe	0.52		OK

TOE DESIGN (Comb. 1.2D+1.6(L+H))

	Force KN/m	Arm m	Moment KN-m/m
Upward Presssure	107.4	0.36	38.4
Concrete Weight ..	-7.9	0.35	-2.8
Soil Cover	0.0	0.35	0.0
	99.4	Mu =	35.6

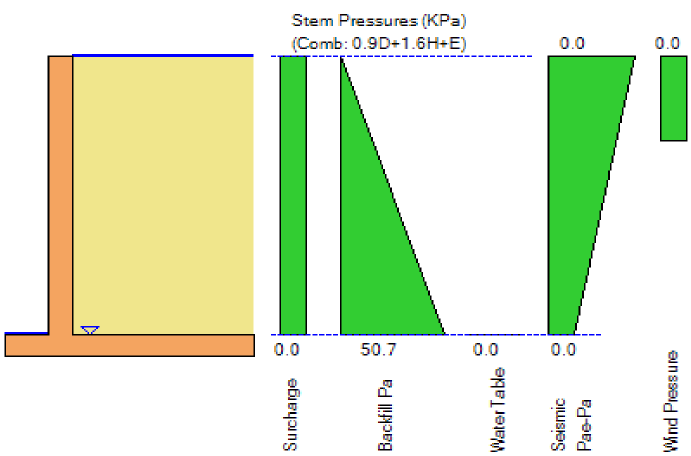
Shear Force @ Crit. Sect. ..	50.6	KN/m	OK
Resisting Shear ϕVc	253.4	KN/m	

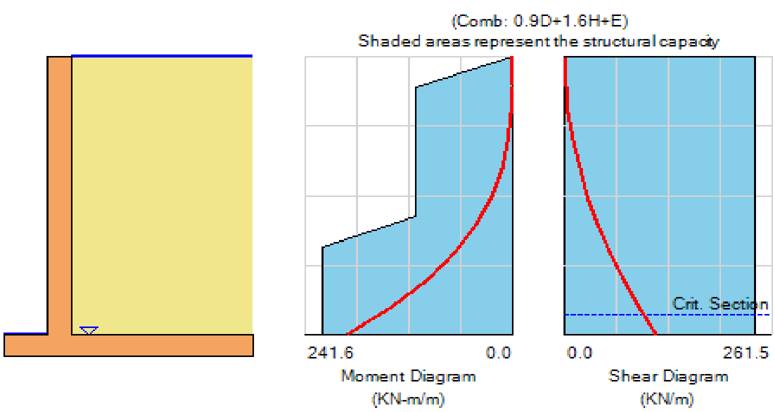
Use bott. bars D12 @ 20 cm , Transv. D12 @ 20 cm

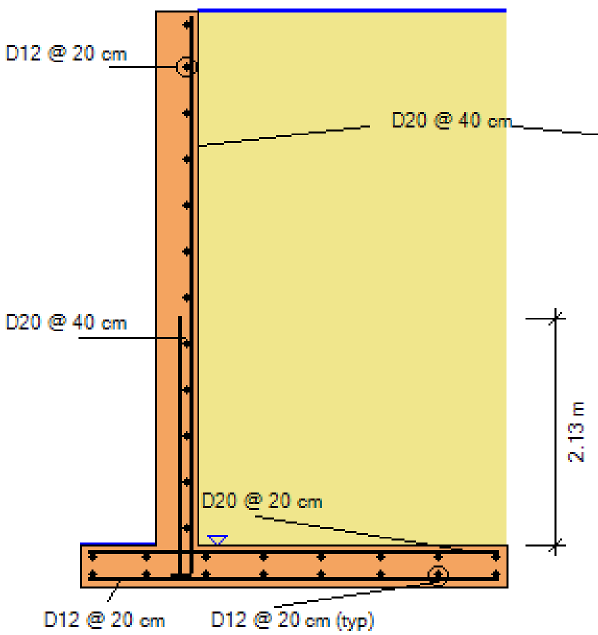
Resisting Moment ϕMn	86.2	KN-m/m	OK
Develop. Length Ratio at End	0.19		OK
Develop. Length Ratio at Stem	0.04		OK

MATERIALS

	Stem	Footing	
Concrete f'c	35.0	35.0	MPa
Rebars fy	500.0	500.0	MPa







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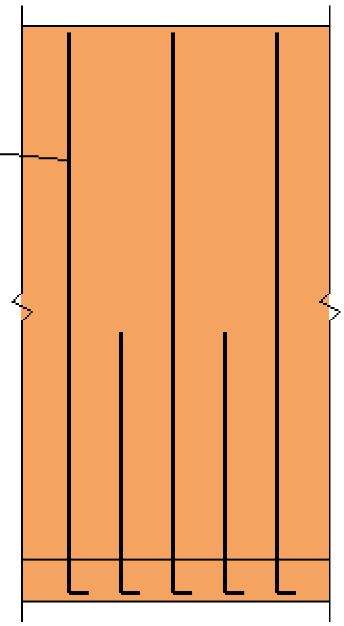

ELEVATION

DESIGN CODES

General Analysis	IBC-12
Concrete Design	ACI 318-11
Masonry Design	MSJC-11
Load Combinations	ASCE 7-05

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GEOMETRY

Conc. Stem Height	5.00	m	
Stem Thickness Top	40.0	cm	
Stem Thickness Bot	40.0	cm	
Footing Thickness	40.0	m	
Toe Length	0.70	m	
Heel Length	2.90	m	
Soil Cover @ Toe	0.00	m	
Backfill Height	5.00	m	OK
Backfill Slope Angle	0.0	deg	

APPLIED LOADS

Uniform Surcharge	0.0	KPa
Strip Pressure	0.0	KPa
Strip 0.6 m deep, 1.2 m wide @ 0.9 m from Stem		
Stem Vertical (Dead)	0.0	KN/m
Stem Vertical (Live)	0.0	KN/m
Vertical Load Eccentricity	15.2	cm
Wind Load on Stem	0.0	KPa
Wind Height from Top	1.52	m

BACKFILL PROPERTIES

Wall taper $\alpha = aTan\ (taper\ /\ H) =$ aTan ((40.0 - 40.0) / 100 / 5.00) = 0.000 rad

Backfill slope $\beta = slope * \pi / 180 =$ 0.0 * 3.14 / 180 = 0.000 rad

Internal friction $\phi = Int.\ friction * \pi / 180 =$ 30.0 * 3.14 / 180 = 0.524 rad

Wall-soil friction $\delta = \phi / 2 =$ 0.524 / 2 = 0.262 rad

Seismic angle $\theta = aTan\ (kh\ /\ (1 - kv)) =$ aTan (0 / (1 - 0)) = 0.000 rad

Footing length $ftg = toe + stem + heel =$ 0.70 + 40.0 / 100 + 2.90 = 4.00 m

Height for Stability $Hs = wedge + backfill + footing =$ 0.00 + 5.00 + 40.0 / 100 = 5.40 m

Earth pressure theory = Rankine Active Moist density = 19 KN/m³ Saturated density = 20 KN/m³

Active coefficient $ka = \dfrac{Cos\,\beta * [Cos\,\beta - (Cos^2\beta - Cos^2\phi)^{1/2}]^2}{Cos\,\beta + (Cos^2\beta - Cos^2\phi)^{1/2}}$ = 0.33

Active pressure $pa = ka * \gamma =$ 0.33 * 19.0 = 6.3 KPa/m of height

- For stability analysis (non-seismic)

Active force $Pa = ka * \gamma * Hs^2 / 2 =$ 0.33 * 19.0 * 5.40² / 2 = 92.3 KN/m

$Pah = Pa * Cos\,\beta =$ 92.3 * Cos (0.000) = 92.3 KN/m , $Pav = Pa * Sin\,\beta =$ 92.3 * Sin (0.000) = 0.0 KN/m

Water force $Pw = (ka * (\gamma s - \gamma w - \gamma) + \gamma w) * (water + footing)^2 / 2$

Pw = (0.33 * (20.4 - 9.8 - 19.0) + 9.8) * (0.00 + 40.0 / 100)² / 2 = 0.0 KN/m

- For stem design (non-seismic)

Active force $Pa = ka * \gamma * H^2 / 2 =$ 0.33 * 19.0 * 5.00² / 2 = 79.2 KN/m

$Pah = Pa * Cos\,\beta =$ 79.2 * Cos (0.000) = 79.2 KN/m , $Pav = Pa * Sin\,\beta =$ 79.2 * Sin (0.000) = 0.0 KN/m

Water force $Pw = (ka * (\gamma s - \gamma w - \gamma) + \gamma w) * (water\ table)^2 / 2$

Pw = (0.33 * (20.4 - 9.8 - 19.0) + 9.8) * 0.00² / 2 = 0.0 KN/m

Active seismic coeff. $kae = \dfrac{Cos^2(\phi - \alpha - \theta)}{Cos\theta * Cos^2\alpha * Cos(\delta + \alpha + \theta) * [1 + (\dfrac{Sin(\phi + \delta) * Sin(\phi - \beta - \theta)}{Cos(\delta + \alpha + \theta) * Cos(\beta - \alpha)})^{½}]^2}$ = 0.30

- For stability analysis (seismic)

Seismic force *Pae = kae * γ * Hs² / 2 * (1 - kv)* = 0.30 * 19.0 * 5.40² / 2 * (1 - 0.0) = 83.5 KN/m

*Paeh = Pae * Cos (δ + α)* = 83.5 * Cos (0.262 + 0.000) = 80.7 KN/m

*Paev = Pae * Sin (δ + α)* = 83.5 * Sin (0.262 + 0.000) = 21.6 KN/m

Water force *Pwe = kh * (γs - γ) * (water + footing)² / 2*

Pwe = 0.00 * (20.4 - 19.0) * (0.00 + 40.0 / 100)² / 2 = 0.0 KN/m

- For stem design (seismic)

Seismic force *Pae = kae * γ * H² / 2* = 0.30 * 19.0 * 5.00² / 2 = 71.6 KN/m

*Paeh = Pae * Cos (δ + α)* = 71.6 * Cos (0.262 + 0.000) = 69.1 KN/m

*Paev = Pae * Sin (δ + α)* = 71.6 * Sin (0.262 + 0.000) = 18.5 KN/m

Water force *Pwe = kh * (γs - γ) * (water table)² / 2*

Pwe = 0.00 * (20.4 - 19.0) * 0.00² / 2 = 0.0 KN/m

OVERTURNING CALCULATIONS (Comb. D+H+W)

- Overturning

Backfill = *Lat factor * Pah* = 1.0 * 92.3 = 92.3 KN/m

Arm = *Hs / 3* = 5.40 / 3 = 1.80 m Moment = 92.3 * 1.80 = 166.2 KN-m/m

Water table = *Lat factor * Pw* = 1.0 * 0.0 = 0.0 KN/m

Arm = *(Water table + Ftg) / 3* = (0.00 + 40.0 / 100) / 3 = 0.13 m Moment = 0.0 * 0.13 = 0.0 KN-m/m

Surcharge = *Lat factor * ka * Surcharge * Hs* = 1.0 * 0.33 * 0.0 * 5.40 = 0.0 KN/m

Arm = *Hs / 2* = 5.40 / 2 = 2.70 m Moment = 0.0 * 2.70 = 0.0 KN-m/m

Strip load = *Σ Lat factor * 2 * Q / π * [β - Sin β * Cos (2α)]* = 0.0 KN/m

Arm = 2.50 m Moment = 0.0 * 2.50 = 0.0 KN-m/m

Wind load = *WL factor * Pressure * Wind height* = 1.0 * 0.0 * 1.52 = 0.0 KN/m

Arm = *Ftg + Stem - Wind height / 2* = 40.0 / 100 + 5.00 - 1.52 / 2 = 4.64 m

Moment = 0.0 * 4.64 = 0.0 KN-m/m

Backfill seismic = *EQ factor * (Paeh - Pah)* = 0.0 * (80.7 - 80.7) = 0.0 KN/m

Arm = *0.6 * Hs* = 0.6 * 5.40 = 3.24 m Moment = 0.0 * 3.24 = 0.0 KN-m/m

Water seismic = *EQ factor * Pwe* = 0.0 * 0.0 = 0.0 KN/m

Arm = *(Water table + Ftg) / 3* = (0.00 + 40.0 / 100) / 3 = 0.13 m Moment = 0.0 * 0.13 = 0.0 KN-m/m

Wall seismic = *EQ factor * (WStem + WTaper + WFtg) * kh* = 0.0 * (0.0 + 0.0 + 37.7) * 0.00 = 0.0 KN/m

Moment = *EQ factor * (WStem * (Ftg + Stem / 2) + WTaper * (Ftg + Stem / 3) + WFtg * Ftg / 2) * kh* =

= 0.0 * (0.0 * (40.0 / 100 + 5.00 / 2) + 0.0 * (40.0 / 100 + 5.00 / 3) + 37.7 * 40.0 / 100 / 2) * 0.00 = 0.0 KN-m/m

Hor. resultant Rh = 92.3 + 0.0 + 0.0 + 0.0 + 0.0 + 0.0 + 0.0 + 0.0 = 92.3 KN/m

Overturning moment OTM = 166.2 + 0.0 + 0.0 + 0.0 + 0.0 + 0.0 + 0.0 + 0.0 = 166.2 KN-m/m

Arm of hor. resultant = *OTM / Rh* = 166.2 / 92.3 = 1.80 m

- Resisting

Stem weight *WStem = DL factor * Thickness * Height * γc =* 1.0 * 40.0 / 100 * 5.00 * 23.56 = 47.1 KN/m
Arm = *Toe + Thickness / 2 =* 0.70 + 40.0 / 100 / 2 = 0.90 m Moment = 47.1 * 0.90 = 42.4 KN-m/m
Stem taper *WTaper = DL factor * ΔThick * Height / 2 * γc =* 1.0 * (40.0 - 40.0) / 100 * 5.00 / 2 * 23.56 = 0.0 KN/m
Arm = *Toe + Thick + ΔThick / 3 =* 0.70 + 40.0 / 100 - (40.0 - 40.0) / 100 * 2 / 3 = 1.10 m
Moment = 0.0 * 1.10 = 0.0 KN-m/m
CMU stem at top = 0.0 KN/m
Arm = *Toe + Thickness / 2 =* 0.70 + 0.0 / 100 / 2 = 0.00 m
Moment = 0.0 * 0.00 = 0.0 KN-m/m
Ftg. weight *WFtg = DL factor * Length * Thickness * γc =* 1.0 * 4.00 * 40.0 / 100 * 23.56 = 37.7 KN/m
Arm = *Length / 2 =* 4.00 / 2 = 2.00 m Moment = 37.7 * 2.00 = 75.4 KN-m/m
Key weight *WKey = DL factor * Depth * Thickness * γc =* 1.0 * 0.00 / 100 * 0.0 / 100 * 23.56 = 0.0 KN/m
Arm = *Toe + Thickness / 2 =* 0.70 + 0.0 / 100 / 2 = 0.70 m Moment = 0.0 * 0.70 = 0.0 KN-m/m
Soil cover = *DL factor * Toe * Soil cover * γ =* 1.0 * 0.70 * 0.00 * 19.0 = 0.0 KN/m
Arm = *Toe / 2 =* 0.70 / 2 = 0.35 m Moment = 0.0 * 0.35 = 0.0 KN-m/m
Stem wedge = *DL factor * ΔThick * Height / 2 * γ =* 1.0 * (40.0 - 40.0) / 100 * 5.00 / 2 * 19.0 = 0.0 KN/m
Arm = *Toe + Thick - ΔThick / 3 =* 0.70 + 40.0 / 100 - (40.0 - 40.0) / 100 / 3 = 1.10 m
Moment = 0.0 * 1.10 = 0.0 KN-m/m
Backfill weight = *DL factor * Heel * Height * γ =* 1.0 * 2.90 * 5.00 * 19.0 = 275.5 KN/m
Arm = *Ftg - Heel / 2 =* 4.00 - 2.90 / 2 = 2.55 m Moment = 275.5 * 2.55 = 702.5 KN-m/m
Backfill slope = *DL factor * (Heel + ΔThick) * Wedge / 2 * γ =*
= 1.0 * (2.9 + (40.0 - 40.0) / 100) * 0.00 / 2 * 19.0 = 0.0 KN/m
Arm = *ftg - (Heel + ΔThick) / 3 =* 4.00 - (2.90 + (40.0 - 40.0) / 100) / 3 = 3.03 m
Moment = 0.0 * 3.03 = 0.0 KN-m/m
Water = *DL factor * Heel * Water table * (γs - γ) =* 1.0 * 2.90 * 0.00 * (20.4 - 19.0) = 0.0 KN/m
Arm = *Ftg - Heel / 2 =* 4.00 - 2.90 / 2 = 2.55 m Moment = 0.0 * 2.55 = 0.0 KN-m/m
Seismic Pae-Pa = *EQ factor * (Paev - Pav) =* 0.0 * (21.6 - 21.6) = 0.0 KN/m
Arm = *Footing length =* 4.00 m Moment = 0.0 * 4.00 = 0.0 KN-m/m
Backfill Pav = *Lat factor * Pav =* 1.0 * 21.6 = 0.0 KN/m
Arm = *Footing length =* 4.00 m Moment = 0.0 * 4.00 = 0.0 KN-m/m
Concentrated = *DL factor * Ver load + LL factor * Ver load =* 1.0 * 0.0 + 0.0 * 0.0 = 0.0 KN/m
Arm = *Toe + Stem - Ecc =* 0.70 + (40.0 - 15.2) / 100 = 0.95 m
Moment = 0.0 * 0.95 = 0.0 KN-m/m

Surcharge = *Srch factor * (Heel + ΔThick) * Surcharge* =1.0 * (2.9 + (40.0 - 40.0) / 100) * 0.0 = 0.0 KN/m

Arm = *ftg - (Heel + ΔThick) / 2* = 4.00 - (2.90 + (40.0 - 40.0) / 100) / 2 = 2.55 m

Moment =0.0 * 2.55 = 0.0 KN-m/m

Strip = *Strip factor * Surcharge * Heel* = 1.0 * 0.0 * 2.90 = 0.0 KN/m

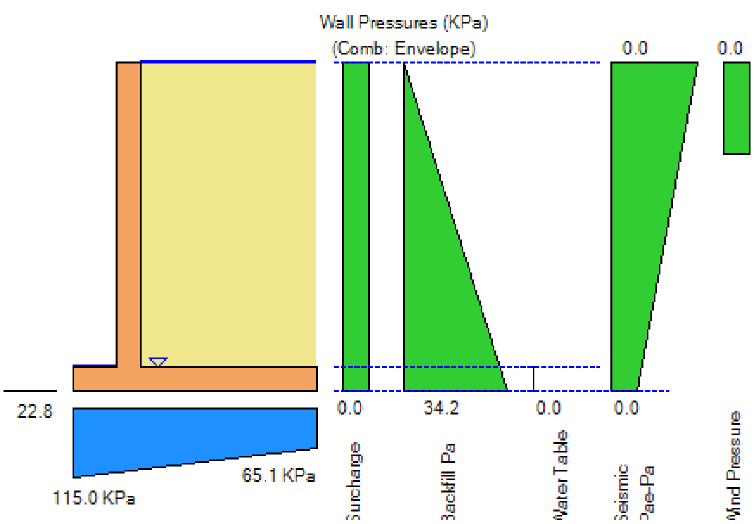
Arm = *Footing - Heel / 2* = 4.00 - 2.90 / 2 = 2.55 m   Moment =0.0 * 2.55 = 0.0 KN-m/m

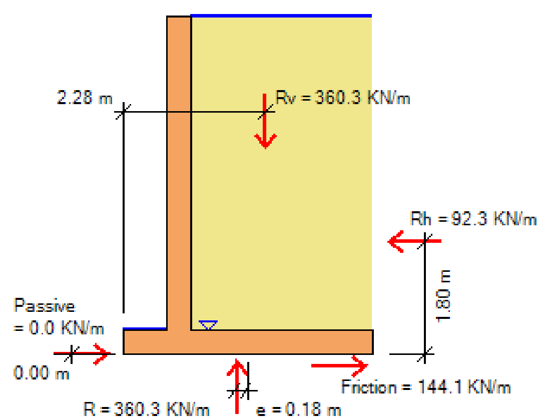
Ver. resultant Rv = *Σ Vertical forces* = 360.3 KN/m

Resisting moment RM = *Σ Moments* = 820.3 KN-m/m

Arm of ver. resultant = *RM / Rv* = 820.3 / 360.3 = 2.28 m

Overturning ratio = *RM / OTM* = 820.3 / 166.2 = 4.94 > 2.00 OK





SOIL BEARING PRESSURES (Comb. D+H+W)

Eccentricity = $\frac{Ftg}{2} - \frac{RM - OTM}{Rv} = \frac{4.00}{2} - \frac{820.3 - 166.2}{360.3} = 0.18$ m

Bearing length = *Min (Ftg , 3 * (Ftg / 2 - Ecc))* = Min (4.00, 3 * (4.00 / 2 - 0.18)) = 4.00 m

Toe bearing = $\frac{Rv}{Ftg} + \frac{6 * Rv * Ecc}{Ftg^2} = \frac{360.3}{4.00} + \frac{6 * 360.3 * 0.18}{4.00^2} = 115.0$ KPa < 120.0 KPa OK

Heel bearing = $\frac{Rv}{Ftg} - \frac{6 * Rv * Ecc}{Ftg^2} = \frac{360.3}{4.00} - \frac{6 * 360.3 * 0.18}{4.00^2} = 65.1$ KPa

SLIDING CALCULATIONS (Comb. D+H+W)

Passive coefficient *kp = 1 / ka* = 1 / 0.33 = 3.00 KPa

Passive depth *Dp = Soil cover + Ftg + Key - Neglect depth* =0.00 + (40.0 + 0.0) / 100 - 0.40 = 0.00 m

Passive pressure top = *kp * γ * Neglect depth* = 3.00 * 19.0 * 0.40 = 22.80 KPa

Passive pressure bot = *kp * γ * (Dp + Neglect depth)* =3.00 * 19.0 * (0.00 + 0.40) = 22.80 KPa

Passive force = *(Pressure top + Pressure bot) / 2 * Dp* =(22.80 + 22.80) / 2 * 0.00 = 0.0 KN/m

Friction force = *Max (0, Rv * Friction coeff.) =* Max (0, 360.3 * 0.40) = 144.1 KN/m

Sliding ratio = *(Passive + Friction) / Rh* = (0.0 + 144.1) / 92.3 = 1.56 > 1.50 OK

STEM DESIGN (Comb. 0.9D+1.6H+E)

Backfill = *Lat factor * Pah =* 1.6 * 78.7 = 126.7 KN/m

Arm = *Hb / 3 =* 5.00 / 3 = 1.67 m Moment = 126.7 * 1.67 = 211.1 KN-m/m

Water table = *Lat factor * Pw* =1.6 * 0.0 = 0.0 KN/m

Arm = *Water table / 3 =* 0.00 / 3 = 0.00 m Moment = 0.0 * 0.00 = 0.0 KN-m/m

Surcharge = *Lat factor * ka * Surcharge * Hb* =1.6 * 0.33 * 0.0 * 5.00 = 0.0 KN/m

Arm = *Hb / 2 =* 5.00 / 2 = 2.50 m Moment = 0.0 * 2.50 = 0.0 KN-m/m

Strip load = *Σ Lat factor * 2 * Q / π * [β - Sin β * Cos (2α)] =* 0.0 KN/m

Arm = 2.50 m Moment = 0.0 * 2.50 = 0.0 KN-m/m

Wind load = *WL factor * Pressure * Wind height* =0.0 * 0.0 * 1.52 = 0.0 KN/m

Arm = *Stem - Wind height / 2* =5.00 - 1.52 / 2 = 4.24 m Moment =0.0 * 4.24 = 0.0 KN-m/m

Backfill seismic = *EQ factor * (Paeh - Pah)* =1.0 * (69.1 - 69.1) = 0.0 KN/m

Arm = *0.6 * Hb* = 0.6 * 5.00 = 3.00 m Moment = 0.0 * 3.00 = 0.0 KN-m/m

Water seismic = *EQ factor * Pwe* = 1.0 * 0.0 = 0.0 KN/m

Arm = *Water table / 3* =0.00 / 3 = 0.00 m Moment = 0.0 * 0.00 = 0.0 KN-m/m

Max. shear = 126.7 + 0.0 + 0.0 + 0.0 + 0.0 + 0.0 + 0.0 = 126.7 KN/m

Shear at critical section = *Max shear - Max shear / Hb * d* =126.7 - 126.7 / 5.00 * 35.5 / 100 = 117.7 KN/m

Max. moment = 211.1 + 0.0 + 0.0 + 0.0 + 0.0 + 0.0 + 0.0 = 211.1 KN-m/m

Shear strength *φ Vn = φ * 0.17 * (f'c)½ * 10 * d* ACI Eq. (11-3)

φ Vn = 0.75 * 0.17 * (35)½ * 10 * 35.5 = 261.5 KN/m > 117.7 KN/m OK

Use D20 @ 20.0 cm As = 15.71 cm²/m *ρ = As / b d =* 15.71 / (100 * 35.5) = 0.0044

Bending strength *φ Mn = φ * d² * f'c * q * (1 - 0.59 * q)* ACI 10.2.7

φ Mn = 0.90 * 35.5² * 35.0 * 0.063 * (1 - 0.59 * 0.063) = 241.6 KN-m/m > 211.1 KN-m/m OK

Hooked *Ldh = 0.24 * fy / (f'c)½ * db * 0.7* =0.24 * 500.0 / (35.0)½ * 2.00 * 0.7 = 28.4 cm ACI 12.5

Dev. length at footing = *Ftg - Cover* = 40.0 - 5.0 = 35.0 cm > 28.4 cm OK

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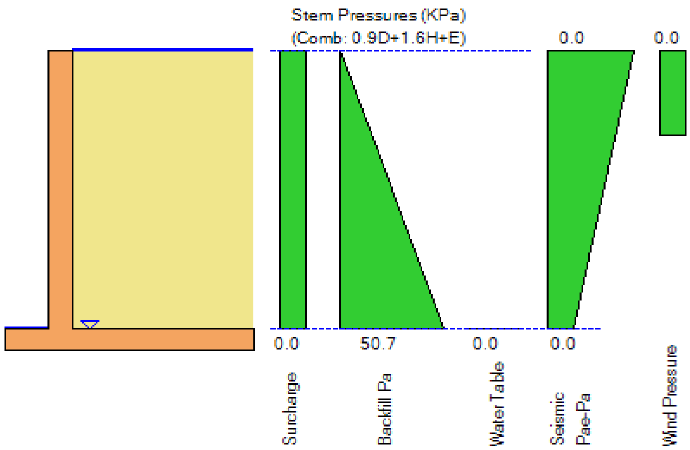
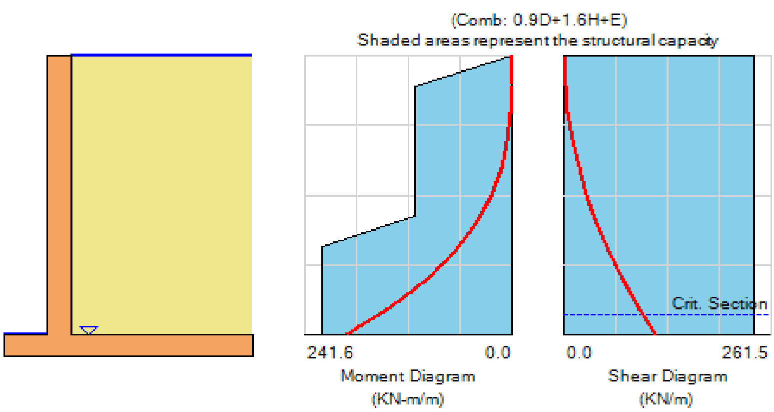




HEEL DESIGN (Comb. 0.9D+1.6H+E)

Bearing force = *(Bearing1 + Bearing2) / 2 * Heel* = (15.0 + 110.8) / 2 * 2.90 = 182.4 KN/m

Arm = *(Bearing1 * Heel² / 2 + (Bearing2 - Bearing1) * Heel² / 6) / Force*

= (15.0 * 2.90² / 2 + (110.8 - 15.0) * 2.90² / 6) / 182.4 = 1.08 m

Moment = 182.4 * 1.08 = 197.3 KN-m/m

Concrete weight = *DL factor * Thick * Heel * γc* = 0.9 * 40.0 / 100 * 2.90 * 23.56 = 24.6 KN/m

Arm = *Heel / 2* = 2.90 / 2 = 1.45 m Moment = 24.6 * 1.45 = 35.7 KN-m/m

Backfill weight = *DL factor * Heel * Height * γ* = 0.9 * 2.90 * 5.00 * 19.0 = 248.0 KN/m

Arm = *Heel / 2* = 2.90 / 2 = 1.45 m Moment = 248.0 * 1.45 = 359.5 KN-m/m

Backfill slope = *DL factor * (Heel + ΔThick) * Wedge / 2 * γ* =

= 0.9 * (2.9 + (40.0 - 40.0) / 100) * 0.00 / 2 * 19.0 = 0.0 KN/m

Arm = *Heel * 2 / 3* = 2.90 * 2 / 3 = 1.93 m Moment = 0.0 * 1.93 = 0.0 KN-m/m

Water = *DL factor * Heel * Water table * (γs - γ)* = 0.9 * 2.90 * 0.00 * (20.4 - 19.0) = 0.0 KN/m

Arm = *Heel / 2* = 2.90 / 2 = 1.45 m Moment = 0.0 * 1.45 = 0.0 KN-m/m

Surcharge = *Srch factor * (Heel + ΔThick) * Surcharge* =0.9 * (2.9 + (40.0 - 40.0) / 100) * 0.0 = 0.0 KN/m

Arm = *Heel / 2* = 2.90 / 2 = 1.45 m Moment =0.0 * 1.45 = 0.0 KN-m/m

Strip = *Strip factor * Surcharge * Width* =0.9 * 0.0 * 1.22 = 0.0 KN/m

Arm = *Distance - ΔStem + Width / 2* = 0.91 - (40.0 - 40.0) / 100 + 1.22 / 2 = 1.45 m

Moment = 0.0 * 1.45 = 0.0 KN-m/m

Max. Shear Vu = -182.4 + 24.6 + 248.0 + 0.0 + 0.0 + 0.0 + 0.0 = 90.1 KN/m

Max. Moment Mu =-197.3 + 35.7 + 359.5 + 0.0 + 0.0 + 0.0 + 0.0 = 197.9 KN/m

Shear strength *φ Vn = φ * 0.17 * (f'c)½ * 10 * d* ACI Eq. (11-3)

φ Vn = 0.75 * 0.17 * (35)½ * 10 * 33.9 = 249.7 KN/m > Vu = 90.1 KN/m OK

Use D20 @ 20.0 cm As = 15.71 cm^2/m *ρ = As / b d* = 15.71 / (100 * 33.9) = 0.0046

Bending strength *φ Mn = φ * d² * f'c * q * (1 - 0.59 * q)* ACI 10.2.7

φ Mn = 0.90 * 33.9^2 * 35.0 * 0.066 * (1 - 0.59 * 0.066) = 230.3 KN-m/m > Mu = 197.9 KN-m/m OK

Cover factor = *Min (2.5, (Cover + db /2, Spacing / 2) / db)* = Min (2.5, (5.1 + 2.00 / 2, 20.0 / 2) / 2.00) = 2.5

Straight *Ld = fy / 1.1 / (f'c)½ * Size * Location / Cover * db* ACI Eq. (12-1)

= 500.0 / 1.1 / (35)½ * 0.8 * 1.3 / 2.5 * 2.00 = 63.9 cm

Hooked *Ldh = 0.24 * fy / (f'c)½ * db * 0.7* =0.24 * 500.0 / (35.0)½ * 2.00 * 0.7 = 28.4 cm ACI 12.5

Dev. length at toe side = *Ftg - Heel - Cover* = (4.00 - 2.90) / 100 - 5.1 = 104.9 cm > 63.9 cm OK

Dev. length at heel side = *Heel - Cover* = 2.90 / 100 - 5.1 = 284.9 cm > 63.9 cm OK

TOE DESIGN (Comb. 1.2D+1.6(L+H))

Bearing force = *(Bearing1 + Bearing2) / 2 * Toe* = (163.0 + 143.8) / 2 * 0.70 = 107.4 KN/m

Arm = *(Bearing1 * Toe² / 2 + (Bearing2 - Bearing1) * Toe² / 3) / Force*

= (143.8 * 0.70^2 / 2 + (163.0 - 143.8) * 0.70^2 / 3) / 107.4 = 0.36 m

Moment = 107.4 * 0.36 = 38.4 KN-m/m

Concrete weight = *DL factor * Thick * Toe * γc* = 1.2 * 40.0 / 100 * 0.70 * 23.56 = 7.9 KN/m

Arm = *Toe / 2* = 0.70 / 2 = 0.35 m Moment =7.9 * 0.35 = 2.8 KN-m/m

Soil cover = *DL factor * Toe * Height * γ* = 1.2 * 0.70 * 0.00 * 19.0 = 0.0 KN/m

Arm = *Toe / 2* = 0.70 / 2 = 0.35 m Moment = 0.0 * 0.35 = 0.0 KN-m/m

Max. Shear Vu = 107.4 - 7.9 - 0.0 = 99.4 KN/m

Shear at crit. section Vu =*Max shear * (Toe - d) / Toe* = 99.4 * (0.70 - 34.4 / 100) / 0.70 = 50.6 KN/m

Max. Moment Mu =38.4 - 2.8 - 0.0 = 35.6 KN/m

Shear strength *φ Vn = φ * 0.17 * (f'c)½ * 10 * d* ACI Eq. (11-3)

φ Vn = 0.75 * 0.17 * (35)½ * 10 * 34.4 = 253.4 KN/m > Vu = 50.6 KN/m OK

Use D12 @ 20.0 cm As = 5.65 cm^2/m *ρ = As / b d* = 5.65 / (100 * 34.4) = 0.0016

Bending strength *φ Mn = φ * d² * f'c * q * (1 - 0.59 * q)* ACI 10.2.7

φ Mn = 0.90 * 34.4^2 * 35.0 * 0.023 * (1 - 0.59 * 0.023) = 86.2 KN-m/m > Mu = 35.6 KN-m/m OK

Cover factor = *Min (2.5, (Cover + db /2, Spacing / 2) / db)* = Min (2.5, (5.0 + 1.20 / 2, 20.0 / 2) / 1.20) = 2.5

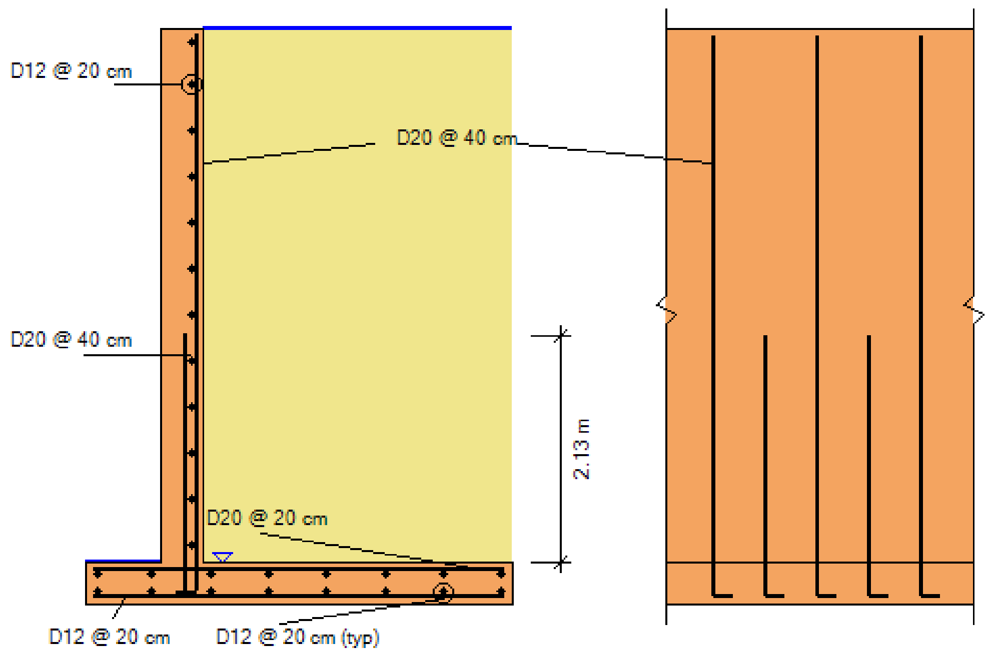
Straight *Ld = fy / 1.1 / (f'c)½ * Size * Location / Cover * db* ACI Eq. (12-1)

= 500.0 / 1.1 / (35)½ * 0.8 * 1.0 / 2.5 * 1.20 = 29.5 cm

Hooked *Ldh = 0.24 * fy / (f'c)½ * db * 0.7* = 0.24 * 500.0 / (35.0)½ * 1.20 * 0.7 = 17.0 cm ACI 12.5

Dev. length at toe side = *Ftg - Toe - Cover* = (4.00 - 0.70) / 100 - 5.0 = 325.0 cm > 29.5 cm OK

Dev. length at toe side = *Toe - Cover* = 0.70 / 100 - 5.0 = 65.0 cm > 29.5 cm OK



SECTION ELEVATION

SHEAR KEY DESIGN (Comb. 0.9D+1.6H+E)

Shear key depth = 0.0 cm Shear key thickness = 0.0 cm

Passive force = *Lat factor * (Passive1 + Passive2) / 2 * Key* = 1.6 * (22.8 + 22.8) / 2 * 0.0 / 100 = 0.0 KN/m

Shear at crit. section Vu = *Max shear * (Key - d) / Key* = 0.0 * (0.0 - 0.1) / 0.0 = 0.0 KN/m

Arm = *(Passive1 * Key² / 2 + (Passive2 - Passive1) * Key² / 3) / Force*

= (22.8 * 0.00² / 2 + (22.8 - 22.8) * 0.00² / 3) / 0.0 = 0.00 m

Max. moment Mu = 0.0 * 0.00 = 0.0 KN-m/m

Shear strength *φ Vn = φ * 0.17 * (f'c)½ * 10 * d* ACI Eq. (11-3)

φ Vn = 0.75 * 0.17 * (35)½ * 10 * 0.1 = 0.7 KN/m > Vu = 0.0 KN/m OK

Use #4 @ 30.5 cm As = 4.23 cm²/m *ρ = As / b d* = 4.23 / (100 * 0.1) = 0.4231

Bending strength *φ Mn = φ * d² * f'c * q * (1 - 0.59 * q)* ACI 10.2.7

φ Mn = 0.90 * 0.1² * 35.0 * 6.044 * (1 - 0.59 * 6.044) = 0.1 KN-m/m > Mu = 0.0 KN-m/m OK

UNIT 13 BUNKERS AND SILOS

Structure

13.1 INTRODUCTION

In the previous units, you have studied gantry girders, plate girder bridges, truss bridges, bearings. In this unit, you are going to learn the theory and design related to bunkers and silos.

The bunkers are large size shallow bins to store grains, coal and cement. In bunkers, the plane of rupture intersects the free surface of the stored material. Generally, steel bunkers are used to store coal at power plants and loco-running sheds. Generally, these are square or rectangular shaped.

The silos are the deep bins for storage. They are circular in shape. The plane of rupture intersects the opposite side of the container.

Objectives

After studying this unit, you should be able to

- understand Airy's theory,
- understand Janssen's theory,
- know the components of bunkers and silos,
- design the bunkers, and
- design the silos.

13.2 COMPONENTS OF BUNKERS

The sectional elevation and plan of the bunker are shown Figure 13.1.

1) Main girder,
2) Cross girder,
3) Beam,
4) Sloping plates,
5) Stiffeners, and
6) Openings.

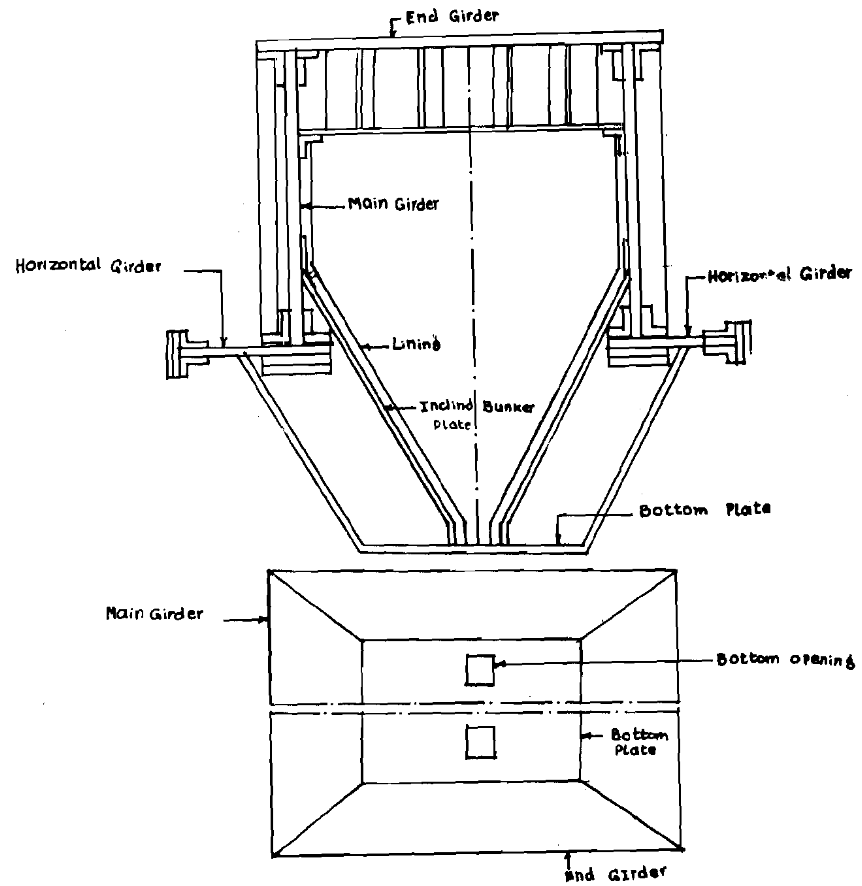


Figure 13.1

Main Girders : The main girders are provided parallel to the longitudinal sides. These are supported on cross girders.

Cross Girders : These are provided parallel to the width.

Sloping Plates : These are provided in the bottom portion of the bunker. The inclination is more than the angle of repose of the material for self cleaning.

Openings : These are provided at the bottom of the bunkers. The size is 500 mm square.

Stiffeners : These are provided with the inclined plates. At top, there are connected with the main girder. At bottom, these are connected with the bottom plates.

SAQ 1

1) What is a bunker?
2) What is a silo?
3) At what places, the steel bunkers are used?
4) Differentiate between a bunker and a silo?
5) What are the components of a bunker?
6) Write a note on openings in bunkers.
7) Write a note on stiffeners used in bunkers.

13.3 AIRY'S THEORY

By using this theory the horizontal pressure per unit length of periphery and position of plane of rupture can be determined. The Airy's theory is actually based on Coulomb's wedge theory of Earth Pressure.

Consider a wedge ABC of unit thickness.

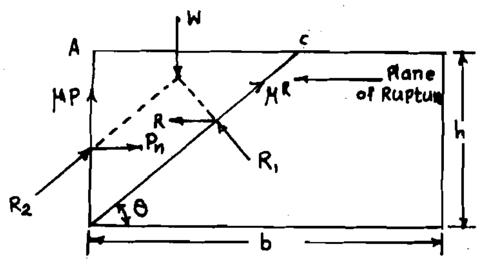


Figure 13.2

Let θ = inclination of plane of rupture with horizontal thickness of wedge

ABC = 1 unit

W = weight of wedge

R_1 & R_2 = reactions to BC and BA respectively

R_n and P_n = normal reactions due to R_1 & R_2 respectively

h = depth of Bunker

b = breadth of Bunker

Now $W = \frac{1}{2} \times AC \,.\, AB \,.\, w$

$$W = \frac{1}{2} \times h \times \cot\theta \,.\, h \,.\, w = \frac{wh^2}{2} \cot\theta \qquad ...(13.1)$$

Resolving forces acting on wedge in vertical direction

$$W = \mu^1 P_n + \mu R_n \sin\theta + R_n \cos\theta$$

$$R_n = (w - \mu^1 P_{n)}/(\mu \sin\theta + \cos\theta) \qquad ...(13.2)$$

Resolving forces on wedge in horizontal direction

$$P_h = \mu R_n \cos\theta = R_n \sin\theta$$

$$R_n = \frac{P_h}{\sin\theta - \mu\cos\theta} \qquad ...(13.3)$$

From (13.2) and (13.3)

$$\frac{w - \mu^1 P_h}{\mu\sin\theta + \cos\theta} = \frac{P_n}{\sin\theta - \mu\cos\theta}$$

$$P_n = \frac{w(\sin\theta - \mu\cos\theta)}{(\mu + \mu^1)\sin\theta + (1 - \mu\mu^1)\cos\theta} \qquad ...(13.4)$$

Substituting value of w and simplifying

$$P_h = \frac{wh^2}{2} \times \frac{(\tan\theta - \mu)}{(\mu + \mu^1)\tan^2\theta + (1 - \mu\mu^1)\tan\theta} \qquad ...(13.5)$$

or $$P_h = \frac{wh^2}{2} \times \frac{(\tan\theta - \mu)}{(\mu + \mu^1)\tan^2\theta + (1 - \mu\mu^1)\tan\theta}$$

or $$P_h = \frac{wh^2}{2}\,\frac{u}{v} \qquad \text{where, } u = \tan\theta - \mu$$

$$v = (\mu + \mu^1)\tan^2\theta + (1 - \mu\mu^1)\tan\theta$$

For maximum value of P_h

$$\frac{dP_h}{d\theta} = 0$$

$$\therefore \quad \frac{dP_h}{d\theta} = \frac{wh^2}{2}\left(\frac{udv - vdu}{v^2}\right) = 0$$

$$\therefore \quad \frac{u}{v} = \frac{du}{dv}$$

$$\therefore \quad \frac{\tan\theta - \mu}{(\mu + \mu^1)\tan^2\theta + (1 - \mu\mu^1)\tan\theta} = \frac{\sec^2\theta}{2(\mu + \mu^1)\tan\theta\sec^2\theta + (1 - \mu\mu^1)\sec^2\theta}$$

On further simplification

$$\tan^2\theta - 2\mu\tan\theta = \frac{\mu(1 - \mu\mu^1)}{(\mu + \mu^1)}$$

$$\therefore \quad \tan\theta = \mu + \sqrt{\frac{\mu(1 + \mu^2)}{(\mu + \mu^1)}}$$

Substituting the value of tan θ in Eqn. (13.5) and simplifying

$$P_h = \frac{wh^2}{2}\left[\frac{1}{\sqrt{1 + \mu^2} + \sqrt{\mu(\mu + \mu^1)}}\right]^2$$

Ph represents total horizontal force per unit length of the wall at depth *h*. The pressure per unit area

$$p_h = \frac{dP_n}{dh} = wh\left[\frac{1}{\sqrt{(1 + \mu)^2} + \sqrt{\mu(\mu + \mu^1)}}\right]^2 \qquad \text{...(13.6)}$$

substituting $\mu = \tan\phi$ and $\mu^1 = \tan\phi^1$ in Eqn. (13.6) we have

$$P_h = wh\left[\frac{\cos\phi}{1 + \sqrt{\sin\phi\sec\phi^1\sin(\phi + \phi^1)}}\right]^2$$

If P_w = vertical load carried by wall

$$P_w = \mu^1 P_h \;\; P_w = \mu^1 P_w$$

Total load carried by the wall will be perimeterlines *Pw*. The maximum depth upto which the shallow bin acts as a bunker may be found as follows

$$\frac{h_{\max}}{b} = \tan\theta = \mu + \sqrt{\frac{(\mu(1 + \mu^2)}{(\mu + \mu^1)}}$$

or $$h_{max} = b\left[\mu + \sqrt{\frac{\mu(1+\mu^2)}{(\mu+\mu^1)}}\right] \quad ...(13.7)$$

Note: Eqn. (13.6) is applicable for maximum depth h = hmax as in Eqn. (13.7) when depth of bin 'h' is greater than hmax, the bin becomes a deep bin (silo) plane of rupture intersects opposite wall at C in case of a silo

$\therefore \quad CD = (h - b\tan\theta)$

$\therefore \quad W = wbh - \frac{1}{2}wb.b\tan\theta$

or $$W = wb\left[h - \frac{1}{2}b\tan\theta\right] \quad ...(i)$$

Similar to shallow bin from Eqn. (i)

$$P_h = \frac{w(\sin\theta - \mu\cos\theta)}{(\mu+\mu^1)\sin\theta + (1-\mu\mu^1)\cos\theta}$$

Substituting the value of w and simplifying

$$P_h = \frac{wb\left[h - \frac{b\tan\theta}{2}\right](\tan\theta - \mu)}{(\mu+\mu^1)\tan\theta + (1-\mu\mu^1)}$$

or $$P_h = \frac{wb\left[\left(h + \frac{b\mu}{2}\right)\tan\theta - \frac{b}{2}\tan^2\theta - h\mu\right]}{(\mu+\mu^1)\tan\theta + (1-\mu\mu^1)}$$

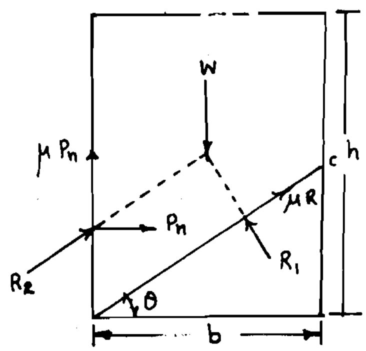


Figure 13.3

For maximum value of $Ph\frac{dPn}{d\theta} = 0$

$$\therefore \quad \frac{\left[\left(h + \frac{b\mu}{2}\right)\tan\theta - \frac{b}{2}\tan^2\theta - h\mu\right]}{(\mu+\mu^1)\tan\theta + (1-\mu\mu^1)} = \frac{\left(h + \frac{b\mu}{2}\right)\sec^2\theta - b\tan\theta\sec^2\theta}{(\mu+\mu^1)\sec^2\theta}$$

On further simplification

$$\tan^2\theta + \frac{2(1-\mu\mu^1)}{\mu+\mu^1}\tan\theta - \frac{2h(1+\mu^2) + b\mu(1-\mu\mu^1)}{b(\mu+\mu^1)} = 0$$

$$\therefore \quad \tan\theta = \frac{1}{\mu+\mu^1}\left[-(1-\mu\mu^1) + \sqrt{(1-\mu\mu^1)(1+\mu^2) + \frac{2h}{b}(1+\mu^2)}\;(\mu+\mu^1)\right]$$

Substituting the value of $\tan\theta$ and simplifying

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

The pressure P_h is given by $\frac{dP_h}{dh}$

$$\therefore \quad P_h = \frac{wb}{\mu+\mu^1} = \left[1 - \frac{\sqrt{1+\mu^2}}{\sqrt{\frac{2h}{b}(\mu+\mu^1) + (1-\mu\mu^1)}}\right]$$

Note: b may be taken as length of side adjacent to the wall on which the pressure is to be determined for a rectangular silo. The vertical taken by wall is $P_w = \mu^1 P_h$. The total vertical load is perimeter $\times P_w$.

Values of μ and μ^1 for same comon materials have been given in Table 13.1.

Table 13.1

S.No.	Materials	μ	μ^1
1)	Wheat	0.466	0.443
2)	Maize	0.521	0.432
3)	Cement	0.316	0.554
4)	Bituminous coal	0.700	0.700

13.4 JANSSEN'S THEORY

Assumptions

1) Most of the weight of the material stored in the bin is supported by friction between the material and the vertical wall.

2) Weight transferred to the hopper bottom is very less. (Hence Rankine or Coulomb's lateral pressure theory cannot be applied.

3) The vertical wall of the bin is subjected to vertical force and horizontal pressure.

Derivation

Let dH = thickness of the elementary layer considered

H = depth from the top

P_v = vertical intensity of pressure acting at top of the layer

$(P_v + dP_v)$ = vertical intensity of pressure acting on the bottom of the layer

Ph = horizontal pressure

f = stress due to friction

v = unit weight of stored material

A = c/s area of material stored

P = bin's interior perimeter

R = A/p = Hydraulic mean depth of the c/s

$\mu^1 = \tan \phi^1$

ϕ^1 = angle of friction on the walls of bin

ϕ = angle of internal friction of stored material consider the equilibrium of the vertical forces acting as shown in Figure.

$$P_v \,.\, A + \gamma \,.\, A \,.\, dH = (P_v + dP_v) \,.\, A + f \,.\, P \,.\, dh$$

or $$Pv \,.\, A + \gamma \,.\, A \,.\, dH = (Pv + dPv) \,.\, A + (\mu^1 + Ph)\, Pdh \qquad ...(i)$$

$$\Rightarrow \qquad \gamma \,.\, dH = dP_v + \frac{\mu^1 + P_h}{R}\, dh$$

$$\Rightarrow \qquad dP_v = \left(\gamma - \frac{\mu^1 P_h}{R}\right) dH \qquad \therefore \qquad P_h = k \,.\, Pv$$

$$\Rightarrow dP_v = \left(\gamma - \mu^1 k \frac{P_v}{R} \right) dH$$

$$\Rightarrow \int \frac{dP_v}{\left(\gamma - \mu^1 k \frac{P_v}{R} \right)} = \int dH \quad \therefore \quad \log \frac{\left(\gamma - \mu^1 k \frac{P_v}{R} \right)}{-\frac{\mu^1 k}{R}} = H + \text{constant}$$

or $\quad \log\left(\gamma - \mu^1 k \frac{Pv}{R} \right) = -\mu^1 \frac{k}{R} h + \theta$

when $h = 0 \quad P_v = 0 \quad C = \log h$

$$\therefore \quad \log\left[\frac{\gamma - \mu^1 P_v \frac{k}{R}}{\gamma} \right] = \mu^1 \frac{k}{R} \cdot H$$

$$\Rightarrow \quad 1 - \frac{\mu^1}{\gamma} \frac{k}{R} \cdot P = e^{-\mu^1 (k/R) H}$$

$$\therefore \quad P_v = \frac{\gamma . R}{\mu^1 k} \left(1 - e^{-\mu^1 \frac{kH}{R}} \right)$$

$$\Rightarrow \quad P_h = \frac{\gamma . R}{\mu^1} \left(1 - e^{-\mu^1 \frac{kH}{R}} \right) \qquad \because P_h = k P_v$$

or $\quad P_h = \frac{\gamma R}{\mu^1} (1 + e^{-h/z_0}) \qquad$ where, $z_0 = \frac{R}{\mu^1 k}$

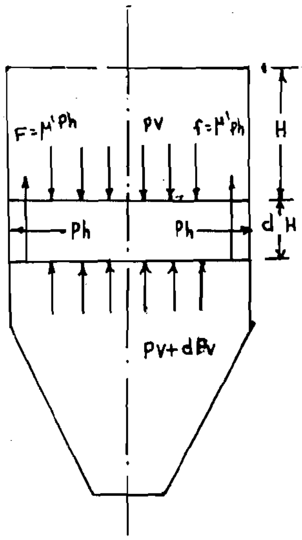


Figure 13.4

values of $(1 - e^{-h/z_0})$ are given in Table 13.2.

Table 13.2: Values of $(1 - e^{-h/z_0})$ as per IS:4995-1968 for Criteria for Design of Reinforced Concrete Bins (Silos)

$\frac{h}{z_0}$	$1 - e^{-h/z_0}$	$\frac{h}{z_0}$	$1 - e^{-h/z_0}$
0.1	0.095	2.1	0.877
0.2	0.181	2.2	0.869
0.3	0.259	2.3	0.899
0.4	0.329	2.4	0.909
0.5	0.393	2.5	0.917
0.6	0.451	2.6	0.925
0.7	0.503	2.7	0.932
0.8	0.550	2.8	0.939
0.9	0.593	2.9	0.945
1.0	0.632	3.0	0.950
1.1	0.667	3.1	0.955
1.2	0.698	3.2	0.959
1.3	0.727	3.3	0.963
1.4	0.753	3.4	0.966

$\frac{h}{z_0}$	$1-e^{-h/z_0}$	$\frac{h}{z_0}$	$1-e^{-h/z_0}$
1.5	0.776	3.5	0.969
1.6	0.798	3.6	0.972
1.7	0.817	3.7	0.975
1.8	0.834	3.8	0.977
1.9	0.850	3.9	0.979
2.0	0.864	4.0	0.981
		∞	1000

The c/s of the silos is generally a circle. The hoop tension on the wall = $P_n \frac{D}{2}$

[D = diameter of silo]

Additionally the vertical wall will be subjected to vertical pressure transferred due to friction P_w

$$P_w = \int_0^h \mu^1 p_n \, dH = \int_0^h \gamma R \left(1 - e^{-\frac{\mu^1 kH}{R}}\right) dH$$

$$\therefore \qquad P_w = \gamma R \left[H - \frac{R}{\mu^1 k} \left(1 - e^{-\frac{\mu^1 kh}{R}}\right)\right]$$

Total vertical pressure = $P_w \times$ Perimeter = $P_w . P$

$$= \gamma . A . \left[h - \frac{R}{\mu^1 k}\left(1 - e^{-\frac{\mu^1 kh}{R}}\right)\right]$$

$$= (\gamma AH - AP_v)$$

The pressure ratio k lies between $\left(\frac{1-\sin\phi}{1+\sin\phi}\right)$ and $\left(\frac{1+\sin\phi}{1-\sin\phi}\right)$. The value of k can be accurately found out experimentally.

Table 13.3: Angle of Wall Friction and Pressure Ratio

S. No.	Material	Angle of Wall Friction δ		Pressure Ratio λ	
		While Filling	While Emptying	While Filling	While Emptying
1)	Granular materials with mean particle diameter ≥ 0.2 mm	0.75ϕ	0.6ϕ	0.5	1.0
2)	Powdery materials (except wheat flour) with mean particle diameter < 0.06 mm	1.0ϕ	1.0ϕ	0.5	0.7
3)	Wheat flour	0.75ϕ	0.75ϕ	0.5	0.7

SAQ 2

1) What is the basis of Airy's theory?

2) What are the assumptions made in Janssen's theory?

3) What are the forces acting on the walls of a bin?

13.5 IS: CODE SPECIFICATIONS

IS: 4995 (Part-I)–1974, gives the general requirements and assessment of bin loads.

The parameters which influence the design of bunkers are:

1) Unit weight of materials.

2) Angle of internal friction.

3) Angle of wall friction.

4) Pressure ratio.

Table 13.4 gives the unit weight and angle of internal friction (ϕ) of some important stored materials.

Table 13.4

Material	Unit Weight (kN/m^3)	Angle of Internal Friction (ϕ)[1] Degrees
Wheat	8.50	28
Paddy	5.75	36
Rice	9.00	33
Maize	8.00	30
Barley	6.90	27
Corn	8.00	27
Sugar	8.20	35
Wheat flour	7.00	30
Coal	8.00	35
Coke	4.30	30
Ash	6.50	30
Cement	15.50	25
Lime	16.50	25

Bin Loads

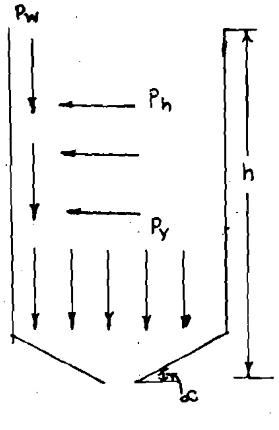


Figure 13.5: Bin Loads

Three types of loads are caused by a stored material in a bin. These are.

1) Horizontal pressure (p_h) acting on the side walls.

2) Vertical pressure (p_v) acting on the cross-sectional area ' `the bin filling,

3) Frictional wall pressure (p_w).

Governing loading cases are given in Table 13.5:

Table 13.5

Pressure	Granular Material	Powdery Material
p_w	Emptying	Filling = Emptying
p_h	Emptying	Filling = Emptying
p_v	Filling	Filling

13.5.1 Calculation of Loads as Per IS Code

Case 1: Granular Materials

1) *Maximum pressure*

Table 13.6

Name of Pressure	During Filling	During Emptying
Frictional wall pressure	γR	γR
Horizontal pressure	$\frac{\gamma R}{\mu_f}$	$\frac{\gamma R}{\mu_e}$
Vertical pressure	$\frac{\gamma R}{\mu_f \lambda_f}$	$\frac{\gamma R}{\mu_e \lambda_e}$

where, γ = unit weight of material stored,

$R = \frac{A}{P}$ ratio

μ_f = coefficient of wall friction during filling,

μ_e = coefficient of wall friction during emptying,

λ_f = pressure ratio during filling,

λ_e = pressure ratio during emptying.

2) *Variation of Pressure along the Depth*

$$p_i(Z) = (p_i)_{\max}(1 - e^{-Z/Z_0})$$

where, p stands for pressure and suffix i stands for w, h or v.

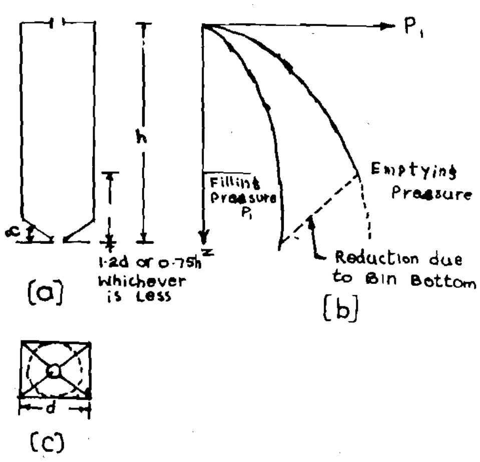


Figure 13.6

During filling $Z_{of} = \dfrac{R}{\mu_f \lambda_f}$

During emptying $= \dfrac{R}{\lambda_e \mu_e}$

To reduce the load effect of the bin bottom, the horizontal pressure during emptying may be reduced upto a height of $1.2d$ or $0.75h$ (Figure 13.6) whichever is smaller from the bin bottom.

Case 2 : Powdery Materials

Maximum design pressures are the same as in case (1).

The lateral and vertical pressures

$p_h = p_v = 0.6\,\gamma Z$ – during homogenization

$p_h = 0.8\,\gamma z_n$ – during rapid filling

where, $Z_n = (v - v_0)\,t$

$v =$ actual filling speed, (m/h)

$v_0 =$ the min-filling speed (m/h)

$t =$ time lapse (hours)

The values of v_0 for some material are given in Table 13.7.

Table 13.7

Material	v_0
Cement	2.6
Lime	1.4
Wheat flour	4.8

During pneumatic emptying air under pressure is blown inside the bin through a number of small holes located in the bin walls near the bin bottom. This causes the liquefaction of the material in the lower portion of the bin and gives rise to higher values of p_h and p_v (Figure 13.7).

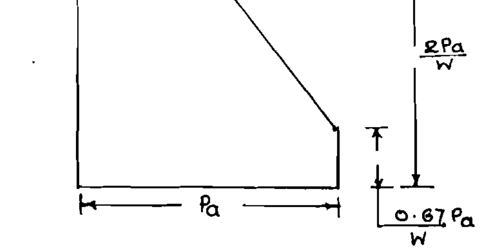


Figure 13.7: Pressure Scheme for Pneumatic Emptying

13.5.2 Factors Increasing the Bin Loads

1) Eccentric emptying.
2) Arching of stored material.
3) Discharge promoting devices.
4) Aeration of stored material.

1) Eccentric Emptying

Eccentric emptying of a bin gives rise to increased horizontal loads non-uniformly distributed over the periphery and extending over the full height of the bin. Eccentric outlets in bins shall be avoided as far as possible and where they have to be provided to meet functional requirements, due consideration shall be given in design to the increased pressure experienced by the walls. This increased pressure shall be considered for the purpose of design, to be acting both on the wall nearer to the outlet as well as on the wall on the opposite side.

The enlarged shape of the bin which is required for the purpose of computation of the pressure P_{hi} shall be obtained as shown in Figures 13.8 and 13.9.

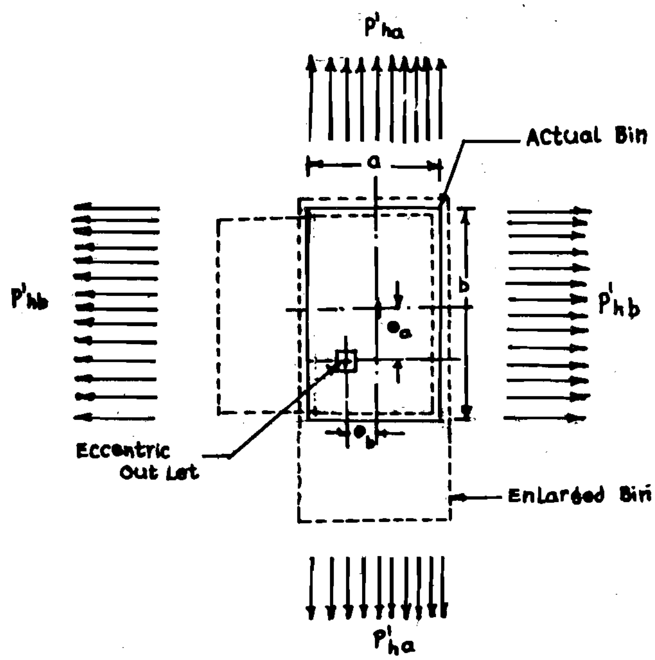


Figure 13.8: Rectangular Bin

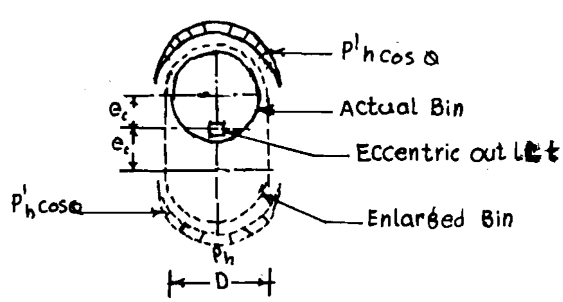


Figure 13.9: Circular Bin

The effect of eccentric outlets may be ignored in design if the eccentricity is less than *d*/6 or the height of the bin is not greater than 2*d*, where *d* is the maximum possible diameter of the circle that can be inscribed in the bin.

2) **Arching of Stored Material**

Some stored materials are susceptible to arching action across the bin walls. The frequent collapse of such order gives rise to increased vertical pressures. The vertical pressures on the bottom of the bin storing such materials shall be taken as twice the filling pressure, P_{vi} however the loadneed not be assumed to be greater than $w.z$.

3) **Discharge Promoting Devices**

Modern bins storing various materials may be provided with various discharge promoting devices such as inserts, bridge like structures above the outlet or relief nose. In all such cases the effective cross-section of the bin is locally reduced. Recent research has given an indication that in such bins, the horizontal wall pressures are excessively increased locally or along the entire bin height.

4) **Aeration of Stored Material**

When bins are provided with equipment for ventilating the bin filling at rest, a distinction must be made between bins for granular material and bins for powdery material.

When the material is granular, an increase in the horizontal pressures is to be expected. Therefore, the horizontal pressure *Ph* (for filling) has to be increased by the inlet pressure of the air over that portion of the height of the bin in which the air inlets are located. From the level of the highest inlet upwards, this increase in pressure may be tappered off uniformly down to zero at the top of the bin.

For powdery materials the measurements made so far do not indicate any significant increases in load when ventilating.

Bins for storage of powdery materials are often equipped with devices for pneumatic emptying, and these bring about a loosening of the bin filling in the region of the outlet. In this case also, no significant increases in load due to the air supply have far been detected.

13.5.3 Analysis of Bunkers

Let us consider a symmetrical rectangular bunker with trough bottom. For the analysis of loads, unit length is considered. The cross-section is showing in Figure 13.10.

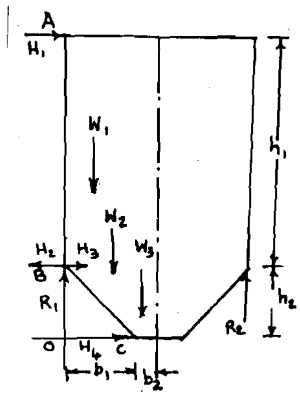


Figure 13.10

Caculate the horizontal pressures at different points. Find the total pressure P_{h_1}, P_{h_2} and P_{h_3} as shown in Figure 13.11.

$$H_1 = \frac{P_{h_1}}{3}$$

$$H_2 = \frac{2}{3} P_{h_1}$$

By taking moments about 0,

$$H_3 = \frac{1}{h_2}\left[w_1 \cdot \frac{b_1}{2} + w_2 \cdot \frac{2b_1}{3} + w_3 \cdot b_1 - \frac{P_{h1} \cdot h_2}{2} - \frac{P_{h3} \cdot h_2}{2} \right]$$

By taking moments about B,

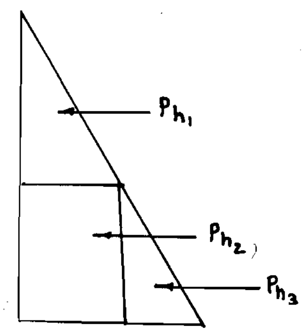


Figure 13.11

$$H_4 = \frac{1}{h_2}\left[w_2 \cdot \frac{b_1}{2} + w_2 \cdot \frac{2b_1}{3} + w_3 \cdot b_1 + \frac{P_{h_2} \cdot h_2}{2} + \frac{2P_{h_3} \cdot h_2}{3} \right]$$

The forces acting on the sloping sides are calculated as follows:

Let P_B and P_C are the normal pressure at B and C, then the normal load acting on the sloping side at the centroid of the pressure diagram.

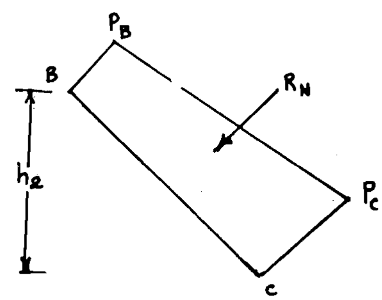


Figure 13.12

$$N = \left(\frac{P_B + P_C}{2} \right),\ h_2 \operatorname{cosec} \alpha$$

Normal pressure,

$$p_n = p_v \cos^2 \alpha + p_h \sin^2 \alpha + w_s \cos \alpha$$

Tangential pressure,

$$p_t = \left(\frac{p_v - p_h}{2} \right) \sin 2\alpha + w_s \sin \alpha$$

where, $w_s =$ self weight of hopper

$\alpha =$ angle of the hopper with the horizontal.

13.5.4 Procedure for Design of Bunkers

Step 1: Force analysis

a) Calculate the vertical forces.

b) Calculate the horizontal forces using code specification.

c) Calculate the bursting forces H_1, H_2, H_3 and H_4. Using equation of equilibrium.

d) Calculate the pressure p_v, p_w, p_h on trough walls.

e) Calculate the normal and tangential pressures.

f) Calculate the normal load on trough.

Step 2: Design of trough plates

a) Span = spacing of stiffeners.

b) Considering truss-way bending, calculate the maximum bending,

$$M_1 = \frac{p\,L^2}{2 \times 12}$$

where, p = maximum normal pressure

L = span of trough plate.

c) Calculate the thickness required

$$t = \sqrt{\frac{6M_1}{\sigma_{bc} \times L}}$$

Min. thickness = 6 mm.

Step 3: Design of inclined stiffeners in trough

a) Calcular the maximum BM and (M_2) and direct tension at mid-span.

b) Choose suitable T-section with plate.

c) Calculate A, I_{XX} and Z_{XX}.

d) Check for tensile stress and bending stress.

Step 4: Design of plate stiffeners for trough

These are provided perpendicular to the T-stiffeners.

a) Calculate the maximum BM.

$$M_3 = \frac{p\,L^2}{2 \times 12} \times L.$$

b) Calculate the section modulers Z_{required}.

c) Assuming thickness (t), find the depth of plate

$$\frac{1}{6}\,t\,d^2 = z$$

Step 5: Design of vertical plate

a) Calculate the maximum BM, $M_4 = \dfrac{p\,L^2}{2 \times 12}$

b) Calculate = $t_{rq} = \sqrt{\dfrac{6\,M_4}{\sigma_{bc} \times L}}$. But min t = 6 mm

Step 6: Design of vertical stiffeners

a) Calculate the max. $BM, M_5 = \dfrac{pL^2}{2 \times 8}$

b) Calculate $Z_{\text{required}} = \dfrac{M_5}{\sigma_{bc}}$

c) Choose a standard T-section with plate

d) Calculate A, I_{XX}, Z_{XX}

e) Check for bending stress.

Step 7: Design main (longitudinal) girder

a) Calculate the moment due to H_1 at top, $M_6 = \dfrac{H_1\,L^2}{8}$

b) Calculate $Z_{\text{required}} = \dfrac{M_6}{\sigma_{bc}}$

c) Choose the suitable section.

Step 8: Design of horizontal girder

a) Calculate the moment due to H_3

$$M_7 = \frac{H_3\,L^2}{8}$$

b) Calculate Z_{required}.

c) Select the suitable section.

13.5.5 Design Problem

Design a rectangular bunker 16 m length and 8 m width supported on ten columns. It stores maize. Height of vertical portion = 4 m. Height of hopper = 4 m.

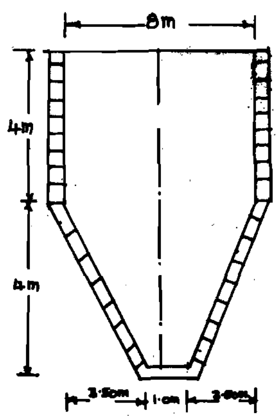


Figure 13.13

Step 1: Force Analysis

ϕ for maize $= 30°$

Unit weight $= 8.00\ \text{kN/m}^3$

For filling, $\phi_f^1 = 0.75\,\phi = 22.5°$

For emptying, $\phi_e^1 = 0.60\phi = 18°$

Pressure ratio,

For filling, $\lambda_f = 0.50$

For emptying, $\lambda_e = 1.00$

Ffor filling, $\mu_f^1 = \tan\phi_f^1 = \tan\phi_f^1 = 0.414$

Ffor emptying, $\mu_f^1 = \tan\phi_e^1 = 0.325$

Taking 1 m length

Weight $w_1 = 8 \times 4 \times 3.5 = 112 \text{ kN}$

Weight $w_2 = 8 \times \left(\frac{1}{2} \times 4 \times 3.5\right) = 56 \text{ kN}$

Weight $w_3 = 8 \times (0.5 \times 8) = 32 \text{ kN}$

Horizontal forces

$$R = \frac{A}{P} = \frac{16 \times 5}{2\,(16+5)} = 2.67 \text{ m}$$

$$Z_{oe} = \frac{R}{\mu_e^1 \cdot \lambda_e} = \frac{2.67}{0.325 \times 1.0} = 8.22 \text{ m}$$

$$Z_{of} = \frac{R}{\mu_f^1 \cdot \lambda_f} = \frac{2.67}{0.414 \times 0.50} = 12.9 \text{ m}$$

$$Z_{of} > Z_{oe}$$

$\therefore$ $\frac{h}{Z_{oe}}$ is more (Emptying)

At B :

$$\frac{h}{Z_{oe}} = \frac{4}{8.22} = 0.487$$

$$(1 - e^{-h/z_0}) = 0.39 \quad \text{(from table)}$$

$$P_h = \frac{\gamma R}{\mu_e} \times 0.39 = \frac{8 \times 2.67}{0.325} \times 0.39$$

$$= 25.63 \text{ kN/m}^2$$

At C :

$$\frac{h}{Z_{oe}} = \frac{8}{8.22} = 0.974$$

$$(1 - e^{-h/z_0}) = 0.62 \quad \text{(from table)}$$

Horizontal pressure $P_h = \frac{\gamma R}{\mu_0} \times 0.62$

$$= 40.75 \text{ kN/m}^2$$

Total Horizontal Pressures

$$P_{h_1} = \frac{1}{2} \times 25.63 \times 4 = 51.26 \text{ kN/m}$$

$$P_{h_2} = 25.63 \times 4 = 102.52 \text{ kN/m}$$

$$P_{h_3} = \frac{1}{2} \times (40.75 - 25.63) \times 4 = \frac{60.48}{2} = 30.24 \text{ kN/m}$$

Bursting Forces on Walls

$$H_1 = \frac{1}{3} \times 51.26 = 17.09 \text{ kN/m}$$

$$H_2 = \frac{2}{3} \times 51.26 = 34.18 \text{ kN/m}$$

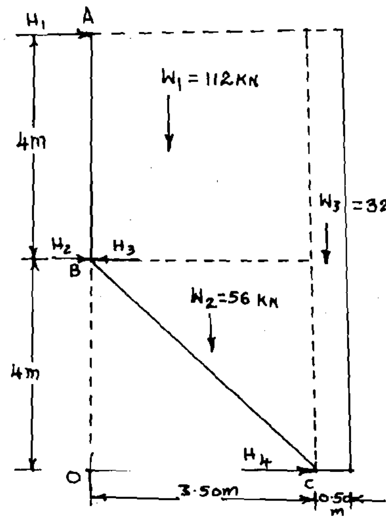


Figure 13.14

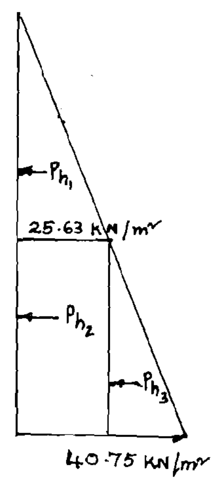


Figure 13.15

Taking moments about 0,

$$H_3 \times 4 = \left\{ 112 \times \frac{3.5}{2} + 56 \times \frac{2}{3} \times 3.5 + 32 \times \left[(3.5) + \frac{0.5}{2} \right] - 102.52 \times 2 - 30.24 \times \frac{4}{3} \right\}$$

$$= 201.307$$

$\therefore \quad H_3 = 50.33 \text{ kN}.$

Taking moments about B,

$$H_4 \times 4 = \left[112 \times \frac{3.5}{2} + 56 \times \frac{2}{3} \times 3.5 + 32 \times 3.75 + 102.52 \times 2 + 30.24 \times \frac{2}{3} \times 4 \right]$$

$$H_4 = 183.09 \text{ kN}$$

$$P_{h_1} + P_{h_2} + P_{h_3} = H_1 + H_2 - H_3 + H_4$$

$\therefore$ Sum of horizontal forces is zero.

Pressures on Trough Walls

$$\text{Max. } p_V = \frac{\gamma R}{\mu_f \cdot \lambda_f} = \frac{8 \times 2.67}{0.414 \times 0.50} = 103.19 \text{ kN/m}^2$$

At $B, p_V = 103.19 \times 0.39 = 40.24 \text{ kN/m}^2$

Vertical pressure due to weight on wall, $p_w = 8 \times 4 = 32 \text{ kN/m}^2$

$$\because p_V > p_w \quad \therefore \quad p_V \text{ is taken.}$$

Vertical pressure at C due to weight of material $= 8 \times 8 = 64 \text{ kN/m}^2$

At B, vertical pressure = 40.24 kN/m^2

horizontal pressure = 25.63 kN/m^2

At C, vertical pressure = 64.00 kN/m^2

horizontal pressure = 40.75 kN/m^2

Length, $BC = \sqrt{4^2 + 3.5^2} = 5.32\text{ m}$

$$\tan\alpha = \frac{4}{3.5} = \alpha = 48.8$$

$$\sin\alpha = 7.5 \quad \sin 2\alpha = 0.975$$

$$\cos\alpha = 0.65$$

Normal Pressures

At B, $p_B = p_V \cdot \cos^2\alpha + p_h \cdot \sin^2\alpha$

$$= (40.24)\,(0.65)^2 + (25.63)(0.75)^2 = 31.42\text{ kN/m}^2$$

At C, $p_C = (64)(0.65)^2 + (40.75)\,(0.75)^2$

$$= 49.96\text{ kN/m}^2$$

Tangential Pressures

At B, $p_{tB} = \dfrac{(p_V - p_h)}{2}\sin 2\alpha$

$$= \frac{(40.24 - 25.63)}{2} \times 0.975$$

$$= 7.12\text{ kN/m}^2$$

At C, $p_{tc} = \dfrac{(64 - 40.75)}{2} \times 0.975$

$$= 11.33\text{ kN/m}^2$$

Normal load on trough wall

$$= \frac{(31.42 + 49.96)}{2} \times (5.32)$$

$$= 216.47\text{ kN}$$

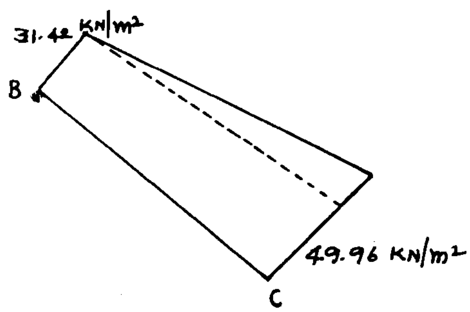


Figure 13.16

Step 2: Design of trough plates

Adopt a spacing of 600 mm for stiffeners. The plates are bending in two-direction.

$$\text{Maximum } BM,\ M = \frac{pL^2}{2 \times 12}$$

$$= \frac{49.96 \times 0.6^2}{24} = 0.75 \text{ kN-m}$$

$$\text{Thickness,} \qquad t = \sqrt{\frac{6M_1}{\sigma_{bc} \times L}} = \sqrt{\frac{6 \times 0.75 \times 10^6}{165 \times 600}} = 6.74 \text{ mm}$$

Adopt 8 mm thick plates.

Step 3: Design of inclined stiffeners in a trough

Length of the stiffeners = 5.32 mm length

BM at the mid space/m length

$$= \frac{31.42 \times 5.32^2}{8} + \frac{(49.96 - 31.42)\, 5.32^2}{2 \times 8} = 143.88 \text{ kN-m}$$

For 600 mm spacing, $M_2 = 0.6 \times 143.88 = 86.33$ kN-m

$$Z_{required} = \frac{M_2}{\sigma_{bc}} = \frac{86.33 \times 10^6}{165} = 523.2 \times 10^3 \text{ mm}^3.$$

Let us try ISST 250 @ 375 N/m with 350 mm wide plate. Thickness of plate is 10 mm

For ISST 250 @ 375 N/m

$$A = 4775 \text{ mm}^2$$

$$C_{XX} = 64 \text{ mm}$$

$$I_{XX} = 2774.4 \times 10^4 \text{ mm}^4$$

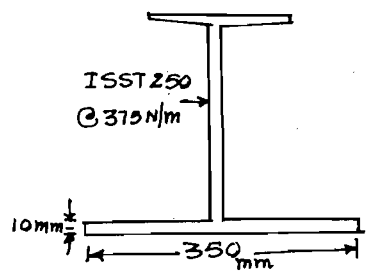


Figure 13.17

$$\bar{y} = \frac{4775 \times 64 + (350 \times 10)\left(250 + \frac{10}{2}\right)}{4775 + 350 \times 10}$$

$$= 144.8 \text{ mm}$$

$$I = 2774.4 \times 10^4 + 4775\,(144.8 - 64)^2 + \frac{350 \times 10^3}{12} + 350 \times 10\,(144.8 - 255)^2$$

$$= 10145.2 \times 10^4 \text{ mm}^4$$

$$Z_{XX} = \frac{I}{y_{max}} = \frac{10145.2 \times 10^4}{144.8}$$

$$= 700.6 \times 10^3 \text{ mm}^3$$

$$> Z_{required} \qquad \therefore \qquad \text{Safe.}$$

Step 4: Design of plate stiffeners in trough

Spacing of plate stiffeners = 600 mm (let)

$$BM, M_3 = \frac{pL^2}{24} \times L = \frac{49.96 \times 0.6^2}{24} \times 0.6 = 0.45 \text{ kN-m}$$

$$Z_{required} = \frac{M_3}{\sigma_{bc}} = \frac{0.45 \times 10^6}{165}$$

$$= 2727.3 \text{ mm.}$$

Adopting 10 mm thick plate,

$$\frac{1}{6} \times 10 \times d^2 = 2727.3$$

$\therefore \quad d = 40.5$ mm.

Step 5: Design of vertical plates

$$\text{Maximum } BM, M_4 = \frac{pL^2}{24}$$

$$= \frac{25.63\,(0.6)^2}{24} = 0.38 \text{ kN-m}$$

$$\text{Thickness } t = \sqrt{\frac{6\,M_4}{\sigma_{bc} \times L}}$$

$$= \sqrt{\frac{6 \times 0.38 \times 10^6}{165 \times 600}} = 4.79 \text{ mm}$$

Adopt 6 mm thickwall.

Step 6: Design of vertical stiffeners

At A, $\quad p_h = 0$

At B, $p_h = 25.63 \text{ kN/m}^2$

Length, $L = 4$ m

$$\text{Max. } BM, M_5 = \frac{pL^2}{2 \times 8}$$

$$= \frac{25.63 \times 4^2}{2 \times 8} = 25.63 \text{ kN-m}$$

Try for ISST 200 @ 284 N/m with 280 mm wide × 10 mm thick plate.

Properties of ISST 200 are:

$$A = 3622 \text{ mm}^2$$

$$C_{XX} = 47.8 \text{ mm}$$

$$I_{XX} = 1267.5 \times 10^4 \text{ mm}^4$$

$$\bar{y} = \frac{3622 \times 47.8 + 280 \times 10 \times \left(200 + \frac{10}{2}\right)}{(3622 + 280 \times 10)}$$

$$= 116.34 \text{ mm}$$

$$I_{XX} = 1267.5 \times 10^4 + 3622\,(116.34 - 47.8)^2$$

$$+ \frac{280 \times 10^3}{12} + 280 \times 10 \times (116.34 - 205)^2$$

$$= 5172.3 \times 10^4 \text{ mm}^4$$

$$X_{XX} = \frac{5172.3 \times 10^4}{116.34} = 444.6 \times 10^3 \text{ mm}^3$$

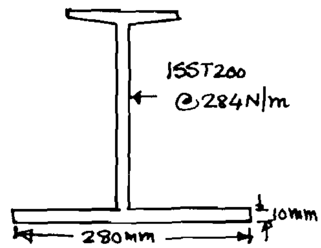


Figure 13.18

$$Z_{required} = \frac{M_5}{\sigma_{bc}} = \frac{25.63 \times 10^6}{165}$$

$$= 155.33 \times 10^3 \text{ mm}^3$$

$Z_{XX} > Z_{required}$. Hence safe.

Step 7: Design of Main Beam

Length of the beam = 4 m

$$H_1 = 17.09 \text{ kN/m}$$

Bending moments $M_6 = \dfrac{17.09 \times 4^2}{8} = 34.18$ kN-m

$$Z_{required} = \frac{M_6}{\sigma_{bc}} = \frac{34.18 \times 10^6}{165} = 207.15 \times 10^3 \text{ mm}^3$$

Adop ISLB 225 @ 235 N/m $\quad (Z_{XX} = 222.4 \times 10^3 \text{ mm}^3)$

Step 8: Design of horizontal beam

Length, $L = 4$ m

$$H_3 = 50.33 \text{ kN/m}$$

$$M_7 = \frac{H_2 L^2}{8} = \frac{50.33 \times 4^2}{8} = 100.66 \text{ kN-m}$$

$$Z_{required} = \frac{M_7}{\sigma_{bc}} = \frac{100.66 \times 10^6}{165}$$

$$= 610 \times 10^3 \text{ mm}^3$$

Use ISLB 350 @ 495 N/m $\quad (Z_{XX} = 7519 \times 10^3 \text{ mm}^3)$

13.6 DESIGN OF SILO

Generally, the Silos are circular in shape. These are designed similar to bunkers.

Design Procedure

Step 1: Calculation of horizontal pressure

By using the codal provisions, find the horizontal and vertical pressures at different depths at some intervals say 3 m, 4 m/5 m.

Step 2: Calculation of max. hoop tension

Hoop tension, $H_t = (p_h) \max \cdot \frac{D}{2} \cdot$

Step 3: Design of wall plate

Calculate total vertical load, self weight, weight of lining, weight of top cover.

Calculate the vertical load.

Calculate thickness of plate from combined loading.

Step 4: Design of hopper

Calculate the total vertical load.

Calculate the direct tension.

Calculate the thickness $= \frac{\text{Direct tension}}{\sigma_{at} \times 1000 \text{ mm}}$

Step 5: Design of ring beam

Calculate the weight of stored material, self weight of silos lining cover, platform.

Calculate the reaction, SF, BM, torsion and compression.

Calculate σ_{ac}, $\sigma_{ac,\,cal}$, σ_{bc}, $\sigma_{bc,\,cal}$

Check for combined stresses.

Example 13.1

Design a circular steel silo of 10 m height and 4 m internal diameter to store cement of unit weight 15.50 kN/m^3 and $\phi = 25°$.

Solution

Step 1: Calculation of pressures

The mean size of particle is less than 0.06 mm.

For powdery material,

For filling, $\phi^1 f = 1.0\,\phi = 25°$

For emptying, $\phi^1 e = 1.0\,\phi = 25°$

Pressure ratio,

For filling, $\lambda_f = 0.5$

For emptying, $\lambda_e = 0.7$

Angle of wall friction,

For filling, $\mu^1{}_f = \tan\phi^1{}_f$

$= 0.47$

For emptying, $\mu^1 e = 0.47$

$$R = \frac{d}{4} = 1.0\ \text{m}$$

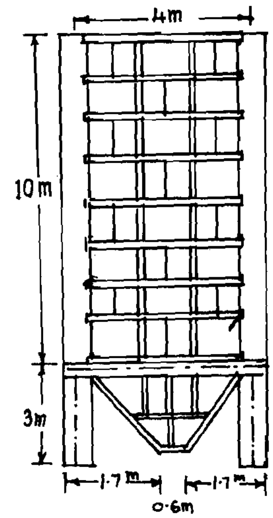


Figure 13.19

$$Z_{oe} = \frac{R}{\mu_e^1 \lambda_e} = \frac{1}{0.47 \times 0.70} = 3.064$$

$$Z_{of} = \frac{R}{\mu_f^1 \lambda_f} = \frac{1}{0.47 \times 0.50} = 4.26$$

$$Z_{oe} < Z_{of}$$

$\therefore$ $\frac{h}{Z_{oe}}$ is more (emptying)

At 12 m

$$\frac{h}{Z_{oe}} = \frac{10}{3{,}064} = 3.26 \quad (1 - e^{-h/z_0}) = 0.96$$

Horizontal pressure, $p_h = \frac{\gamma R}{\mu_f^1} \times 0.96$

$$= \frac{1 \times 15.5 \times 0.96}{0.47} = 31.66\ \text{kN/m}^2$$

Vertical pressure, $p_v = \frac{\gamma R}{\mu_f^1 \lambda_f} \times 0.96 = \frac{31.66}{0.50} = 63.32\ \text{kN/m}^2$

Step 2: Max. Hoop Tension

$$H_t = p_h \frac{D}{2} = 31.66 \times \frac{4}{2} = 61.32\,\text{kN/m}$$

Step 3: Design of Wall Plate

Total vertical load $= \frac{\pi}{4} \times 4^2 \times 10 \times 15.5 = 1948\ \text{kN}$

Assume wt of silo + stiffeners and lining = 2 kN/m^2

Total wt. = $2 \times \pi \times 4 \times 10 = 251$ kN

Weight of top cover = 4 kN/m^2

Total wt. $= \frac{\pi}{4} \times 4^2 \times 4 = 50$ kN

Total vertical load = 1948 + 251 + 50 = 2249 kN

Vertical load/mm length = $\frac{2249 \times 10^3}{\pi \times 4000} = 179$ N/mm

H_t = 61.32 kN/m = 61.32 N/mm

Let t be the thickness, 0.3 be the poisson ratio.

Max. compressive stress = $\frac{179}{t} + \frac{(0.3 \times 61.32)}{t} = 150$

$\therefore \quad t = 1.32$ mm

Adopt 8 mm thick plate.

Since the thickness provide is very lower than the required, nominal stiffeners are provided @ 1.50 m spacing.

Stiffeners ISA @ 6060, 6 mm

Step 4: Design of hopper

Vertical load $= \frac{\pi}{4} d^2 p_v$

$= \frac{\pi}{4} \times 4^2 \times 63.32$

$= 795.70$ kN

Weight of material is hopper

$= \frac{\pi}{12} \times 4^2 \times 3 \times 15.5$

$= 194.68$ kN

Self wt. = 60 kN (let)

Total load = 795.70 + 194.68 + 60 = 1050.38 kN

Length of hopper $= \sqrt{3^2 + 1.7^2} = 3.45$ m

Load/mm run $= \frac{1050.38 \times 10^3}{\pi \times 4000} = 84$ N/mm

Direct tension $= \frac{84 \times 3.45}{3} = 96.6$ N/mm

Assuming 8 mm thick plate tensile stress

$= \frac{96.6}{8} = 12.1 \text{ N/mm}^2 < 150 \text{ N/mm}^2$

$\therefore$ Safe.

Step 5: Design of Ring Beam

Weight of material stored

$$= \left(\frac{\pi}{4} \times 4^2 \times 10 = \frac{1}{3} \times \frac{\pi}{4} \times 4^2 \times 3 \right) \times 15.5 = 2143 \text{ kN.}$$

Self wt. of silo lining, cover, platform

$$= 251 + 50 + 50 = 351 \text{ kN}$$

Total load = 2143 + 351

= 2494 kN

≃ 2500 kN (say)

Using 8 No. of supports.

$$\text{Reaction} = \frac{2500}{8} = 312.5 \text{ kN}$$

$$\text{Shear force} = \frac{2500}{16} = 156.25 \text{ kN}$$

Bending moment at support

$= 0.00827\ WR$

$= 0.00827 \times 2500 \times 2$

$= 41.35$ kN-m.

$$\text{Direct compression} = \frac{1}{2} \times \frac{2500}{1.7} \times 3 = 2206 \text{ kN}$$

Assume $\sigma_{ac} = 1200 \text{ N/mm}^2$

$$\text{Gross area required} = \frac{2206 \times 10^3}{120} = 18382.4 \text{ mm}^2$$

Adopting the sect shown in Figure 13.20.

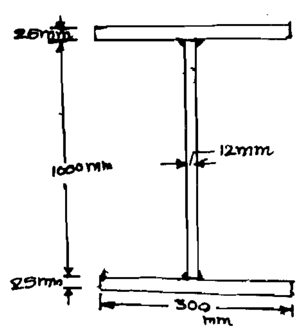


Figure 13.20

$$A = 2\,(300 \times 25) + 1000 \times 12 = 27000 \text{ mm}^2$$

$$I_{XX} = \frac{300 \times 1050^3}{12} - \frac{288 \times 1000^3}{12}$$

$$= 49.4 \times 10^8 \text{ mm}^4$$

$$I_{YY} = \frac{2 \times 25 \times 300^3}{12} + \frac{1000 \times 12^3}{12}$$

$$= 1.13 \times 10^8 \text{ mm}^4$$

$$I_{\min} = 1.13 \times 10^8 \text{ mm}^4$$

$$r_{\min} = \sqrt{\frac{I_{\min}}{A}} = \sqrt{\frac{1.13 \times 10^8}{27000}} = 64.7\,\text{mm}$$

Length between two adjacent columns

$$l = \frac{\pi D}{8} = \frac{\pi \times 4}{8} \times 1000 = 1570.8\,\text{mm}$$

Slenderness ratio, $\lambda = \frac{l}{r_{\min}} = \frac{1570.8}{64.7} = 24.2$

From Table 5.1, $\sigma_{ac} = 146\,\text{N/mm}^2$

$$\sigma_{ac,\,cal} = \frac{P}{A} = \frac{2206 \times 10^3}{27000} = 81.7\,\text{N/mm}^2$$

$$\sigma_{bc,\,cal} = \frac{M}{I} \cdot y$$

$$= \frac{41.35 \times 10^6}{49.40 \times 10^8} \times \left(\frac{1050}{2}\right)$$

$$= 4.4\,\text{N/mm}^2$$

$$\sigma_{bc} = 165\,\text{N/mm}^2$$

$$\frac{\sigma_{ac,\,cal}}{\sigma_{ac}} + \frac{\sigma_{bc,\,cal}}{\sigma_{bc}} = \frac{81.7}{146} + \frac{4.4}{165}$$

$$= 0.59 < 1.00$$

Hence safe.

SAQ 2

1) What are the factors causing increase in load on bunkers?

2) Design a bunker of size 12 m length × 6 width. It has 4 m depth vertical plate and height of trough is 4 m. Use coal for storing.

3) Design a silo for time having 4 m dia and 16 m height. Assume any suitable data.

13.7 SUMMARY

In this unit you have studied about the basic theories and design philosophy of bunkers and silos. After reading this unit, you can understand the theories related to bunkers and silos. The design procedures are given for easy understanding. After studying, you are able to design the steel bunkers and silos.

13.8 ANSWERS TO SAQs

SAQ 1

1) Section 13.1

2) Section 13.1

3) Section 13.1

4) Section 13.1

5) Section 13.2

6) Section 13.2

7) Section 13.2

SAQ 2

1) Sub-section 13.5.2

2) Sub-section 13.5.5

3) Sub-section 13.6.2

Chapter 1 Introduction to Bridge Engineering

- Bridge is a structure that forms part of a highway that covers a gap
- Bridges carry a road or railway across a natural or artificial obstacle such as *a river, canal or another railway or another road*
- Bridge is a structure corresponding to the heaviest responsibility in carrying a free flow of transport and is the most significant component of a transportation system in case of communication over spacings/gaps for whatever reason such as aquatic obstacles, valleys and gorges etc.

Bridge is the KEY ELEMENT in a Transportation System

Chapter 1 Introduction to Bridge Engineering

- If the width of a bridge is insufficient to carry the number of lanes required to handle the traffic volume, the bridge will be a constriction to the flow of traffic.
- If the strength of a bridge is deficient and unable to carry heavy trucks, load limits will be posted and truck traffic will be rerouted.
- The bridge controls both the volume and weight of the traffic carried by the transportation system.
- Bridges are expensive. The typical cost per mile of a bridge is many times that of the approach roads to the bridge.

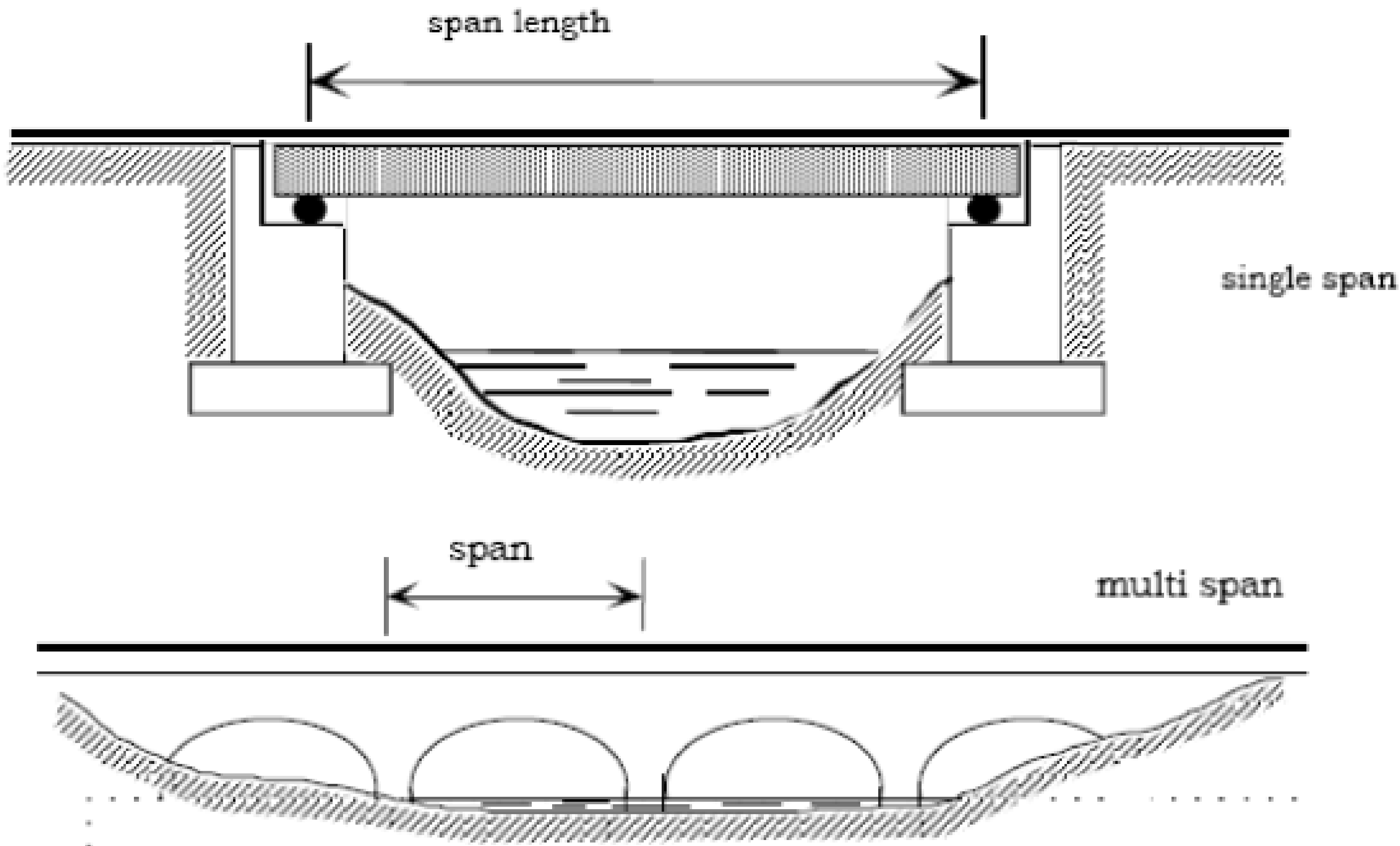
Bridge Classifications

- Span
- Traffic
- Materials and fabrications
- Structural Systems

Types of Bridges by Span Lengths

Span > 20 ft - Bridge
Span < 20 ft - Culvert

Short span: 20-100 ft
Medium span: 100-330 ft
Long span: > 330 ft



Types of Bridges by Traffic

- Highway bridge (trucks, cars)
- Pedestrian bridge (pedestrians, bicycles)
- Railway bridge (trains)
- Transit guideway (light rail, commuter rail)
- Other types (pipelines, utilities, industrial, aqueduct, airport structure)

Types by Material & Fabrications

- **Fabrications**
  - Precast (RC/PC)
  - Cast-in-place (RC/PC)
  - Pretensioned (PC)
  - Post-tensioned (PC)
  - Prefabricated (steel)
  - Bolted (steel/timber)
  - Welded (steel)

Types by Material & Fabrications

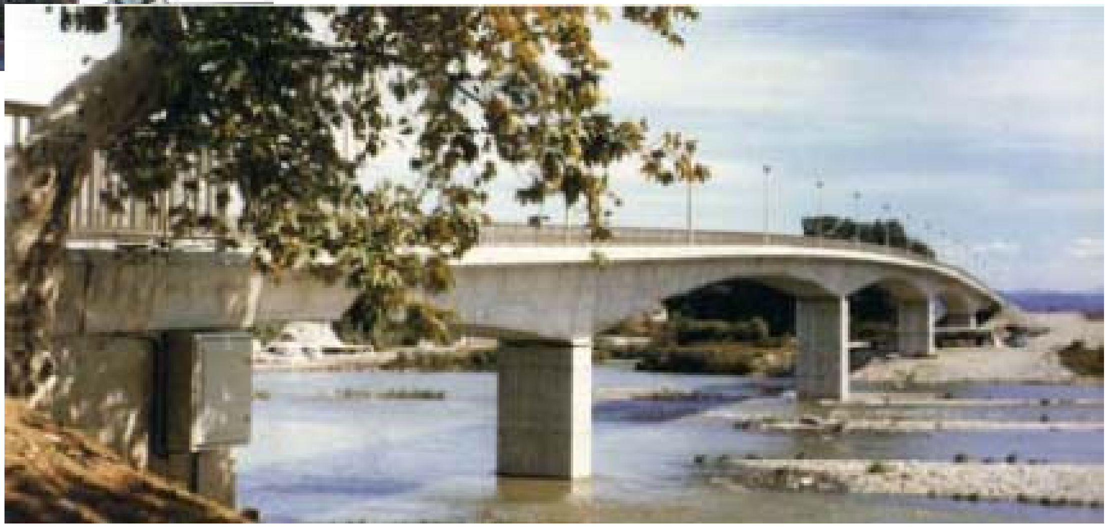
- **Materials**
  - Masonry (brick, rock)
  - Timber
  - Reinforced Concrete (RC)
  - Prestressed Concrete (PC)
  - Steel
  - Aluminum
  - Composites
  - Plastics

Types of Bridge by Traffic Position

- **Deck type**
  - ➢ Structural components under the deck
  - ➢ Preferred by drivers (can clearly see the view)
  - ➢ Requires space under the bridge
- **Through type**
  - ➢ Structural components above the deck
  - ➢ Obstructed view (not a problem for railway bridges)
  - ➢ No structure under the bridge
- **Half-through type**

Types: Deck Type

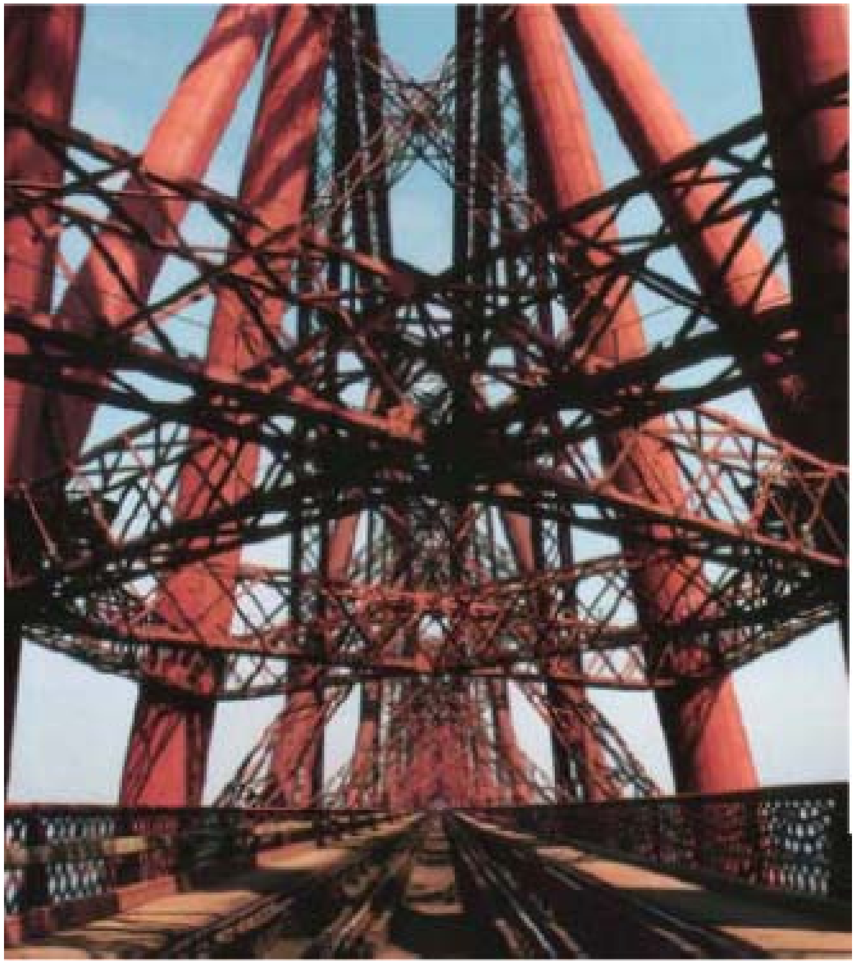
Henry Hudson Bridge (1936)



Rhone Bridge France

Types: Through Type

Firth of Forth Bridge (1890), Scotland, 521m span



Types: Through Type

Tonegawa River Bridge (1972), Japan

Types: Half-Through

Bridge Types by Structural Systems

- Beam/Girder
- Arch
- Cantilever
- Cable-Stayed
- Suspension
- Others

Types: Beam/Girder Bridges

Common Materials

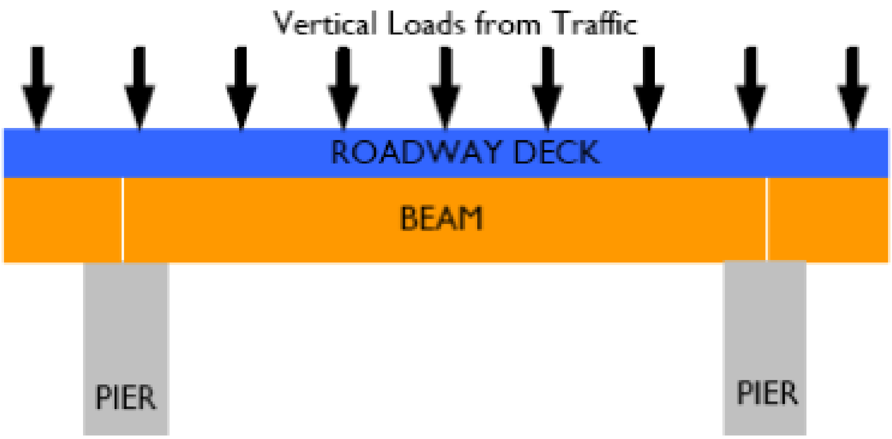
- Timber
- Reinforced Concrete
- Prestressed Concrete
- Steel

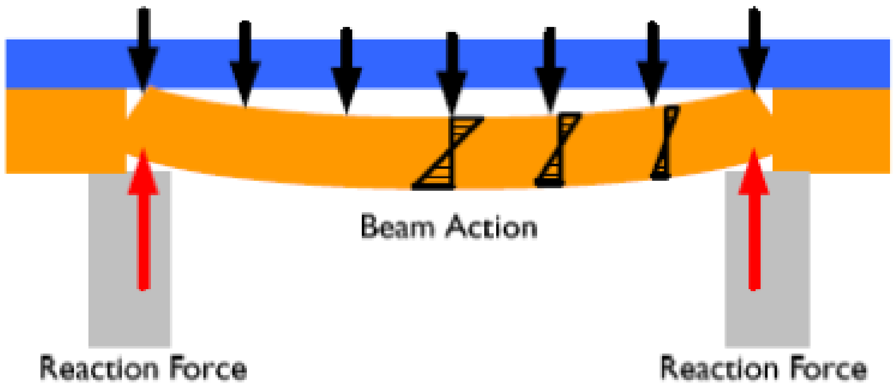
➢ I-Beam, U-Beam, T-Beam, Box

➢ Segmentally Prestressed Box Beam

Types: Beam/Girder Bridges

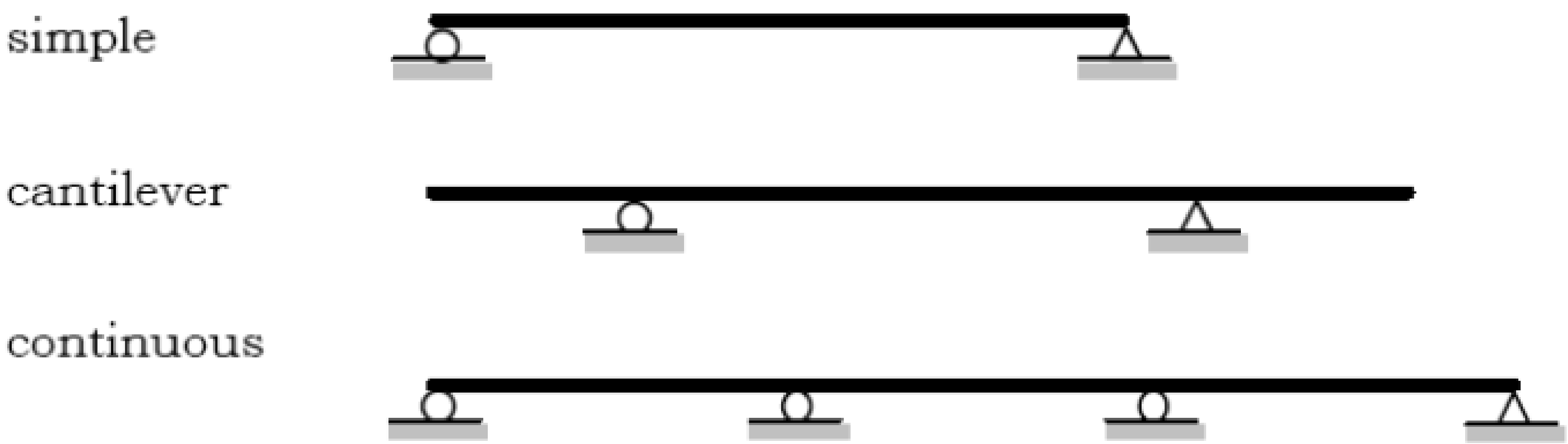
- The most basic type of bridge
- Typically consists of a beam simply supported on each side by a pier and can be made continuous later
- Carry load in Shear and Flexural bending
- Usually used for Short and Medium spans
- Decks and girder usually act together to support the entire load in highway bridges
- Typically inexpensive to build





Types: Beam/Girder Bridges

- Currently, most of the beam bridges are precast (in case of RC and PC), steel, or prefabricated
- Simply-supported
- Cantilever
- Continuous



Types: Beam/Girder Bridges

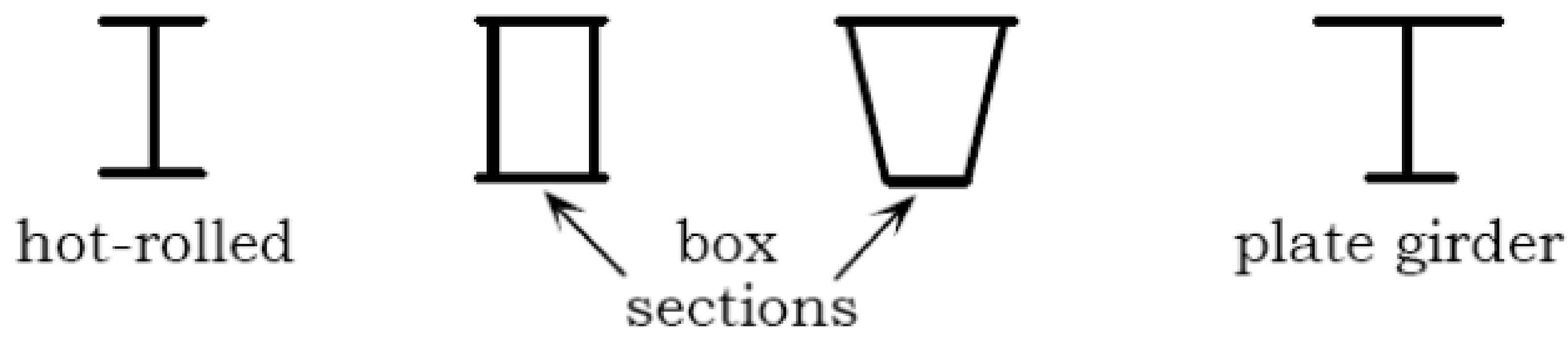
Raftsundet Bridge

Types: Beam/Girder Bridges

Confederation Bridge

Types: Beam/Girder Bridges

Steel sections may be hot-rolled shapes (for short-span bridge), Box section (medium span), or Plate Girder (medium span)



Types: Beam/Girder Bridges

Steel Box Girder Bridge

Types: Beam/Girder Bridges

- Upper: Steel Plate Girder Bridge
- Lower: Prestressed Concrete I-Girder Bridge

Types: Beam/Girder Bridges

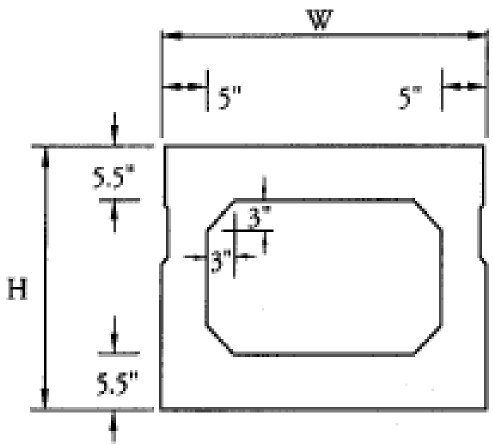
Steel Plate Girder

Types: Beam/Girder Bridges

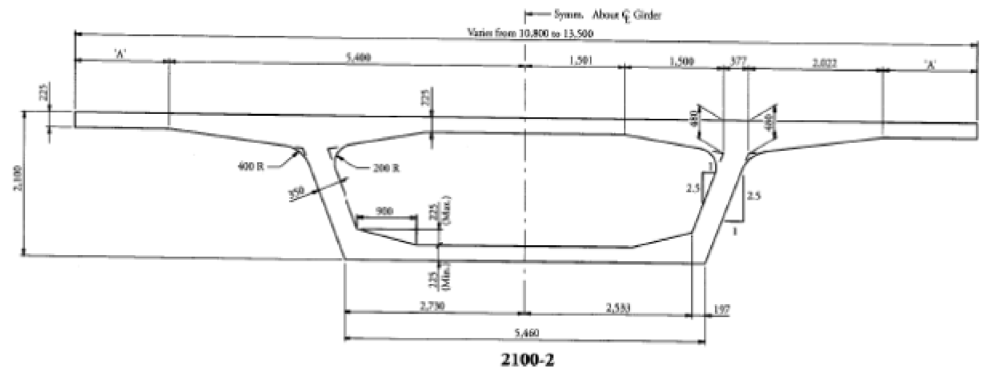
Steel Plate Girder Bridge

Types: Beam/Girder Bridges

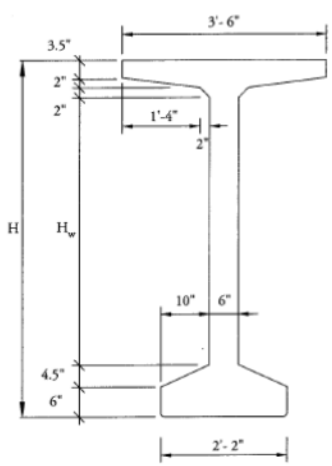
- **Prestressed Concrete Precast sections**



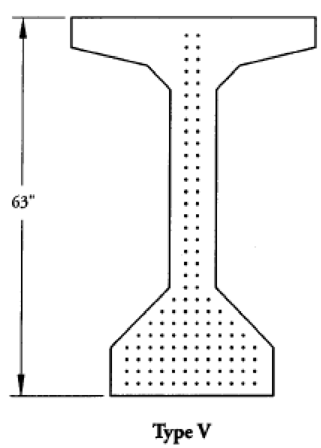
Box Girder



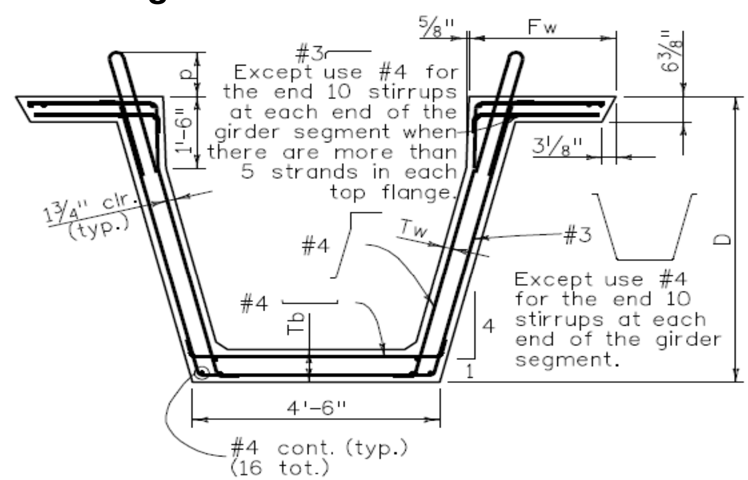
Segmental Box Girder



AASHTO I Girder



Bulb Tee Girder



Tub (U) Girder

Types: Beam/Girder Bridges

Types: Beam/Girder Bridges

Types: Beam/Girder Bridges

Types: Beam/Girder Bridges

Post-Tensioned Prestressed Concrete are often found in the form of segmentally precast members

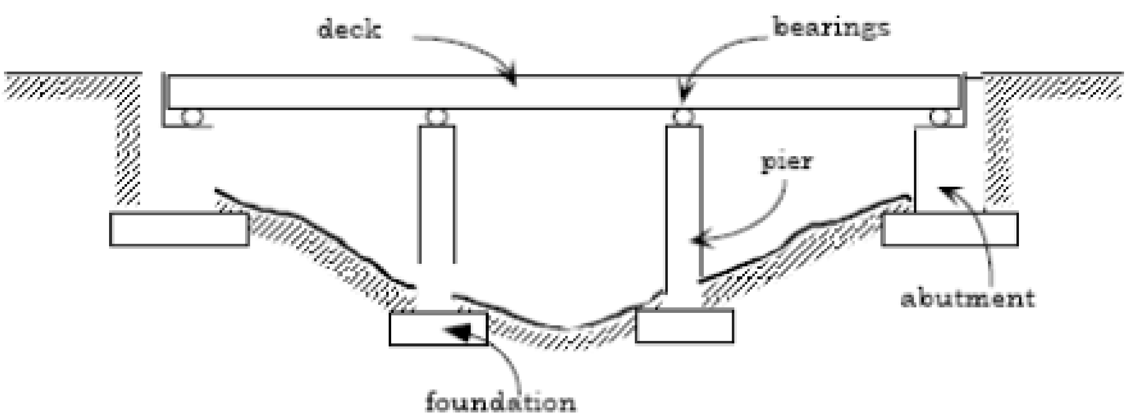
Types: Beam/Girder Bridges

Types: Beam/Girder Bridges

- Segmental construction may be constructed in 2 ways
- Cantilever Construction-construct from the pier equally on both sides
- Span-by-Span Construction-finish one span at a time

Components of Bridge

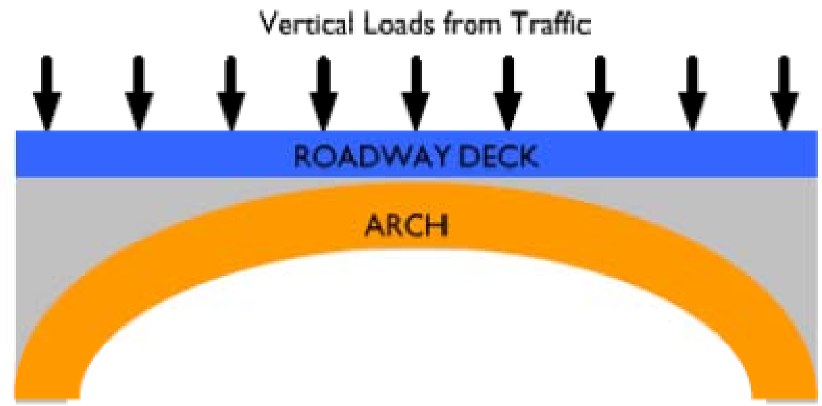
- Substructure
  - Foundation Pile/Spread Footing)
  - Pier (Column, wall)
  - Abutment
- Superstructure
  - Any structures above bearing which support the roadway
  - Wearing Surface

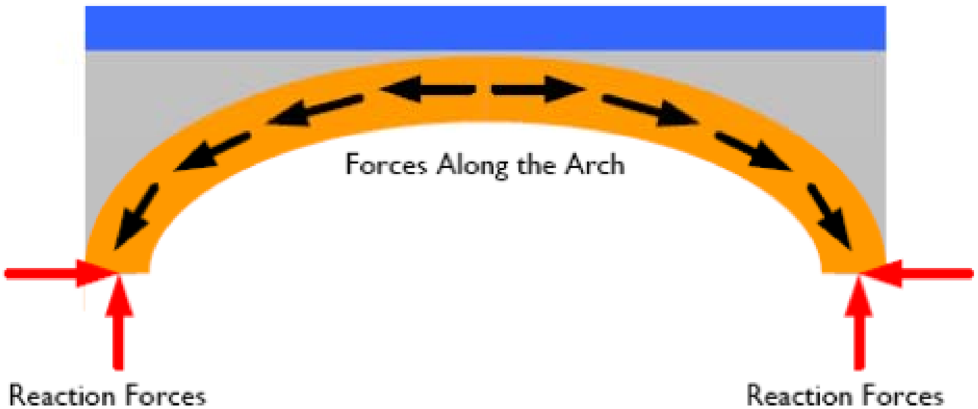


Components of Bridge

Types: Arch Bridges

- Arch action reduces bending
- Economical as compared to equivalent straight simply supported Girder or Truss bridge
- Suitable site is a Valley with arch foundations on a DRY ROCK SLOPES
- Conventional curved arch rib has high Fabrication and Erection costs
- Arch is predominantly a Compression member. Buckling must be worked to the detail so as to avoid reductions in allowable stresses.





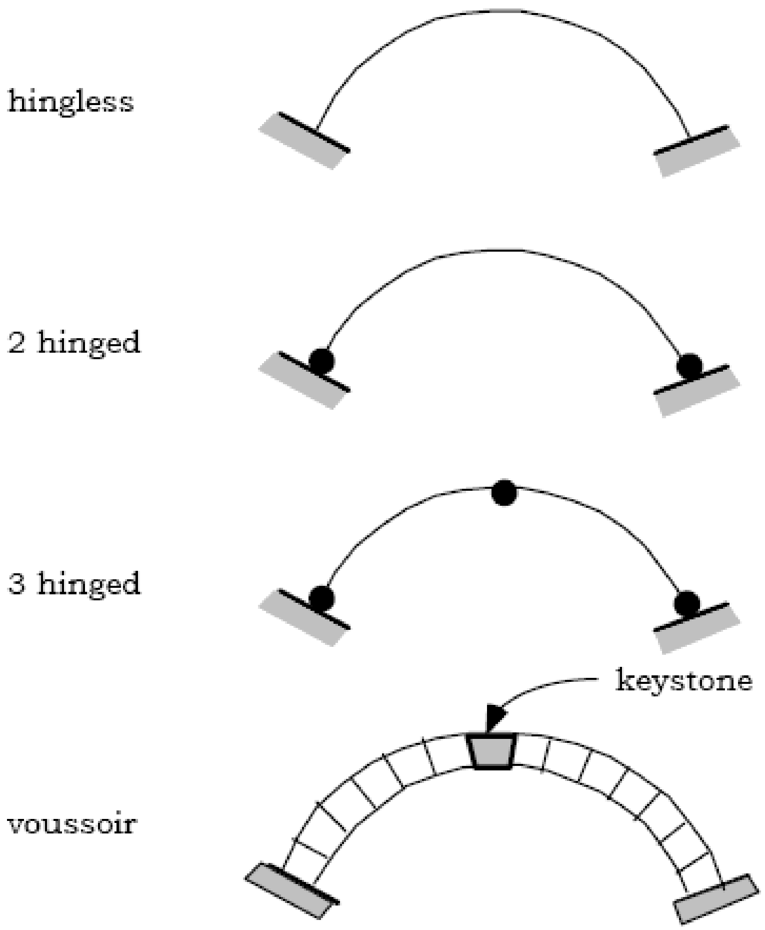
Types: Arch Bridges

Arch profiles

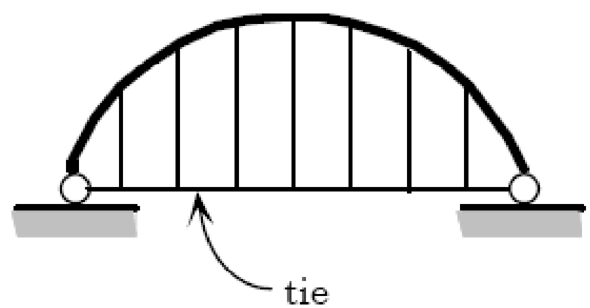
- Semi-circle (has vertical reaction force only)
- Flat arch (has vertical and horizontal forces at the support)
- Tied arch (tie resists tension force)

Types: Arch Bridges

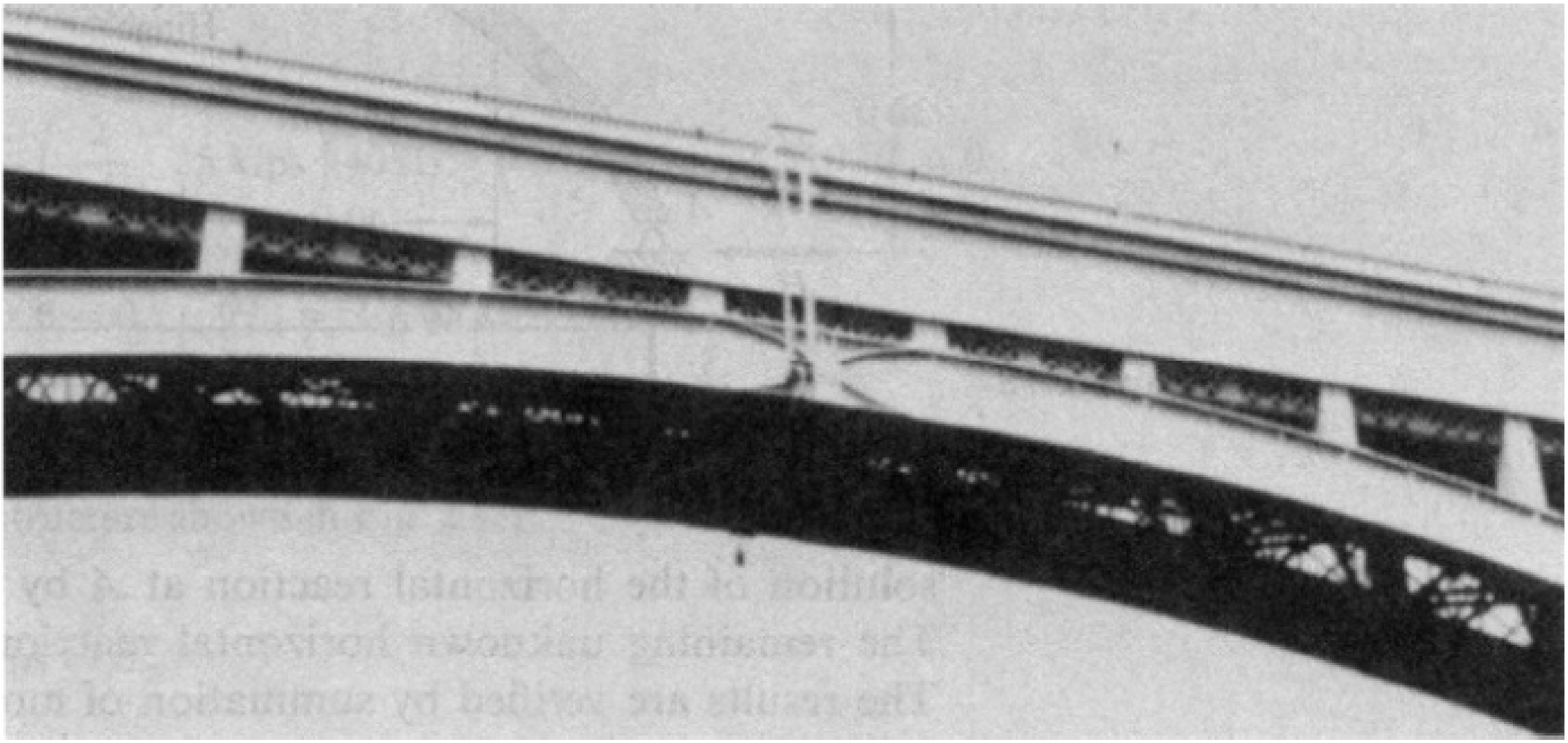
Arch Bridge



Hinge changes the degree of indeterminatcy in the structure



Types: Arch Bridges



Hinge Detail at the top of an arch bridge

Types: Arch Bridges

- Materials: Masonry, Timber, Concrete (Reinforced/ Prestressed), Steel

Types: Arch Bridges

Masonry Arch Bridge

Ponte Fabricio and Ponte Cestio (65 BC), Tiberina Island, Italy

Types: Arch Bridges

Masonry Arch Bridge

Zhaozhou Bridge (605 BC), China

Types: Arch Bridges

Masonry Arch Bridge

Types: Arch Bridges

- Bixby Bridge (1932), California, USA, 320 ft span
- Concrete arch

Types: Concrete Arch Bridge

Enz Bridge (1961), Mülacker, Germany, 46 m span, Concrete arch

Types: Arch Bridges

Prestressed Concrete Arch

Natchez Trace Parkway Bridge (1994), Tennessee, USA, 502 m span

Types: Arch Bridges

Salginatobel Bridge is a reinforced concrete arch bridge. It was constructed across an alpine valley in Schiers, Switzerland 1930. The bridge arch is 133 metres (436 ft) long in total.

Types: Steel Arch Bridge

Sydney Harbor Bridge (1938), Sydney, Australia, parabolic arch, 503 m span

Types: Steel Arch Bridge

Lupu Bridge, Shanghai, China

Types: Steel Arch Bridge

Chaotianmen Bridge, Chongqing, China

Chaotianmen Bridge, which spans the Yangtze River in Chongqing, China, is the world's longest arch bridge, opened on April 29, 2009. It has a main span of 552 metres (1,811 ft) and a total length of 1,741 m (5,712 ft)

Types: Steel Arch Bridge

Wushan Yangtze River Bridge, China

Types: Steel Arch Bridge

Hoover Dam bridge has a length of 1,900 feet (579 m) and a 1,060 ft (320 m) span. This is the first concrete-steel composite arch bridge built in the United States. The twin arch ribs are connected by steel struts. The composite design used concrete for the arch and columns and steel for the roadway deck.

Types: Steel Arch Bridge

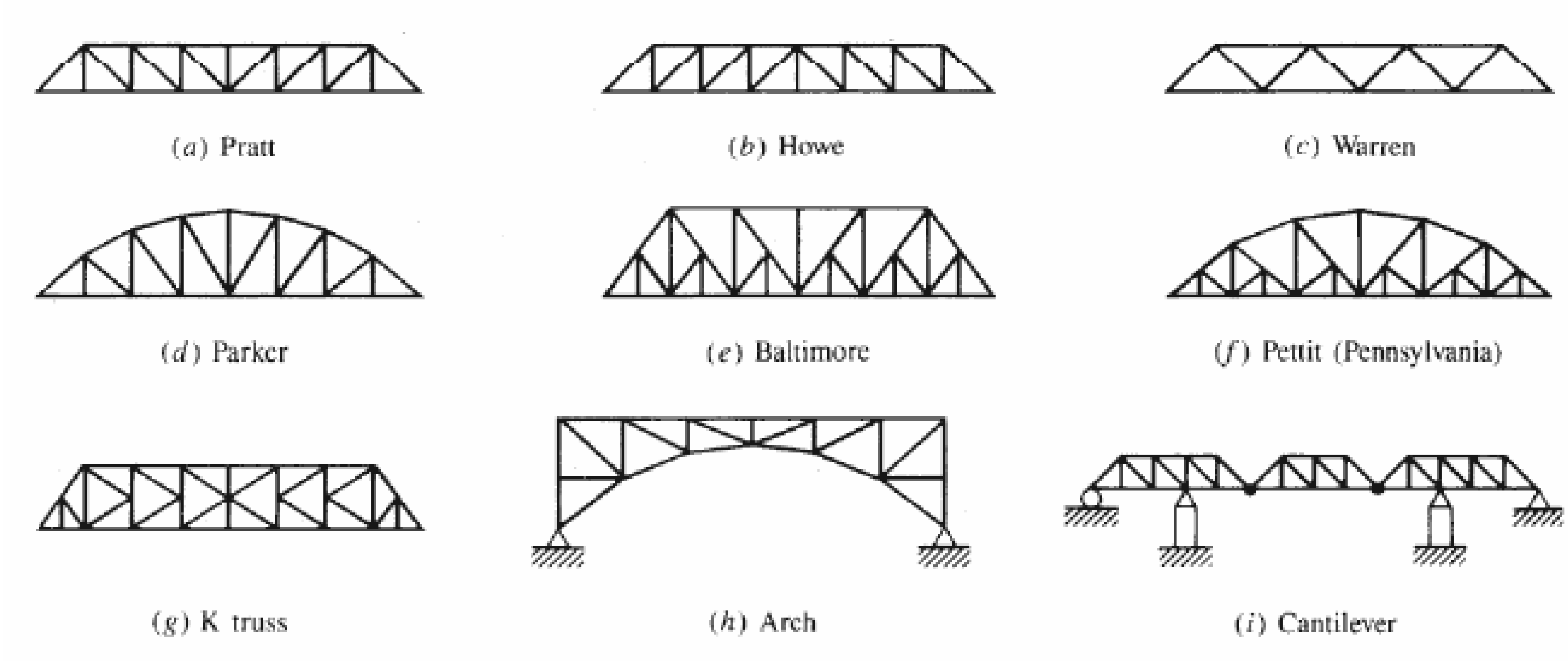
The New River Gorge Bridge is a steel arch bridge 3,030 feet (924 m) long over the New River Gorge near Fayetteville, West Virginia. The arch is 1,700 feet (518 m) long. The roadway of the New River Gorge Bridge is 876 feet (267 m) above the New River.

Types: Truss Bridges

- The primary member forces are axial loads
- The open web system permits the use of a greater overall depth than for an equivalent solid web girder, hence reduced deflections and rigid structure
- Both these factors lead to Economy in material and a reduced dead weight
- These advantages are achieved at the expense of increased fabrication and maintenance costs
- Truss bridges are not used extensively due to its high maintenance and fabrication costs.
- The truss is instead being used widely as the stiffening structure for the suspension bridges due to its acceptable aerodynamic behavior.

Types: Truss Bridges

Steel Truss can be of beam type, arch type, or cantilever type depending on the primary mechanisms



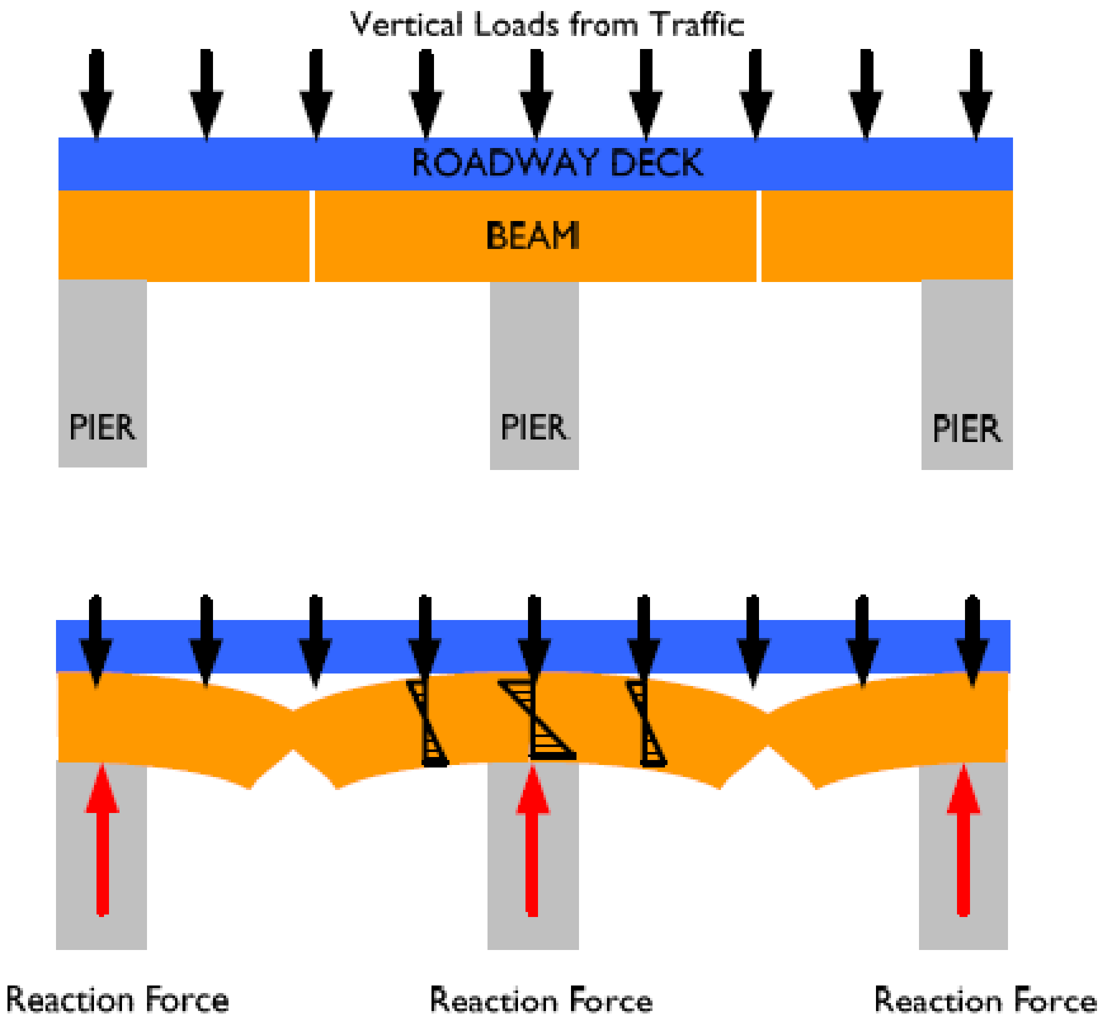
Types: Truss Bridges

- Some types of truss bridges can also be considered as a “beam bridge” when looked globally

Types: Cantilever Bridges

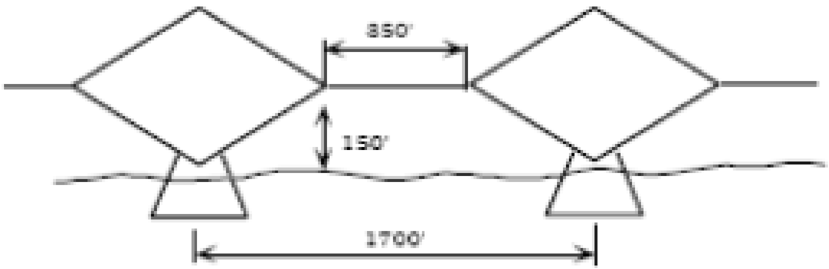
- In a cantilever bridge, the roadway is constructed out from the pier in two directions at the same time so that the weight on both sides counterbalance each other
- Notice the larger section at the support to resist negative moments

Types: Cantilever Bridges



Types: Cantilever Bridges

- Steel Truss Cantilever
- Prestressed Concrete Segmental Cantilever Beam



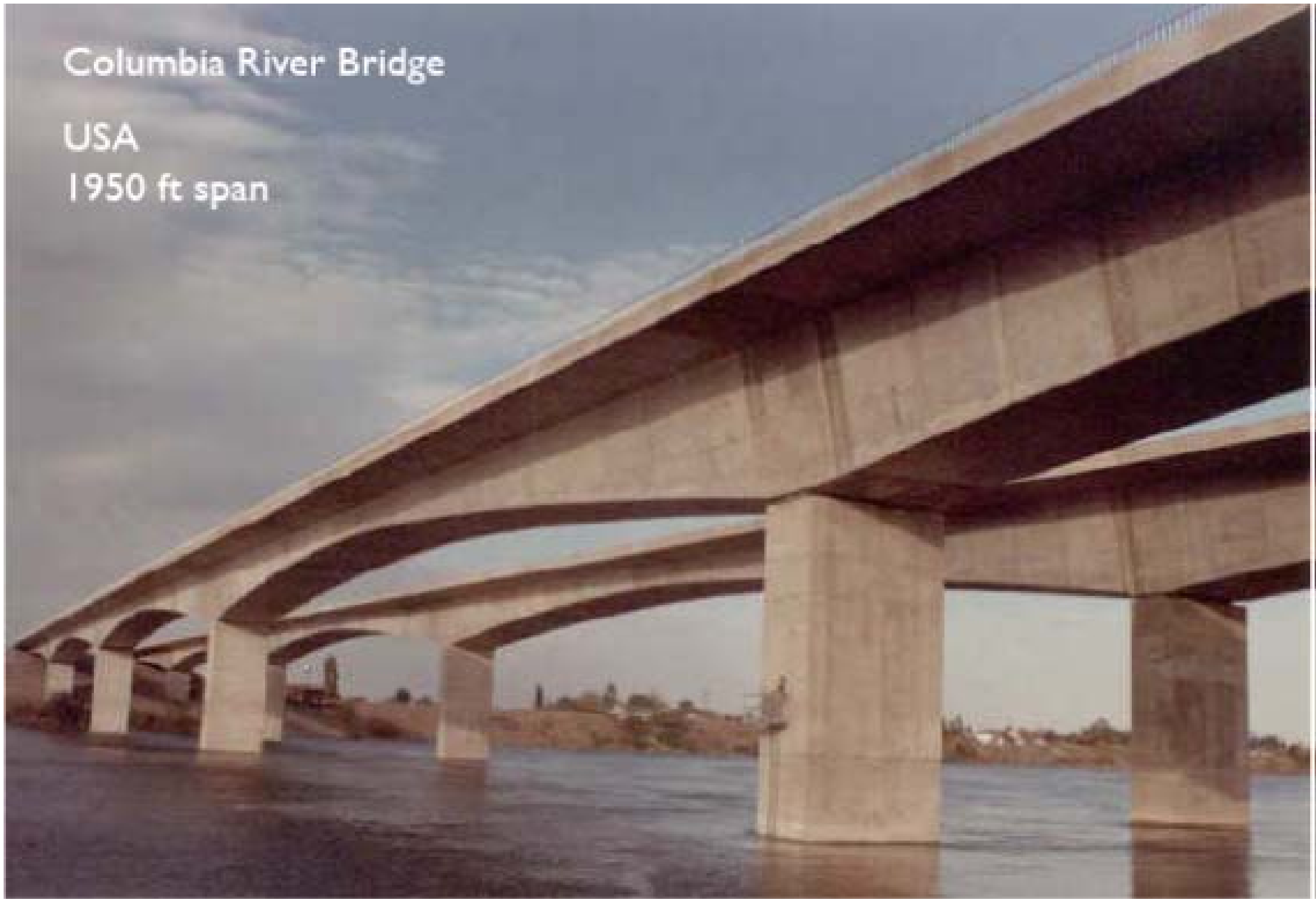
Firth of Forth Bridge (1890), Scotland
521m span

Types: Steel Truss Cantilever Bridge

Types: Steel Truss Cantilever Bridge

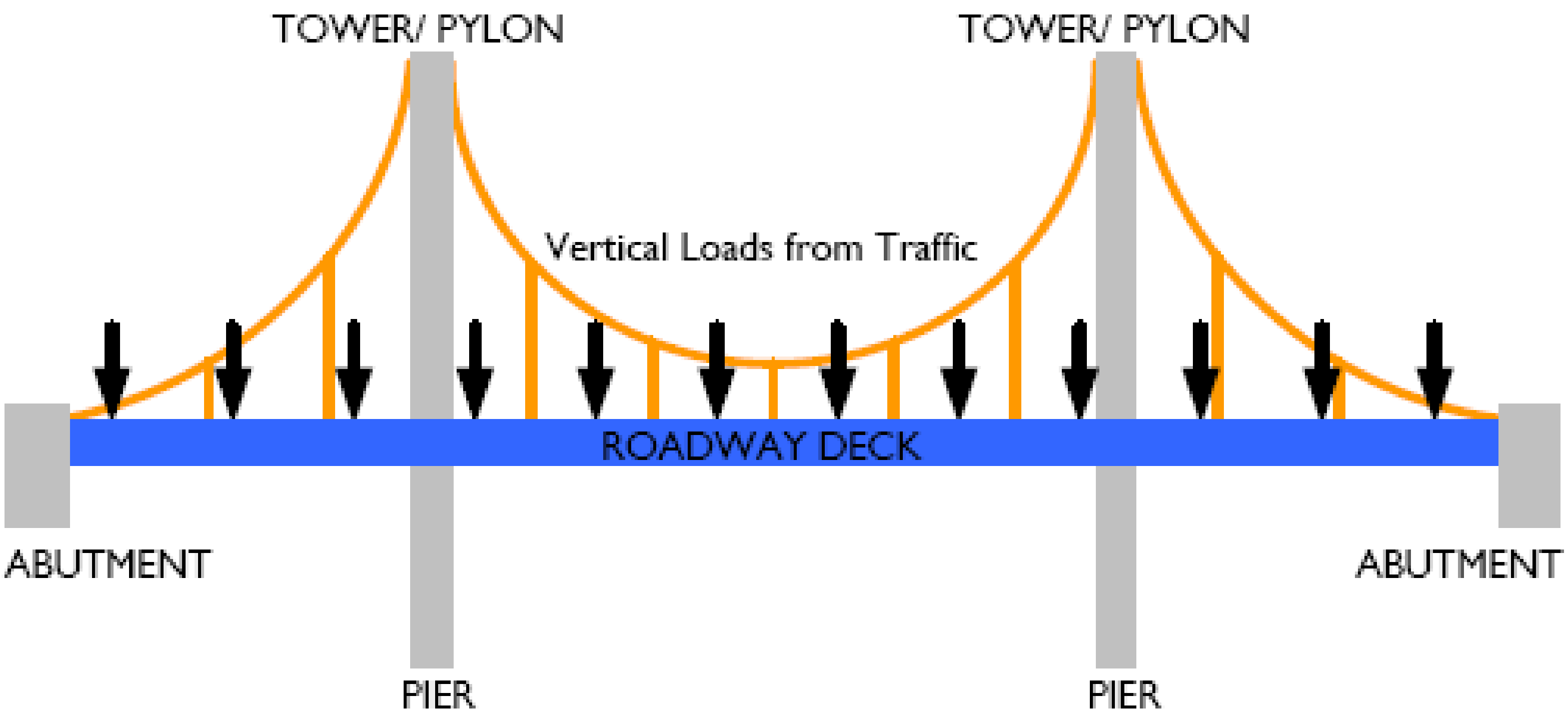
Types: Cantilever Bridges

Prestressed Concrete Segmental Cantilever Beam

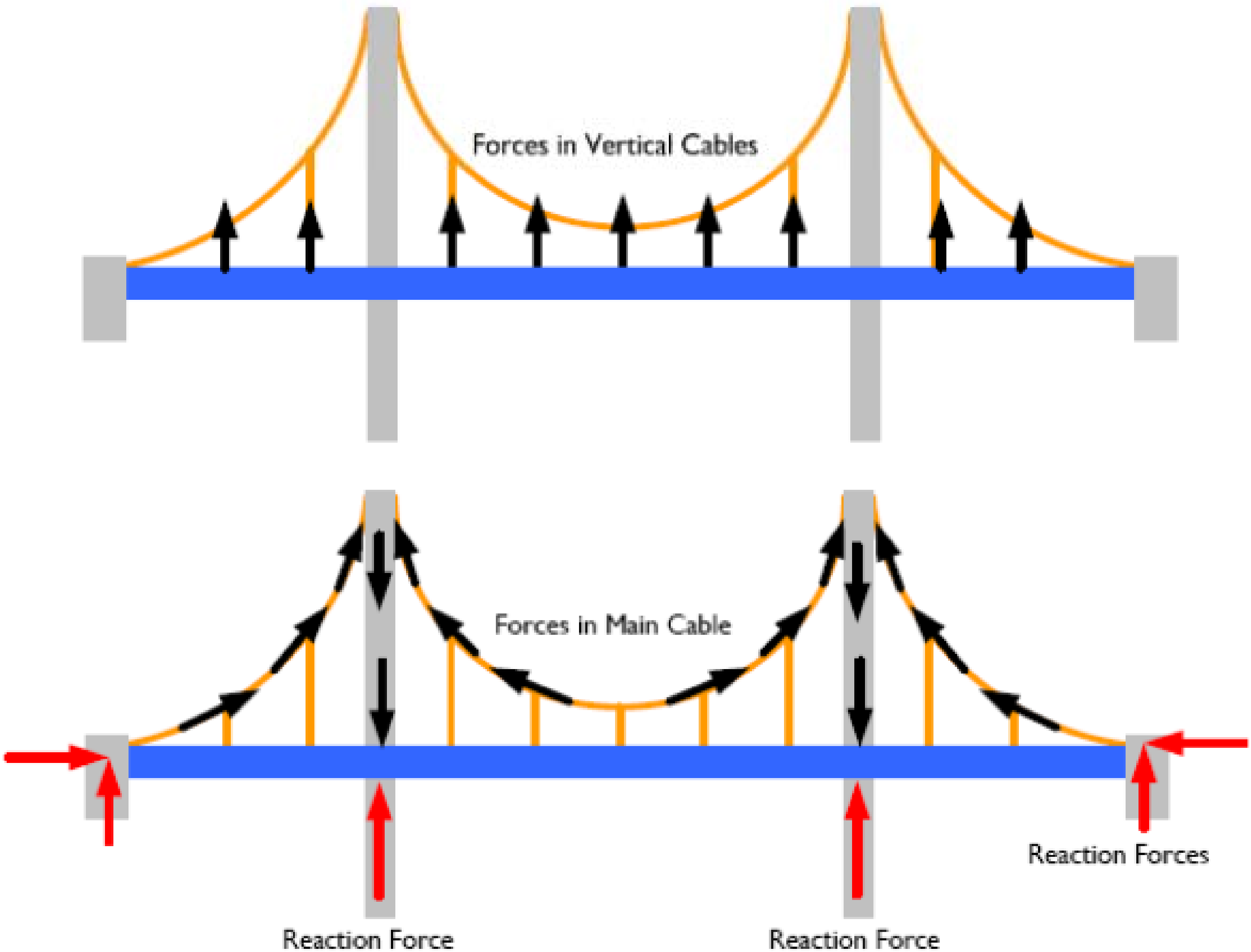


Types: Suspension Bridge

- Suspension bridge needs to have very strong main cables
- Cables are anchored at the abutment; abutment has to be massive



Types: Suspension Bridge



Types: Suspension Bridge

- Major element is a flexible cable, shaped and supported in such a way that it transfers the loads to the towers and anchorage
- This cable is commonly constructed from High Strength wires
- The deck is hung from the cable by Hangers constructed of high strength ropes in tension
- The main cable is stiffened either by a pair of stiffening trusses or by a system of girders at deck level.
- The main structure is elegant and neatly expresses its function.

Types: Suspension Bridge

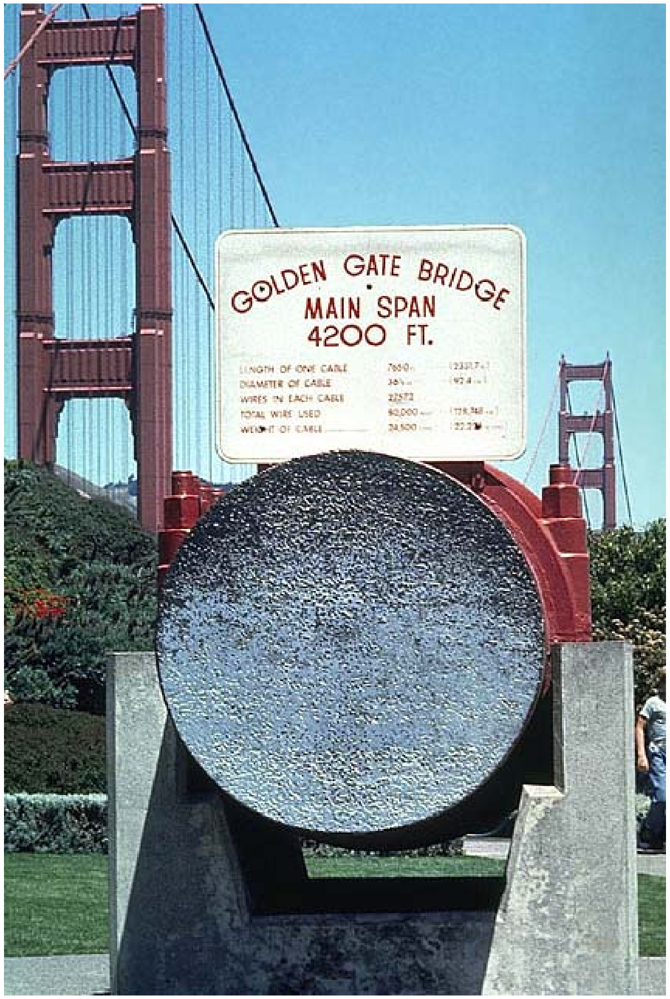
Anchor of a suspension bridge

Types: Suspension Bridge

Golden Gate Bridge spans the Golden Gate. The main span is 4200 ft. The Golden Gate Bridge was built between 1933 and 1937.

Types: Suspension Bridge

Golden Gate Bridge, California, USA

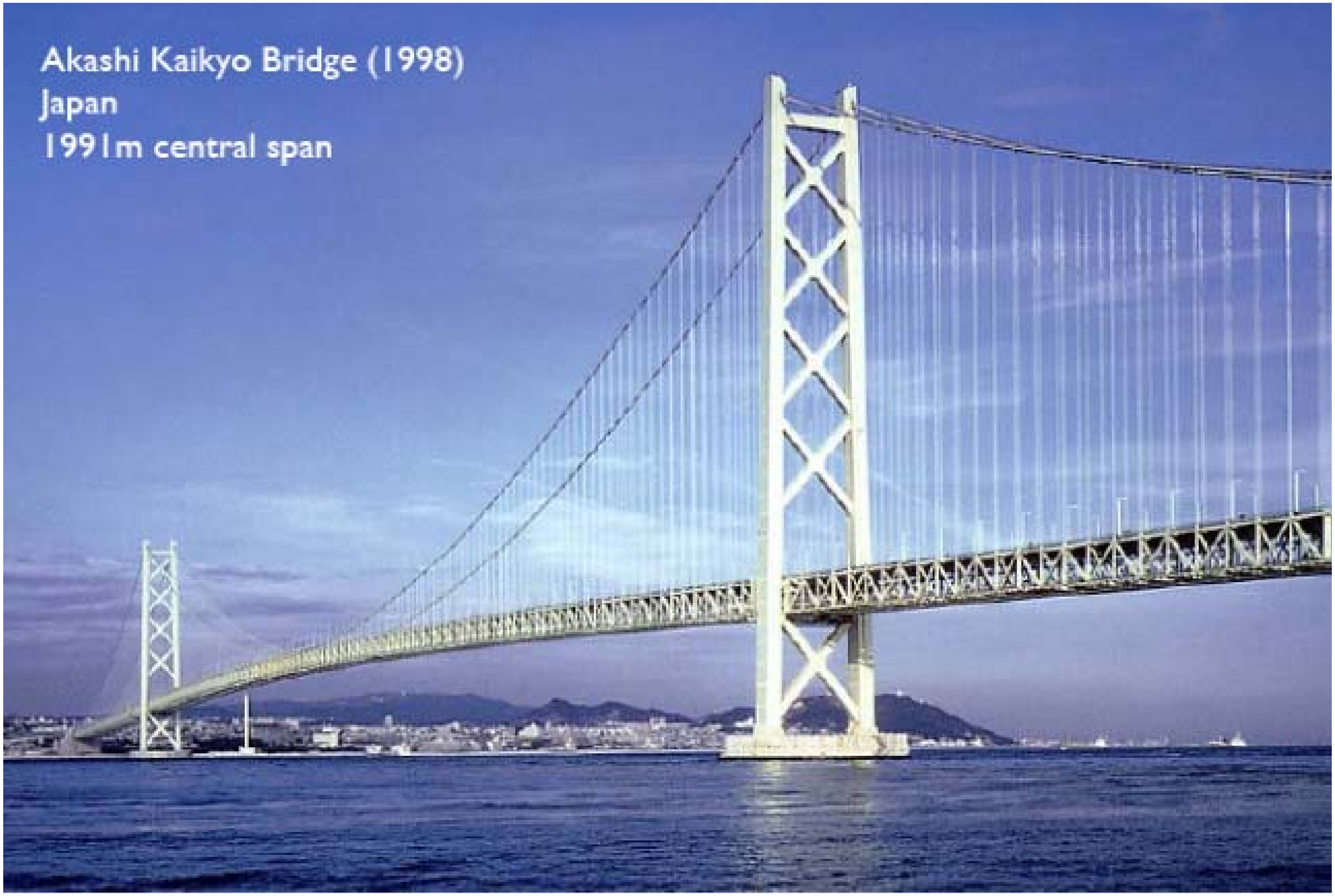


Types: Suspension Bridge

Mackinac Bridge (1957)
Michigan, USA
1158 m span

Tacoma Narrows Bridge (1940)
Washington, USA
2800 ft span

Types: Suspension Bridge

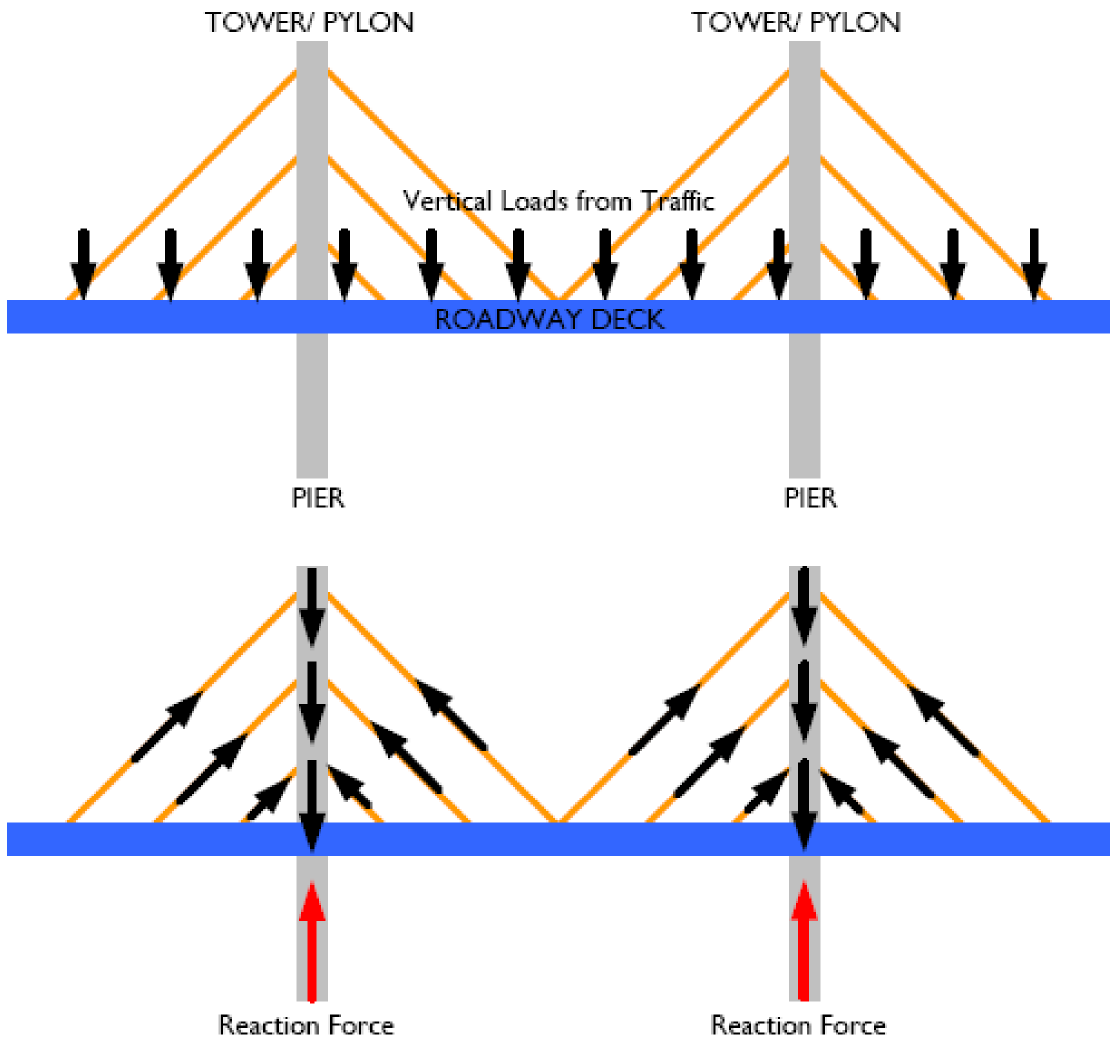


Types: Suspension Bridge

Royal Gorge Bridge (1929) Colorado, USA
286 m main span

Types: Cable-Stayed Bridge

- Pylon – Resisting compression from anchorage
- Main span beam – Resisting the horizontal forces of cable and local bending
- Cable – Transfer the load of main beam to pylon



Types: Cable-Stayed Bridge

- The use of high strength cables in tension leads to economy in material, weight, and cost..
- As compared with the stiffened suspension bridge, the cables are straight rather than curved. As a result, the stiffness is greater
- The cables are anchored to the deck and cause compressive forces in the deck. For economical design, the deck must participate in carrying these forces
- All individual cables are shorter than full length of the superstructure. They are normally constructed of individual wire ropes, supplied complete with end fittings, prestretched and not spun.
- There is a great freedom of choice in selecting the structural arrangement
- Less efficient under Dead Load but more efficient in support Live Load. It is economical over 100-350m, some designer would extend the upper bound as high as 800m

Types: Cable-Stayed Bridge

- Cable-stayed bridge uses the prestressing principles but the prestressing tendons are external of the beam
- All the forces are transferred from the deck through the cables to the pylon
- Roadway deck can be:
  - (Prestressed) Concrete Box Deck
  - Steel Box Deck
  - Steel Truss Deck

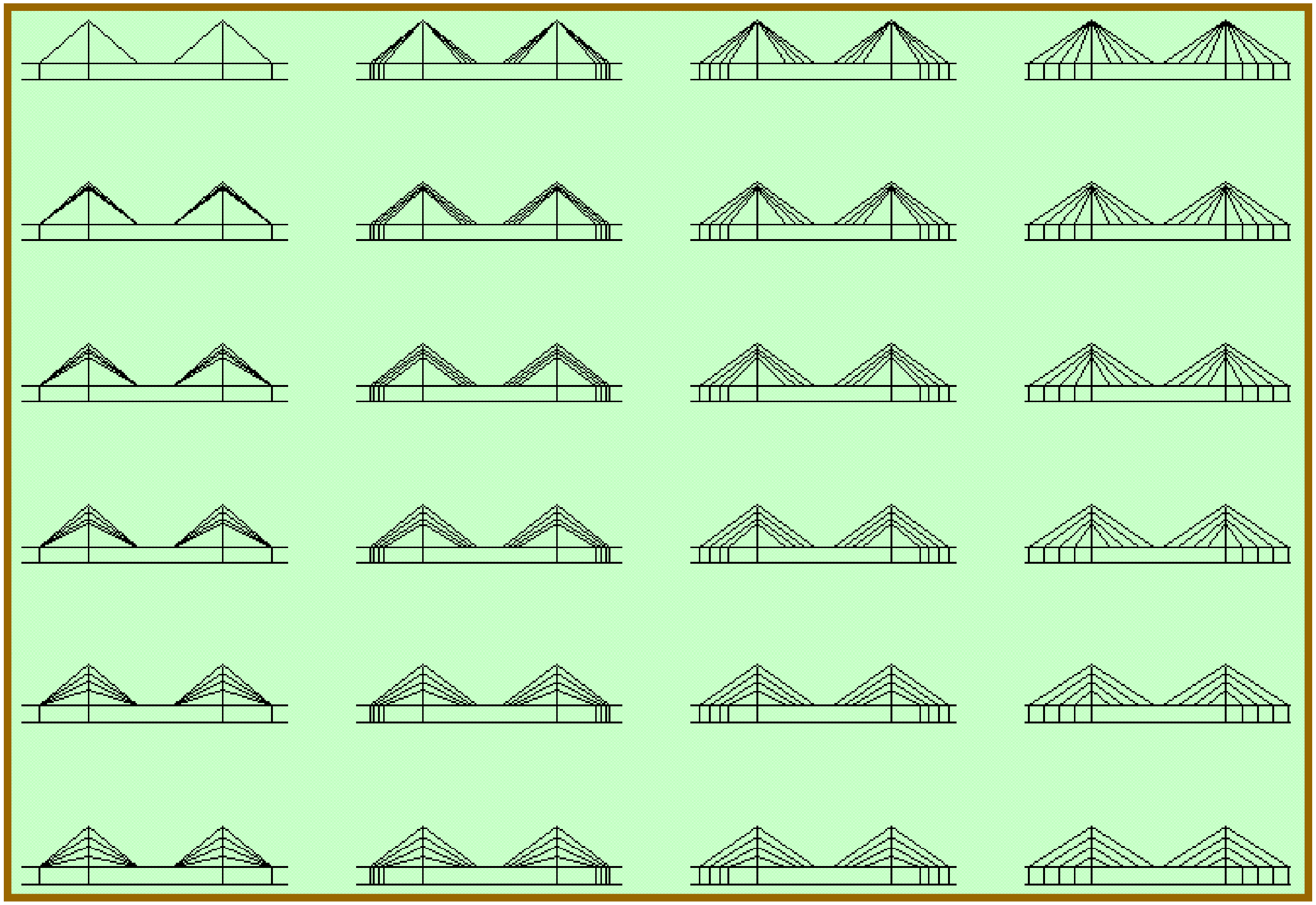
Types: Cable-Stayed Bridge

Types of Cable-Stayed Bridges

- Twin, single and multi-tower
- Sparse cables and dense cables
- Single and double cable plane
- Radiation-shaped, harp-shaped, semi-radiation shaped
- Self-anchored and ground anchored
- Concrete, Steel and Concrete Composite, Steel

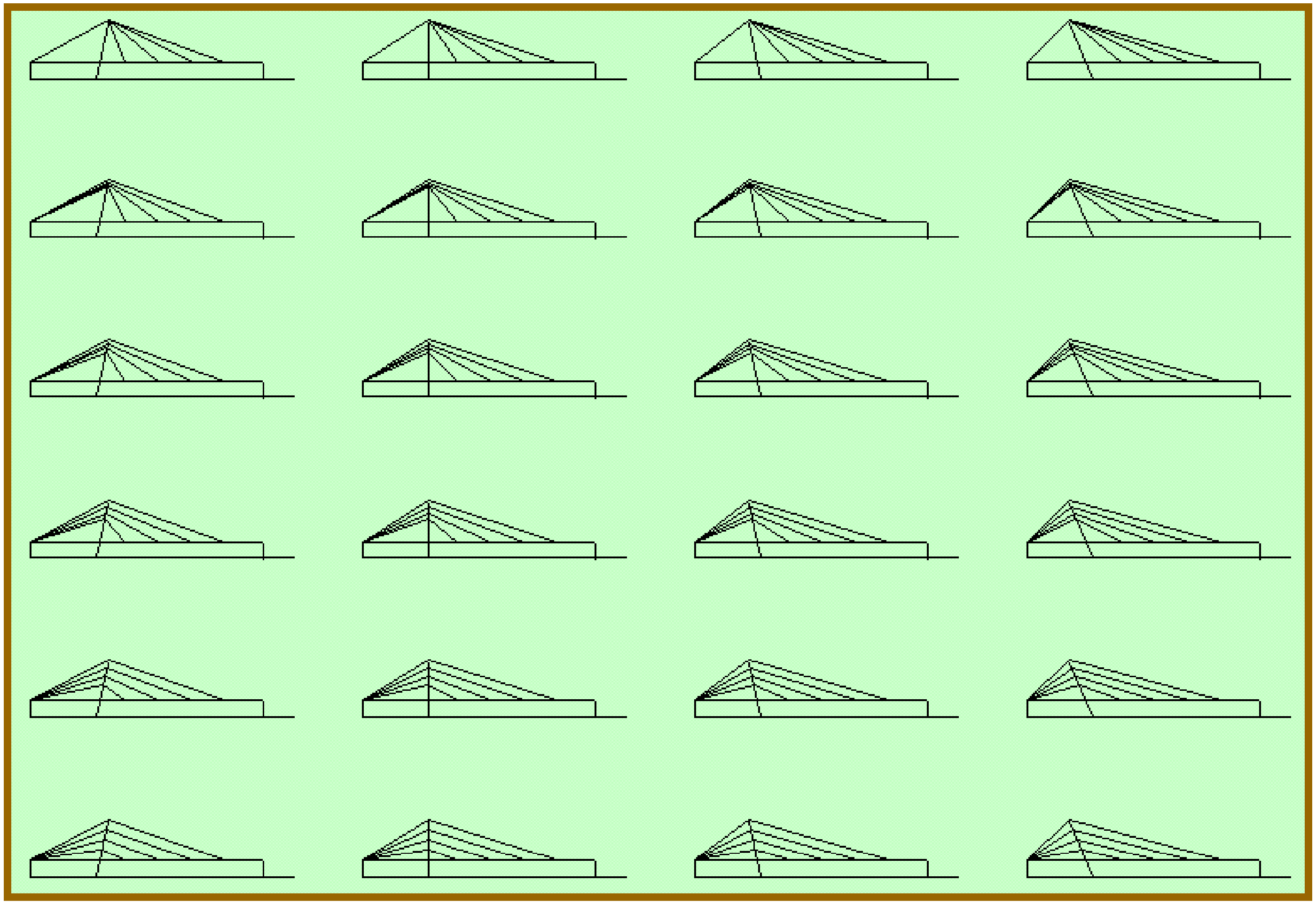
Types: Cable-Stayed Bridge

Double-Towers



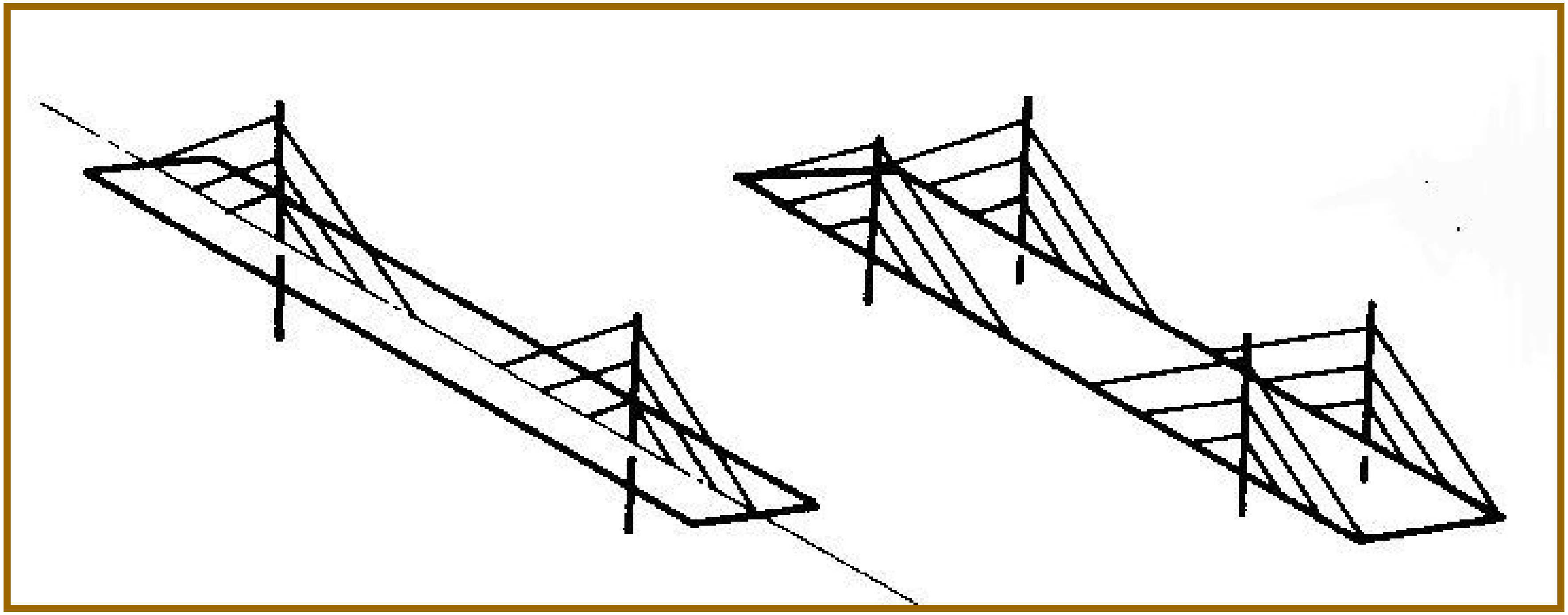
Types: Cable-Stayed Bridge

Single-Towers



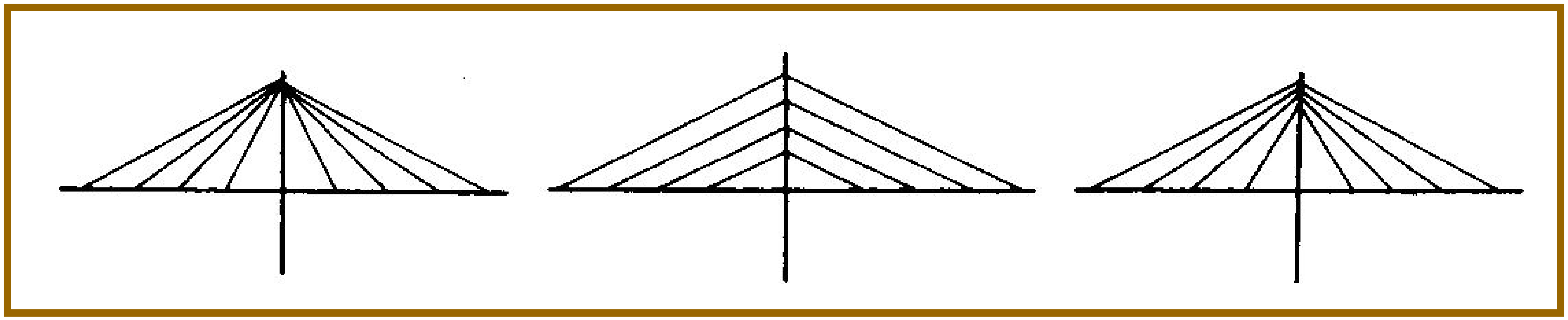
Types: Cable-Stayed Bridge

Single cable plane Double cable plane



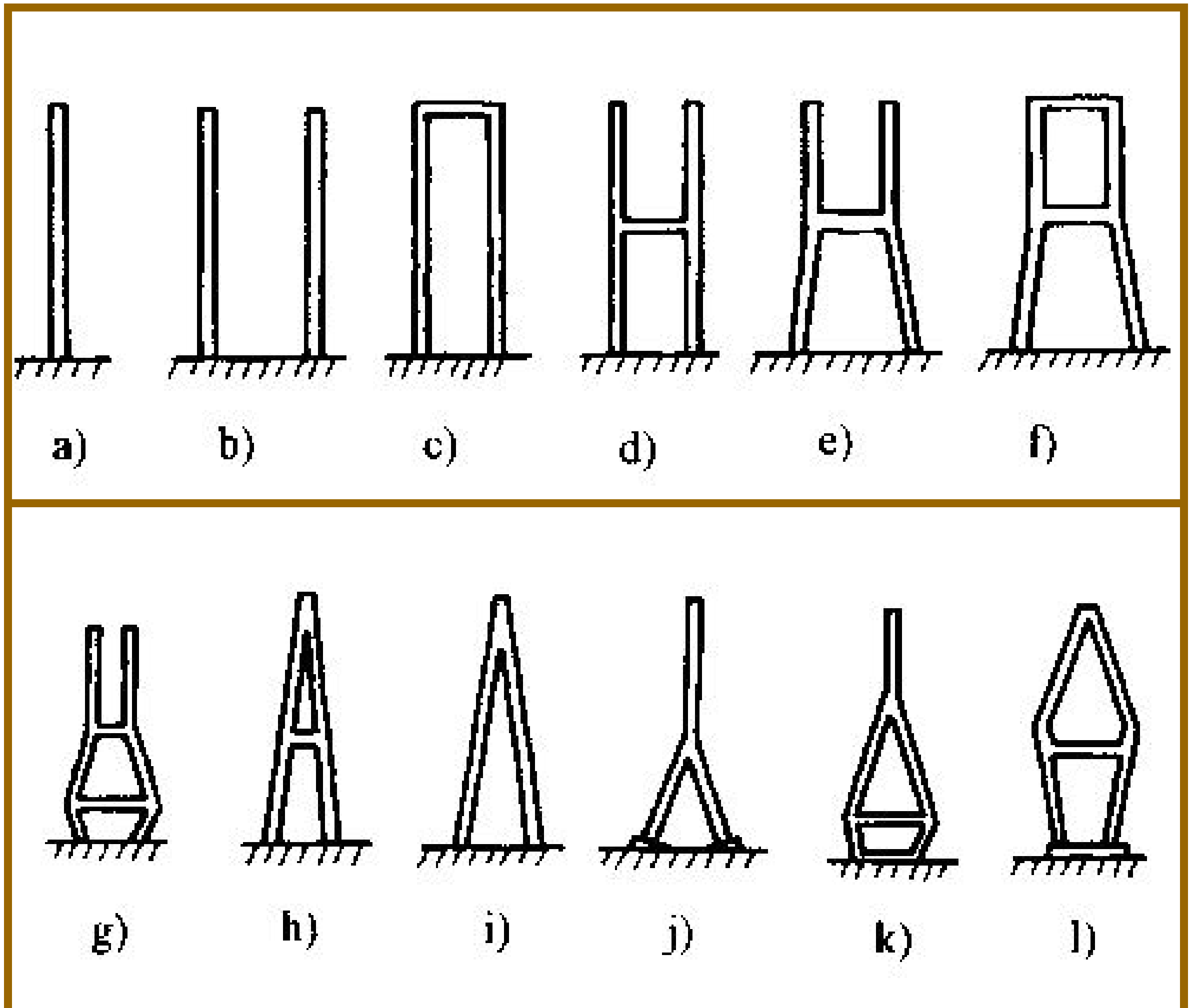
Types: Cable-Stayed Bridge

Radiation-Shaped, Harp-Shaped, Semi-Radiation Shaped



Types: Cable-Stayed Bridge

Shapes in Transverse Direction



Types: Cable-Stayed Bridge

Sutong Bridge in Sutong, China, the world longest cable stayed bridge with a main span of approximately 3570 feet and the tallest tower of 1004 ft.

Types: Cable-Stayed Bridge

Sutong Bridge

Types: Cable-Stayed Bridge

Sutong Bridge

At the time I visited the bridge site, they were about to erect the last closure segment.

Types: Cable-Stayed Bridge

Sutong Bridge

All steel box segments were fabricated in a mill a few miles away. Segments were barged to the bridge site and lifted by the cranes on the bridge.

Types: Cable-Stayed Bridge

Stonecutters Bridge is a high level cable-stayed bridge which spans the Rambler Channel in Hong Kong. The bridge is the second longest cable-stayed span (3340 ft) in the world.

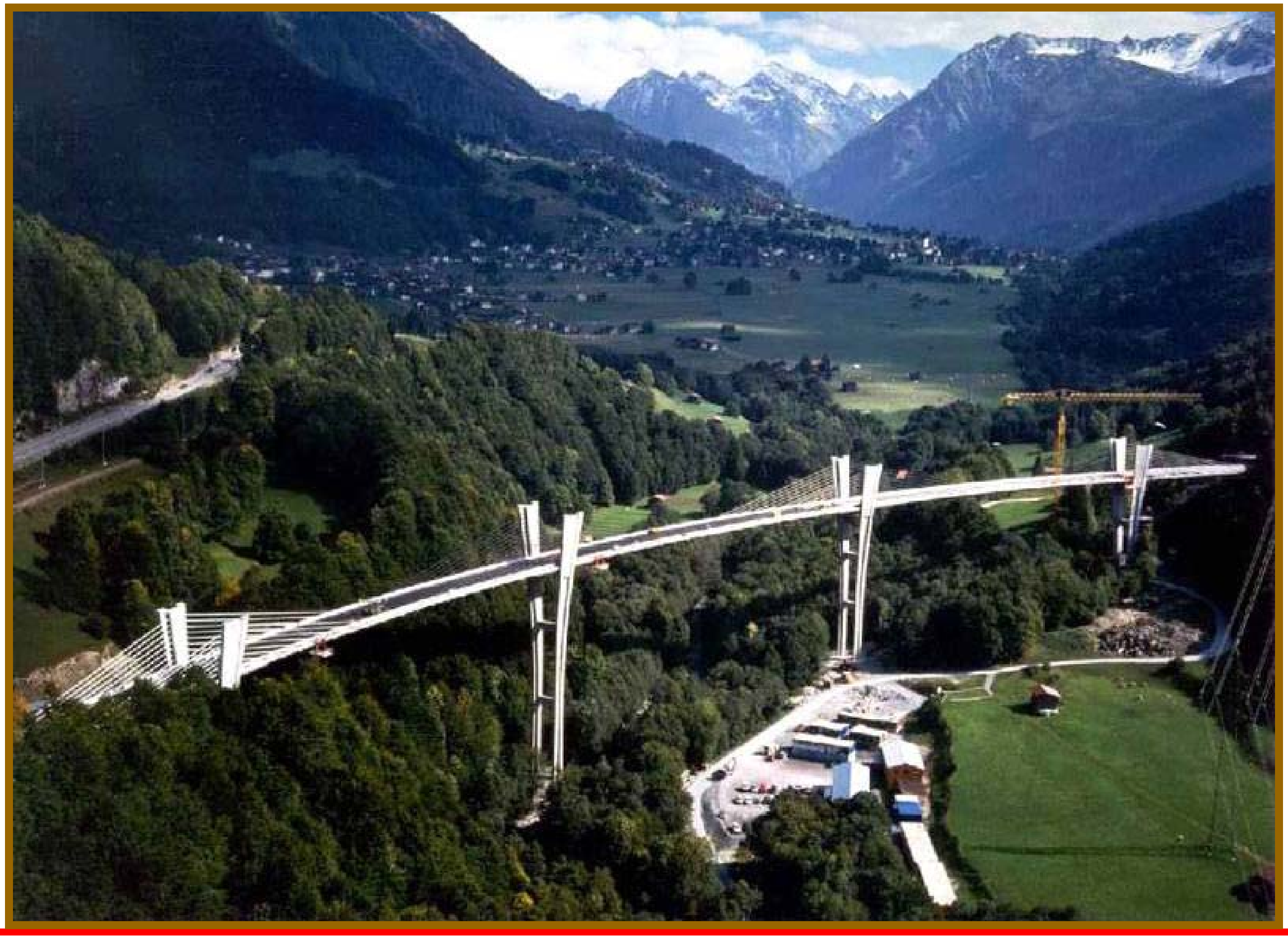
Types: Cable-Stayed Bridge

The Millau Viaduct is a cable-stayed bridge near Millau in southern France. It is the tallest bridge in the world, with one mast's summit at 343.0 metres (1,125 ft).

The Millau Viaduct consists of an eight-span steel roadway supported by seven concrete pylons. The six central spans each measure 342 m (1,122 ft) with the two outer spans measuring 204 m (669 ft).

Types: Cable-Stayed Bridge

The Sunniberg Bridge is a cable-stayed road bridge near Klosters in Switzerland. It is notable because of its innovative design and aesthetically pleasing appearance. Span lengths: 59 m - 128 m - 140 m - 134 m - 65 m



Types: Cable-Stayed Bridge

- Construction sequence

➢ Construct Pylons

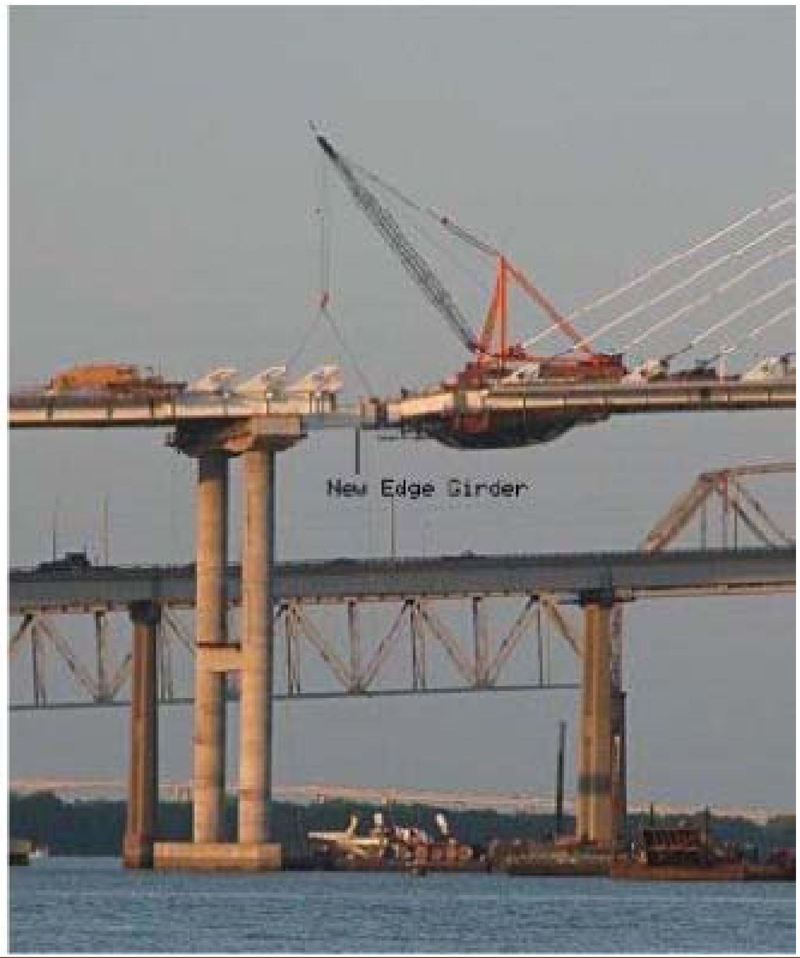
Types: Cable-Stayed Bridge

- Construction sequence

➢ Erect the deck away from the pylon in both of the pylons

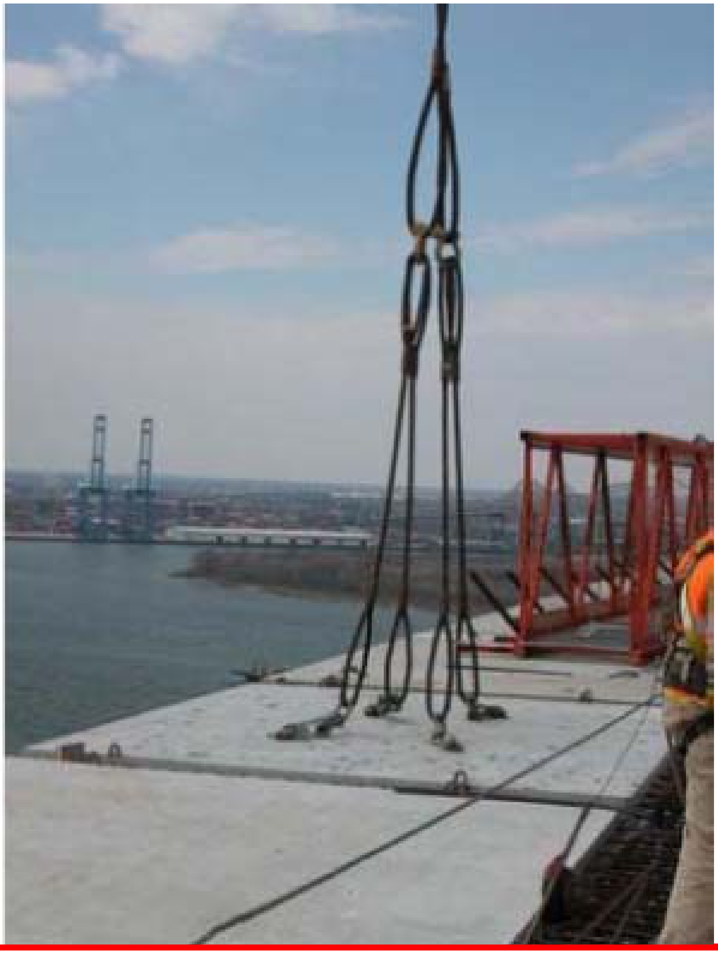
Types: Cable-Stayed Bridge

- Construction sequence
  - Join the cable-stayed sections with the back piers



Types: Cable-Stayed Bridge

- Construction sequence
  - The concrete roadway deck is laid



Types: Cable-Stayed Bridge

- Construction sequence

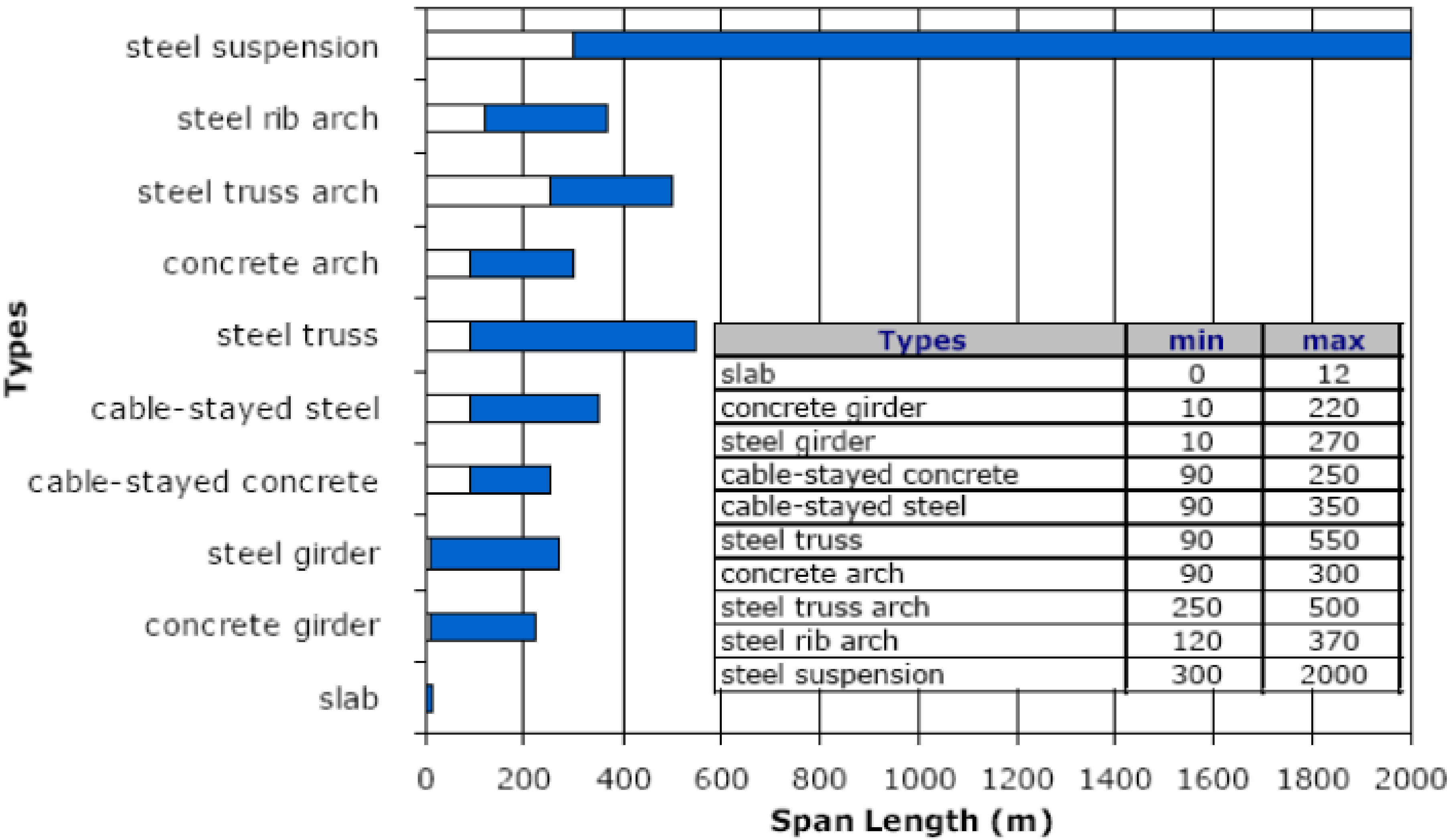
➢ Finally, join the two cantilevers at the midspan

Which type should I use?

Consider the followings:

- Span length
- Bridge length
- Beam spacing
- Material available
- Site conditions (foundations, height, space constraints)
- Speed of construction
- Constructability
- Technology/Equipment available
- Aesthetics
- Cost
- Access for maintenance

Span Length



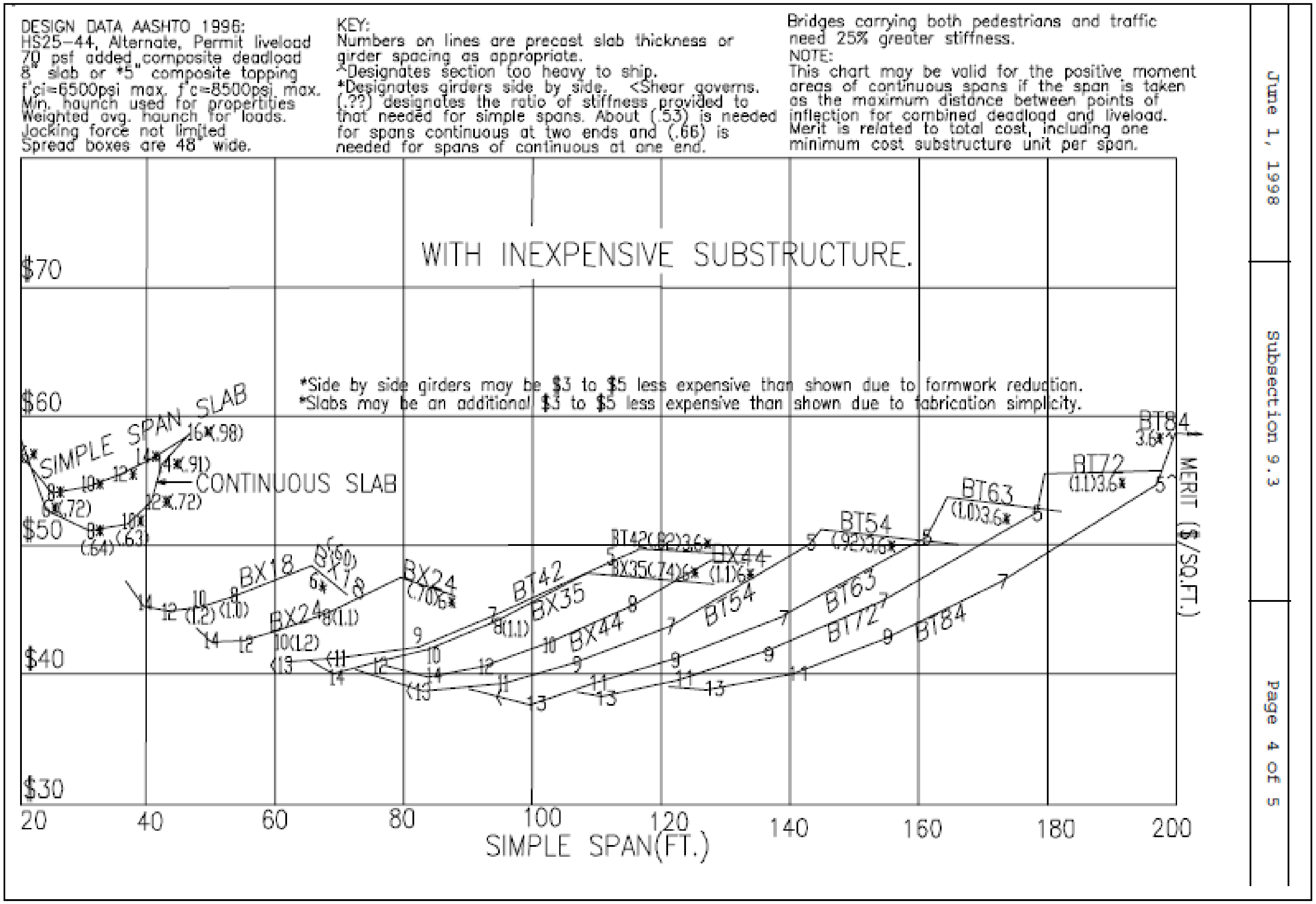
Types	min	max
slab	0	12
concrete girder	10	220
steel girder	10	270
cable-stayed concrete	90	250
cable-stayed steel	90	350
steel truss	90	550
concrete arch	90	300
steel truss arch	250	500
steel rib arch	120	370
steel suspension	300	2000

Cost vs. Span Length

- The span length may be influenced by the cost of superstructure and substructure
- If the substructure cost is about 25% of total cost; shorter span is more cost-effective
- If the substructure cost is about 50% of total cost; longer spans are more economical

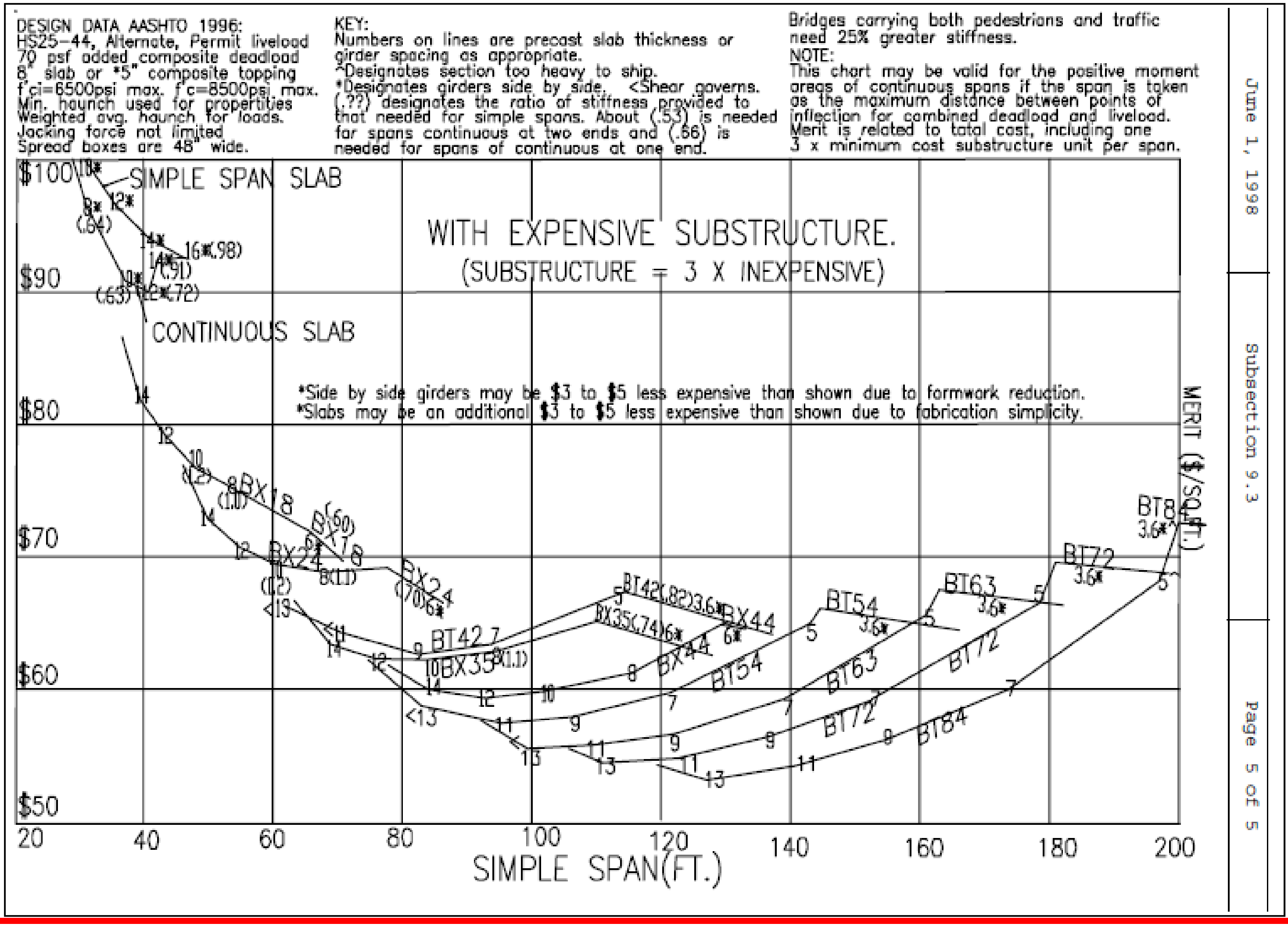
Cost vs. Span Length

CDOT Bridge Design Manual



Cost vs. Span Length

CDOT Bridge Design Manual



Cost vs. Span Length

Substructure here is expensive compared with superstructure

If the water is shallow, substructure is inexpensive compared with the superstructure

Access for Maintenance

- Total Cost = Initial Cost + Maintenance Cost
- Bridge should be made easy to inspect and maintain
- Maintenance cost may govern the selection of bridge
  - Steel bridge needs a lot of maintenance in coastal regions
  - Concrete bridge usually require the least maintenance

Materials

- Steel
- Concrete
  - Cast-in-place
  - Precast
- Material choice depends on the cost of material at the bridge site
- Shipping cost from fabricators

Speed of construction

- In urban areas, the construction of bridge may disrupt traffic
  - Prefabricated/Precast member are the only choice
  - Substructure construction may disrupt traffic more than the superstructure erection; may consider longer spans

Site Requirement

- Is the bridge straight or curved
- Precast I-Girder cannot be curved
- Segmental prestressed can have slight curve
- Cast-in-place
- Shipping of prefabricated pieces to site
- Is shipping channel required?
- Is the temporary falsework
- required? Can it be done with
- the site conditions?

Site Requirement

Requirement for shipping channel leads to long span bridge

Site Requirement

In the Millau Aqueduct, the superstructure was completed inland and pushed into the span

Aesthetics

- An ugly bridge, however safe, serviceable, and inexpensive, is not a good bridge
- Long span bridge over a river can be a landmark; thus, aesthetics should be an important factor
- Bridge should blend with the environment
- Smooth transition between members
- Avoid unnecessary decorations
- Bridge should have an appearance of adequate strength

Aesthetics

- Determinant of bridge's appearance (in order of importance)
  - ➢ Vertical and Horizontal geometry relative to surrounding topography and other structures
  - ➢ Superstructure type: arch , girder, etc…
  - ➢ Pier placement
  - ➢ Abutment placement
  - ➢ Superstructure shape, parapet and railing
  - ➢ Pier shape
  - ➢ Abutment shape
  - ➢ Color, surface texture, ornamentations
  - ➢ Signing, Lighting, Lanscaping

Aesthetics

- Vertical and Horizontal Geometry

Aesthetics

Aesthetics

Aesthetics

Aesthetics

El Alamillo Bridge
Seville, Spain, 200m span

Highland Bridge, Denver: The Highland Bridge over I-25 opened December 16, 2007. Its gleaming white, triple rib steel architecture certainly fits the need for drama. Three steel ribs arch 70 feet above the ground and span 320 feet over the highway.

Aesthetics

The **Denver Millennium Bridge:**

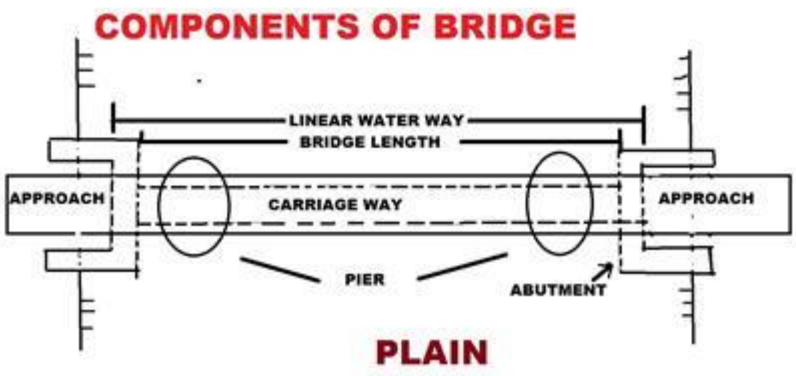
Its 200-foot (61m) white tapered steel mast rises above Denver's northwestern skyline, connected to the bridge deck and foundation anchors by steel cables. This unique footbridge crosses railroad tracks and the regional light rail system, climbing no higher than 25 feet (8m) above street level, thereby minimizing the height pedestrians must climb.

Aesthetics

Millennium Footbridge (2002), London, UK, 144 m span

Components of a bridge

The components of a bridge are divided into three parts are as foundation, substructure and superstructure. The foundation is designed to carry the total load of the structure. It also includes the foundation of the piers and abutments. The structural components are known as the components of the bridge up to the level of bearing. It includes abutments, piers and wing walls. The components which are situated above the level of bearing are known as super structural components. It includes beams, girders, arches, cables, flooring and handrails.



Requirements of bridge

Bridges are required to connect big towns and cities. It provides communication between two cities and provide business aspects. It also helps in the war time for the mobility of the Army. Bridges are required in the road or rail projects. where large numbers of accuracy are required. These projects take longer time for completing and required more accuracy and large number of planning and consideration. The economy of bridges depends upon the material used at the time of construction.

How to select a site for bridge construction?

Keep the following points in your mind while you are selecting a site for the bridge construction.

- Connected with roads
- Strong embankments on both sides
- Type of foundation
- Requirement of material and labor
- Flow of water
- Straight stretch of river
- Flow of river
- Width of river
- Connected with roads:
  If you are going to select a site for bridge construction then you should have good roads which are connected with each others. The road should be

connected either side of the river or any dam. The bed end of the approaches should be dry and hard.

- Strong embankment on both sides:

  The bridges are constructed where the embankment present on the both sides of the river. The embankment should be firm on both the upstream and downstream side of the river. The firm should be strong, solid, permanent, straight and well defined. The embankment should not disturb at the time of the flood.

- Type of foundation:

  Good foundations is required to make a bridge. At the bed of the river, the foundation should be good and have reasonable depth for the substructure. The property of soil should be good.

- Requirement of material and labor:

  In bridge construction, it is the important point to the requirement of good material and labor near the site. Material and labor should be easily available near the site. Due to this the transportation charges for labor and material goes minimum. So, if you are doing planning for bridge construction, then this point will help you more. Always search for good material and labor near the site.

- Flow of water:

  The flow of water should be perpendicular to the center line of the bridge. These types of crossings are also known as the right angle crossing or square crossing. Sometime it is also known as the normal crossing. Following points are kept in mind while measuring the flow of water.

  There should be a smooth flow of water

  i. The proper arrangement of segmental wing wall and return wall near the bridge construction. It permits the formation of eddies and cross currents are avoided.
  ii. It also provides the shortest length of the bridge span as well as the length of the pier.
  iii. The skew bridges are difficult to construct as compared to normal bridges.
  iv. The depth of bridge foundation is depends upon the type of design. Skew bridges are complicated in design. The maintenance of these types of bridges is also difficult.
  v. The passage of water under the skew bridges are not smooth. The piers have also resisted excessive water pressure.

- Straight stretch of river:

  The river should be a straight stretch at the upstream side and downstream side. It also allows the smooth and uniform flow of water. The curve stretch of river provides an irregular flow of water. So, the river should be a straight stretch at the upstream or downstream of the bridge. It creates many problems during the time of maintenance and construction.

- Flow of river:

  The construction of the bridge also depends on the flow of the river. If the velocity of flow is less than the particular value, the silting will occur and more particular value occurs in the river bed. The velocity of flow in the river should be medium according to the particular value. It also depends on the nature of the river bed. So, it is very important to check the river bed first, then start construction on it.

- Width of river:

  The width of the river also affects the construction of the bridge. If the width of the river is less, then you can construct economy and cheaper bridge on it. If the width of the river is more than you have to pay more for it. If you have a longer width river, then you have to go for it.

All the conditions are above are for an ideal site of the bridge. You cannot obtain any site which contains all the above qualities. So you have to construct bridges according to your requirement only. If you are satisfied with the present site, then its good for you. It is very important to study the design data of the particular bridge before the construction. Read all the terms and conditions before start construction of bridges.

UNIT - VI

WATER TANKS

In general there are three kinds of water tanks-tanks resting on ground, underground tanks and elevated tanks. The tanks resting on ground like clear water reservoirs, settling tanks, aeration tanks etc. are supported on the ground directly. The walls of these tanks are subjected to pressure and the base is subjected to weight of water and pressure of soil. The tanks may be covered on top.

The tanks like purification tanks, Imhoff tanks, septic tanks, and gas holders are built underground. The walls of these tanks are subjected to water pressure from inside and the earth pressure from outside. The base is subjected to weight of water and soil pressure. These tanks may be covered at the top. Elevated tanks are supported on staging which may consist of masonry walls, R.C.C. tower or R.C.C. columns braced together. The walls are subjected to water pressure. The base has to carry the load of water and tank load. The staging has to carry load of water and tank. The staging is also designed for wind forces.

From design point of view the tanks may be classified as per their shape-rectangular tanks, circular tanks, intze type tanks. spherical tanks conical bottom tanks and suspended bottom tanks.

Design requirement of concrete (I.S.I)

In water retaining structures a dense impermeable concrete is required therefore, proportion of fine and course aggregates to cement should be such as to give high quality concrete.

Concrete mix weaker than M200 is not used. The minimum quantity of cement in the concrete mix shall be not less than 300 kg/m^3.

The design of the concrete mix shall be such that the resultant concrete is sufficiently impervious. Efficient compaction preferably by vibration is essential. The permeability of the thoroughly compacted concrete is dependent on water cement ratio. Increase in water cement ratio increases permeability, while concrete with low water cement ratio is difficult to compact. Other causes of leakage in concrete are defects such as segregation and honey combing. All joints should be made water-tight as these are potential sources of leakage.

Design of liquid retaining structures is different from ordinary R.C.C, structures as it requires that concrete should not crack and hence tensile stresses in concrete should be within permissible limits.

A reinforced concrete member of liquid retaining structures is designed on the usual principles ignoring tensile resistance of concrete in bending. Additionally it should be ensured that tensile stress on the liquid retaining face of the equivalent concrete section does not exceed the permissible tensile strength of concrete as given in table

1. For calculation purposes the cover is also taken into concrete area.

Cracking may be caused due to restraint to shrinkage, expansion and contraction of concrete due to temperature or shrinkage and swelling due to moisture effects. Such restraint may be caused by –

(i) the interaction between reinforcement and concrete during shrinkage due to drying.

(ii) the boundary conditions.

(iii) the differential conditions prevailing through the large thickness of massive concrete.

Use of small size bars placed properly, leads to closer cracks but of smaller width. The risk of cracking due to temperature and shrinkage effects may be minimised by limiting the changes in moisture content and temperature to which the structure as a whole is subjected. The risk of cracking can also be minimised by reducing the restraint on the free expansion of the structure with long walls or slab founded at or below ground level, restraint can be minimised by the provision of a sliding layer. This can be provided by founding the structure on a flat layer of concrete with interposition of some material to break the bond and facilitate movement.

In case length of structure is large it should be subdivided into suitable lengths separated by movement joints, specially where sections are changed the movement joints should be provided.

Where structures have to store hot liquids, stresses caused by difference in temperature between inside and outside of the reservoir should be taken into account.

The coefficient of expansion due to temperature change may be taken as $11 \times 10^{-6}/°$ C and coefficient of shrinkage may be taken as 450×10^{-6} for initial shrinkage and 200×10^{-6} for drying shrinkage.

3. Joints in Liquid Retaining Structures. Joints are classified as given below.

(a) Movement Joints. There are three types of movement joints.

(i) ***Contraction Joint.*** It is a movement joint with deliberate discontinuity without initial gap between the concrete on either side of the joint. The purpose of this joint is to accommodate contraction of the concrete. The joint is shown in Fig. 1(a).

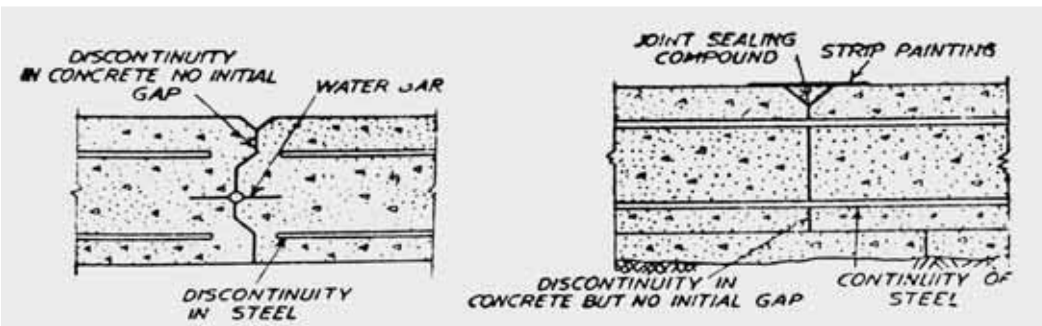


Fig. 1(a) Contraction joint

Fig. 1(b) Complete Contraction joint

A contraction joint may be either complete contraction joint or partial contraction joint. A complete contraction joint is one in which both steel and concrete are interrupted and a partial contraction joint is one in which only the concrete is interrupted, the reinforcing steel running through as shown in Fig. 1(b).

(ii) ***Expansion Joint.*** It is a joint with complete discontinuity in both reinforcing steel and concrete and it is to accommodate either expansion or contraction of the structure. A typical expansion joint is shown in Fig. 2.

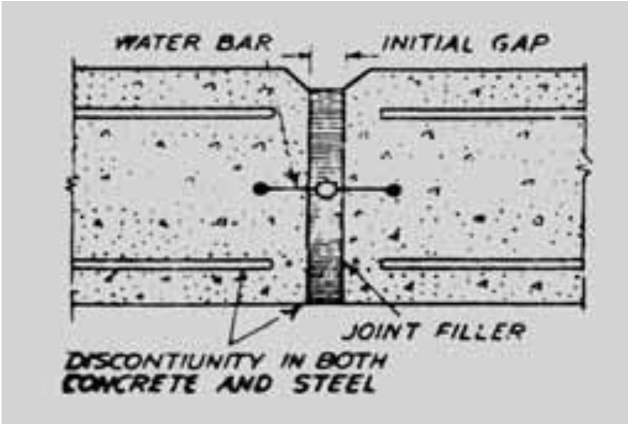


Fig. 2 Expansion joint

This type of joint requires the provision of an initial gap between the adjoining parts of a structure which by closing or opening accommodates the expansion or contraction of the structure.

(iii) ***Sliding Joint***. It is a joint with complete discontinuity in both reinforcement and concrete and with special provision to facilitate movement in plane of the joint. A typical joint is shown in Fig. 3. This type of joint is provided between wall and floor in some cylindrical tank designs.

(b) ***Construction Joint***. This type of joint is provided for convenience in construction. Arrangement is made to achieve subsequent continuity without relative movement. One application of these joints is between successive lifts in a reservoir wall. A typical joint is shown in Fig. 4.

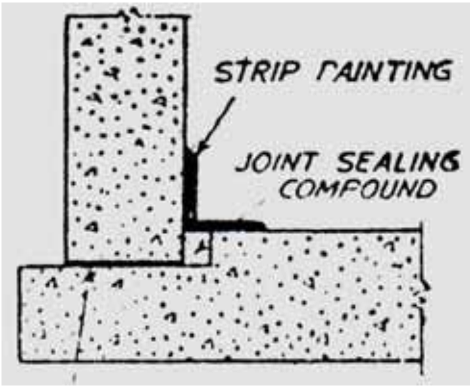


Fig. 3. Sliding joint

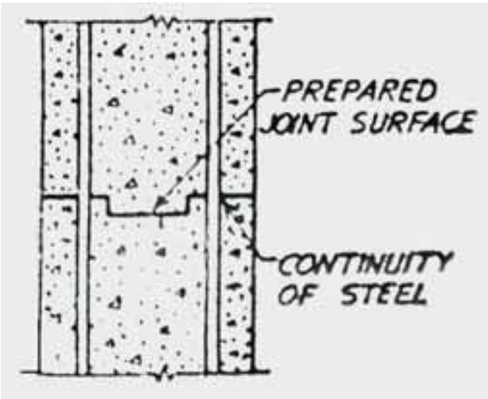


Fig. 4. Construction joint

The number of joints should be as small as possible and these joints should be kept from possibility of percolation of water.

(c) ***Temporary Open Joints***. A gap is sometimes left temporarily between the concrete of adjoining parts of a structure which

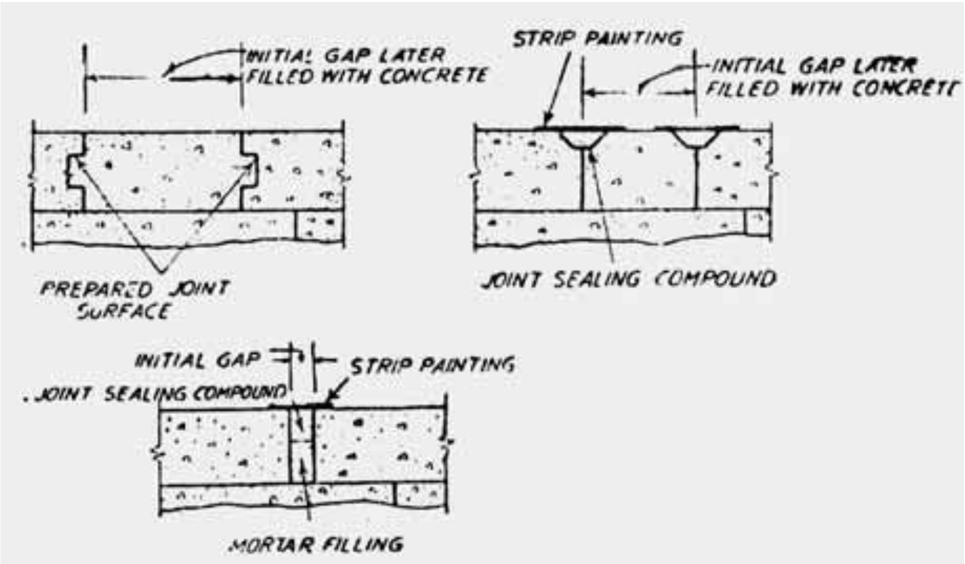


Fig. 5 Temporary open joints

after a suitable interval and before the structure is put to use, is filled with mortar or concrete completely as in Fig. 5(a) or as shown in Fig. 5 (b) and (c) with suitable jointing materials. In the first case width of the gap should be sufficient to allow the sides to be prepared before filling.

Spacing of Joints. Unless alternative effective means are taken to avoid cracks by allowing for the additional stresses that may be induced by temperature or shrinkage changes or by unequal settlement, movement joints should be provided at the following spacings:-

(a) In reinforced concrete floors, movement joints should be spaced at not more than 7.5 m apart in two directions at right angles. The wall and floor joints should be in line except where sliding joints occur at the base of the wall in which correspondence is not so important.

(b) For floors with only nominal percentage of reinforcement (smaller than the minimum specified) the concrete floor should be cast in panels with sides not more than 4.5 m.

(c) In concrete walls, the movement joints should normally be placed at a maximum spacing of 7.5 m. in reinforced walls and 6m. in unreinforced walls. The maximum length desirable between vertical movement joints will depend upon the tensile strength of the walls, and may be increased by suitable reinforcement. When a sliding layer is placed at the foundation of a wall, the length of the wall that can be kept free of cracks depends on the capacity of wall section to resist the friction induced at the plane of sliding. Approximately the wall has to stand the effect of a force at the place of sliding equal to weight of half the length of wall multiplied by the co-efficient of friction.

(d) Amongst the movement joints in floors and walls as mentioned above expansion joints should normally be provided at a spacing of not more than 30 m. between successive expansion joints or between the end of the structure and the next expansion joint; all other joints being of the construction type.

(e) When, however, the temperature changes to be accommodated are abnormal or occur more frequently than usual as in the case of storage of warm liquids or in uninsulated roof slabs, a smaller spacing than 30 m should be adopted, that is greater proportion of movement joints should be of the expansion type). When the range of temperature is small, for example, in certain covered structures, or where restraint is small, for example, in certain elevated structures none of the movement joints provided in small structures up to 45m. length need be of the expansion type. Where sliding joints are provided between the walls and either the floor or roof, the provision of movement joints in each element can be considered independently.

4. General Design for Requirements (I.S.I)
5.

1. Plain Concrete Structures. Plain concrete member of reinforced concrete liquid retaining structures may be designed against structural failure by allowing tension in plain concrete as per the permissible limits for tension in bending. This will automatically take care of failure due to cracking. However, nominal reinforcement shall be provided, for plain concrete structural members.

2. Permissible Stresses in Concrete

(a) For resistance to cracking. For calculations relating to the resistance of members to cracking, the permissible stresses in tension (direct and due to bending) and shear shall confirm to the values specified in Table 1. The permissible tensile stresses due to bending apply to the face of the member in contact with the liquid. In members less than 225 mm. thick and in contact with liquid on one side these permissible stresses in bending apply also to the face remote from the liquid.

(b) For strength calculations. In strength calculations the permissible concrete stresses shall be in accordance with Table 1. Where the calculated shear stress in concrete alone exceeds the permissible value, reinforcement acting in conjunction with diagonal compression in the concrete shall be provided to take the whole of the shear.

Table 1
Permissible concrete stresses in calculations relating to resistance to cracking

Grade of concrete	Permissible stress in kg/cm^2 Tension		Shear (= Q/bjd)
	Direct	Due to Bending	
M 150	11	15	15
M 200	12	17	17
M 250	13	18	19
M 300	15	20	22
M 350	16	22	25
M 400	17	24	27

3. Permissible Stresses in Steel

(a) For resistance to cracking. When steel and concrete are assumed to act together for checking the tensile stress in concrete for avoidance of crack, the tensile stress in steel will be limited by the requirement that the permissible tensile stress in the concrete is not exceeded so the tensile stress in steel shall be equal to the product of modular ratio of steel and concrete, and the corresponding allowable tensile stress in concrete.

(b) For strength calculations. In strength calculations the permissible stress shall be as follows:

(i) Tensile stress in member in direct tension — 1000 kg/cm^2

(ii) Tensile stress in member in bending on liquid retaining face of members or face away from liquid for members less than 225 mm thick. — 1000 kg/cm^2
On face away from liquid for members 225 mm. or more in thickness. — 1250 kg/cm^2

(iii) Tensile stress in shear reinforcement, For members less than 225 mm thickness — 1000 kg/cm^2
For members 225 mm or more in thickness — 1250 kg/m^2

(iv) Compressive stress in columns subjected to direct load. — 1250 kg/cm^2

Note 1. Stress limitations for liquid retaining faces shall also apply to:

(a) Other faces within 225 mm of the liquid retaining face.
(b) Outside or external faces of structures away from the liquid but placed in water logged soils upto the level of highest subsoil water level.

Note 2. The permissible stress of 1000 kg/cm^2 in (i), (ii) and (iii) may be increased to 1125 kg/cm^2 in case of deformed bars and in case of plain mild steel bars when the cross reinforcement is spot welded to the main reinforcement.

4. Stresses due to drying Shrinkage or Temperature Change.

(i) Stresses due to drying shrinkage or temperature change may be ignored provided that –

(a) The permissible stresses specified above in (ii) and (iii) are not otherwise exceeded.

(b) Adequate precautions are taken to avoid cracking of concrete during the construction period and until the reservoir is put into use.

(c) recommendation regarding joints given in article 8.3 and for suitable sliding layer beneath the reservoir are complied with, or the reservoir is to be used only for the storage of water or aqueous liquids at or near ambient temperature and the circumstances are such that the concrete will never dry out.

(ii) Shrinkage stresses may however be required to be calculated in special cases, when a shrinkage co-efficient of 300 x 10^{-6} may be assumed.

(iii) When the shrinkage stresses are allowed, the permissible stresses, tensile stresses to concrete (direct and bending) as given in Table 1. may be increased by 33 per cent.

5. Floors

(i) Provision of movement joints. Movement joints should be provided as discussed in article 3.

(ii) Floors of tanks resting on ground. If the tank is resting directly over ground, floor may be constructed of concrete with nominal percentage of reinforcement provided that it is certain that the ground will carry the load without appreciable subsidence in any part and that the concrete floor is cast in panels with sides not more than 4.5 m. with contraction or expansion joints between. In such cases a screed or concrete layer less than 75 mm thick shall first be placed on the ground and covered with a sliding layer of bitumen paper or other suitable material to destroy the bond between the screed and floor concrete.

In normal circumstances the screed layer shall be of grade not weaker than M 100, where injurious soils or aggressive water are expected, the screed layer shall be of grade not weaker than M 150 and if necessary a sulphate resisting or other special cement should be used.

(iii) Floor of tanks resting on supports

(a) If the tank is supported on walls or other similar supports the floor slab shall be designed as floor in buildings for bending moments due to water load and self weight.

(b) When the floor is rigidly connected to the walls (as is generally the case) the bending moments at the junction between the walls and floors shall be taken into account in the design of floor together with any direct forces transferred to the floor from the walls or from the floor to the wall due to suspension of the floor from the wall.

If the walls are non-monolithic with the floor slab, such as in cases, where movement joints have been provided between the floor slabs and walls, the floor shall be designed (only for the vertical loads on the floor.

(c) In continuous T-beams and L-beams with ribs on the side remote from the liquid, the tension in concrete on the liquid side at the face of the supports shall not exceed the permissible stresses for controlling cracks in concrete. The width of the slab shall be determined in usual manner for calculation of the resistance to cracking of T-beam, L-beam sections at supports.

(d) The floor slab may be suitably tied to the walls by rods properly embedded in both the slab and the walls. In such cases no separate beam (curved or straight) is necessary under the wall, provided the wall of the tank itself is designed to act as a beam over the supports under it.

(e) Sometimes it may be economical to provide the floors of circular tanks, in the shape of dome. In such cases the dome shall be designed for the vertical loads of the liquid over it and the ratio of its rise to its diameter shall be so adjusted that the stresses in the dome are, as far as possible, wholly compressive. The dome shall be supported at its bottom on the ring beam which shall be designed for resultant circumferential tension in addition to vertical loads.

6. Walls

(i) Provision of Joints

(a) Sliding joints at the base of the wall. Where it is desired to allow the walls to expand or contract separately from the floor, or to prevent moments at the base of the wall owing to fixity to the floor, sliding joints may be employed.

(b) The spacing of vertical movement joints should be as discussed in article 8.3 while the majority of these joints may be of the partial or complete contraction type, sufficient joints of the expansion type should be provided to satisfy the requirements given in article

(ii) Pressure on Walls.

(a) In liquid retaining structures with fixed or floating covers the gas pressure developed above liquid surface shall be added to the liquid pressure.

(b) When the wall of liquid retaining structure is built in ground, or has earth embanked against it, the effect of earth pressure shall be taken into account.

(iii) Walls or Tanks Rectangular or Polygonal in Plan.

While designing the walls of rectangular or polygonal concrete tanks, the following points should be borne in mind.

(a) In plane walls, the liquid pressure is resisted by both vertical and horizontal bending moments. An estimate should be made of the proportion of the pressure resisted by bending moments in the vertical and horizontal planes. The direct horizontal tension caused by the direct pull due to water pressure on the end walls, should be added to that resulting from horizontal bending moments. On liquid retaining faces, the tensile stresses due to the combination of direct horizontal tension and bending action shall satisfy the following condition:

$$\frac{t'}{t} + \frac{\sigma'_{ct}}{\sigma_{ct}} \leq 1$$

where,

t' = calculated direct tensile stress in concrete.

t = permissible direct tensile stress in concrete (Table 1)

σ'_{ct} = calculated tensile stress due to bending in concrete.

σ_{ct} = permissible tensile stress due to bending in concrete.

(d) At the vertical edges where the walls of a reservoir are rigidly joined, horizontal reinforcement and haunch bars should be provided to resist the horizontal bending moments even if the walls are designed to withstand the whole load as vertical beams or cantilever without lateral supports.

(c) In the case of rectangular or polygonal tanks, the side walls act as two-way slabs, whereby the wall is continued or restrained in the horizontal direction, fixed or hinged at the bottom and hinged or free at the top. The walls thus act as thin plates subjected triangular loading and with boundary conditions varying between full restraint and free edge. The analysis of moment and forces may be made on the basis of any recognized method.

(ii) **Walls of Cylindrical Tanks**. While designing walls of cylindrical tanks the following points should be borne in mind:

(a) Walls of cylindrical tanks are either cast monolithically with the base or are set in grooves and key ways (movement joints). In either case deformation of wall under influence of liquid pressure is restricted at and above the base. Consequently, only part of the triangular hydrostatic load will be carried by ring tension and part of the load at bottom will be supported by cantilever action.

(b) It is difficult to restrict rotation or settlement of the base slab and it is advisable to provide vertical reinforcement as if the walls were fully fixed at the base, in addition to the reinforcement required to resist horizontal ring tension for hinged at base, conditions of walls, unless the appropriate amount of fixity at the base is established by analysis with due consideration to the dimensions of the base slab the type of joint between the wall and slab, and, where applicable, the type of soil supporting the base slab.

7. Roofs

(i) Provision of Movement Joints. To avoid the possibility of sympathetic cracking it is important to ensure that movement joints in the roof correspond with those in the walls, if roof and walls are monolithic. It, however, provision is made by means of a sliding joint for movement between the roof and the wall correspondence of joints is not so important.

(ii) Loading. Field covers of liquid retaining structures should be designed for gravity loads, such as the weight of roof slab, earth cover if any, live loads and mechanical equipment. They should also be designed for upward load if the liquid retaining structure is subjected to internal gas pressure.

A superficial load sufficient to ensure safety with the unequal intensity of loading which occurs during the placing of the earth cover should be allowed for in designing roofs. The engineer should specify a loading under these temporary conditions which should not be exceeded. In designing the roof, allowance should be made for the temporary condition of some spans loaded and other spans unloaded, even though in the final state the load may be small and evenly distributed.

(iii) Water tightness. In case of tanks intended for the storage of water for domestic purpose, the roof must be made water-tight. This may be achieved by limiting the stresses as for the rest of the tank, or by the use of the covering of the waterproof membrane or by providing slopes to ensure adequate drainage.

(iv) Protection against corrosion. Protection measure shall be provided to the underside of the roof to prevent it from corrosion due to condensation.

8. Minimum Reinforcement

(a) The minimum reinforcement in walls, floors and roofs in each of two directions at right angles shall have an area of 0.3 per cent of the concrete section in that direction for sections upto 100 mm, thickness. For sections of thickness greater than 100 mm, and less than 450 mm the minimum reinforcement in each of the two directions shall be linearly reduced from 0.3 percent for 100 mm. thick section to 0.2 percent for 450 mm, thick sections. For sections of thickness greater than 450 mm, minimum reinforcement in each of the two directions shall be kept at 0.2 per cent. In concrete sections of thickness 225 mm or greater, two layers of reinforcement steel shall be placed one near each face of the section to make up the minimum reinforcement.

(b) In special circumstances floor slabs may be constructed with percentage of reinforcement less than specified above. In no case the percentage of reinforcement in any member be less than 0°15% of gross sectional area of the member.

9. Minimum Cover to Reinforcement.

(a) For liquid faces of parts of members either in contact with the liquid (such as inner faces or roof slab) the minimum cover to all reinforcement should be 25 mm or the diameter of the main bar whichever is grater. In the presence of the sea water and soils and water of corrosive characters the cover should be increased by 12 mm but this additional cover shall not be taken into account for design calculations.

(b) For faces away from liquid and for parts of the structure neither in contact with the liquid on any face, nor enclosing the space above the liquid, the cover shall be as for ordinary concrete member.

5. Tanks Resting on Ground. For small capacities rectangular tanks are generally used and for bigger capacities circular tanks are used. The walls of circular tanks may have flexible joints or rigid joints at the base.

6. Ciruclar Tanks with Flexible Joint at the Base. In these tanks walls are subjected to hydrostatic pressure. The tank wall is designed as thin cylinder.

At the base, minimum pressure

$= \boldsymbol{wH}$.

This causes hoop tension

$$= \frac{wHD}{2}$$

where

w is the density of water.

H is depth of water, and

D is diameter of the tank

Steel area required at the base for one metre height

$$= \frac{wHD}{2 \times 1000} cm^2$$

If '*t*' is the thickness of wall, tensile stress in concrete

$$= \frac{wHD/2}{100t + (m-1)A_t} kg/cm^2$$

As the hoop tension reduces gradually to zero at top, the reinforcement is gradually reduced to minimum reinforcement at top. The main reinforcement consists of circular hoops Vertical reinforcement equal to 0.3% of concrete area is provided and hoop reinforcement is tied to this reinforcement. In smaller tanks main reinforcement is placed near the outer face. For bigger tanks the wall thickness is more and the reinforcement is placed on both faces.

Though assumed that base is flexible, but in reality there will always be some restraint at the base and some pressure will be resisted by cantilever action of the wall. The minimum vertical reinforcement provided will be adequate to resist bending stresses caused by cantilever action.

Example 1. Design a circular tank with flexible base for capacity of 500,000 litres.

Sol. Depth of 4 m, is provided with free board of 20 cm. If *D* is the diameter or the tank capacity of the tank will be

$$\frac{\pi}{4} \times D^2 \times 3.8 = \frac{500,000 \times 10^3}{10^6}$$

$$D = \sqrt{\frac{500 \times 4}{3.8\pi}} = 12.94 m.$$

Provide diameter of 13 m.

Maximum hoop tension at base for one metre height

$$= \frac{wHD}{2} = \frac{1000 \times 4 \times 13}{2} = 26,000 kg$$

Area of steel required

$$= \frac{26,000}{1000} = 26 cm^2$$

Provide 16 mm ϕ bars at 7 cm centres.

$$A_t = 28.73 \text{ cm}^2$$

Use mix *M* 200. Allowable stress in tension

$$= 12 \text{ kg/cm}^2.$$

Let '*t*' be thickness of wall.

Tensile stress = $\dfrac{26{,}000}{100t+(13-1)\times 28.73} = 12$

$$t = \frac{26{,}000}{12\times 100} - \frac{12\times 28.73}{100}$$

$$= 21.67 - 3.448 = 25.118 \text{ cm}$$

Provide thickness of 26 cm.

Hoop reinforcement will be provided on both faces. 16 mm ϕ bars at 14 cm centres are provided on each face.

Vertical reinforcement

$$= \left(0.3 - 0.1\times\frac{(26-10)}{35}\right)\%$$

$$= 0.26\%$$

Bending moment

$$M = \frac{EId^2y}{dx^2}, \text{shear force F} = EI\frac{d^3y}{dx^3}$$

Loading intensity

$$P_c = EI\frac{d^4y}{dx^4}$$

If the effect of the lateral restraint is taken into account modified flexural rigidity will be

$$= \frac{Et^3}{12(1-\mu^2)} \qquad (\mu = \text{Poisson's ratio})$$

$$\therefore \frac{Et^3}{12(1-\mu^2)}\frac{d^4y}{dx^4} = w(H-x) - \frac{4tEy}{D^z}$$

$$\therefore \frac{d^4y}{dx^4} + \frac{48(1-u^2)y}{t^2D^2} = \frac{12w(1-u^2)(H-x)}{Et^3}$$

Putting $\alpha = 4\sqrt{\dfrac{1(1-u^2)}{t^2D^2}}$

$$\frac{d^4y}{dx^4} - 4x^4y = \frac{12w(H-x)(1-\mu^2)}{Et^3}$$

The solution of this differential equation is

$$y - e^{\alpha z}(C_1\cos\alpha x + C_2 \text{ Sin }\alpha x) + e^{-\alpha z}(C_3\cos\alpha x + C_4\sin\alpha x) + \frac{w(H-x)D^2}{4Et}$$

The values of constants C_1, C_2, C_3 and C_4 will depend on the restraint provided at top and bottom.

The value of μ may be taken as 0.2.

1. Circular Tanks Fixed at Base and Free at top. At top the shear force and *B.M* will be zero. At the base slope and deflection will be zero. Applying these four conditions, four equations will be obtained which can be solved and constants C_1, C_2, C_3 and C_4 evaluated.

$$At\ x = H,\quad M = 0 \quad \therefore EI\frac{d^2y}{dx^2} = 0$$

$$At\ x = H,\quad F = 0 \quad \therefore EI\frac{d^3y}{dx^3} = 0$$

$$At\ x = 0,\quad \theta = 0 \quad \therefore EI\frac{dy}{dx} = 0$$

$$At\ x = 0,\quad y = 0 \quad \therefore EIy = 0$$

Knowing four constant solution of elastic curve is known. Hence values of P_c and P_r can be found at different heights. Table gives coefficients for ring tension and *B.M* at various heights and shear at base.

CYLNDRICAL TANKS WITH FIXED BASE, FREE TOP

Coefficients for Tension in Circular Rings
Triangular Load
T = Coefficient x *wHR* kg. per m.
Positive sign indicates tension.

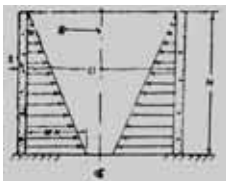


Table 2.A **Cylindrical Tanks with Fixed Base and free top**

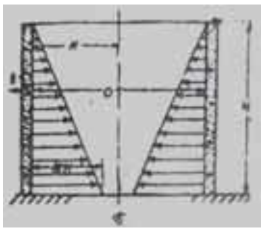
H^2	Coefficient at Point									
D_t	0.0H	0.1H	0.2H	0.3H	0.4H	0.5H	0.6H	0.7H	0.8H	0.9H
0.4	+0.149	+0.134	+0.120	+0.101	+0.062	+0.066	+0.049	+0.029	+0.014	+0.004
0.2	+0.263	+0.239	+0.215	+0.190	+0.160	+0.130	+0.096	+0.063	+0.034	+0.010
1.2	+0.283	+0.271	+0.254	+0.234	+0.209	+0.180	+0.142	+0.099	+0.054	+0.016
1.6	0.265	+0.268	+0.268	+0.266	+0.250	+0.226	+0.185	+0.134	+0.075	+0.023
2.0	+0.234	+0.251	+0.273	+0.285	+0.285	+0.274	+0.232	+0.172	+0.104	+0.031
3.0	+0.134	+0.203	+0.267	+0.322	+0.357	+0.362	+30.30	+0.262	+0.157	+0.052
4.0	+0.067	+0.164	+0.256	+0.389	+0.403	+0.429	+0.409	+0.334	+0.210	+0.073
5.0	+0.025	+0.137	+0.245	+0.346	+0.428	+0.477	+0.469	+0.398	+0.259	+0.092
6.0	+0.018	+0.119	+0.234	+0.344	+0.441	+0.505	+0.514	+0.447	+0.301	+0.112
8.0	-0.011	+0.104	+0.218	+0.335	+0.443	+0.534	+0.575	+530	+0.381	+0.151
10.0	-0.011	+0.098	+0.028	+0.323	+0.437	+0.542	+0.608	+0.589	+0.440	+0.179
12.0	-0.005	+0.097	+0.202	+0.312	+0.429	+0.543	+0.628	+0.633	+0.494	+0.211
14.0	-0.002	+0.098	+0.200	+0.306	+0.420	+0.539	+0.639	+0.666	+0.541	+0.241
16.0	0.000	+0.099	+0.199	+0.300	+0.413	+0.531	+0.541	+0.687	+0.582	+0.265

Coefficient of Point

	0.75H	0.80H	0.85H	0.90H	0.5H
20	+0.716	+0.654	+0.520	+0.325	+0.115
24	+0.746	+0.702	+0.577	+0.372	+0.137
32	+0.782	+0.768	+0.663	+0.459	+0.182
40	+0.800	+0.805	+0.731	+0.530	+0.217
48	+0.701	+0.828	+0.785	+0.593	+0.254
56	+0.763	+0.838	+0.824	+0.536	+0.285

Table 2.B
CYLINDRICAL TANKS

Coefficients for Moments in Cylindrical Walls.
Triangular Load
Moments = Coefficient x wH^3 kg .m per m.
Positive sign indicates tension in the outside



H^2	Coefficient at Point									
D_t	0.1H	0.2H	0.3H	0.4H	0.5H	0.6H	0.7H	0.8H	0.9H	1.0H
0.4	+.0005	+.0014	+.0021	.0007	-.0063	-.0150	-.0302	-.0529	-.0616	-.1205
0.2	+.0011	+.0037	+.0065	+.0060	+.0070	+.0023	-.0065	-.0234	-.0445	-.0795
1.2	+.0013	+.0043	+.0077	+.0103	.0113	+.0090	+.0022	-.0108	-.0311	-.0005
1.6	+.0011	+.0041	+.0075	+.0107	+.0131	+.0111	+.0058	-.0051	-.0222	-.0505
2.0	+.0010	+.0035	+.0065	+0089	+.0120	+.0115	+.0075	-.0021	-.0135	-.0436
3.0	+.0006	+.0024	+.0047	+.0071	+.0090	+.0097	+.0077	+.0012	-.0119	-.0333
4.0	+.0002	+.0015	+.0028	+.0067	+.0065	+.0077	+.0069	+.0025	-.0080	-.0266
5.0	+.0002	+.0006	+.0016	+.0029	+.0046	+.0059	+.0059	+.0028	-.0058	-.0222
6.0	+.0001	+.0008	+.0008	+.0019	+.0032	+.0046	+.0051	+.0029	-.0041	-.0187
8.0	.0000	+.0001	+.0008	+.0008	+.0016	+.0028	+.0038	+.0029	-.0022	-.0146
10.0	.0000	.0000	+.0001	+.0004	+.0007	+.0019	+.0029	+.0025	-.0002	-.0122
12.0	.0000	-.0001	+.0001	+.0002	+.0008	+.0013	+.0023	+.0026	-.0006	-.0104
14.0	.0000	.0000	.0000	.0000	+.0001	+.0009	+.0019	+.0023	-.0001	-.0090
16.0	.0000	.0000	-.0001	-.0001	-.0001	+.0004	+.0013	+.0019	-.0001	-.0079

Coefficient of Point

	0.80H	0.85H	0.90H	0.25H	1.00H
20	+.0016	+.0014	+.0005	-.0018	-.0003
24	+.0012	+.0012	+.0007	-.0013	-.0053
32	+.0007	+.0009	+.0007	-.0008	-.0040
40	+.0002	+.0005	+.0006	-.0005	-.0032
48	+.0000	+.0001	+.0006	-.0003	-.0026
56	+.0000	+.0000	+.0006	-.0001	-.0023

2. Circular tanks hinged at base and free at top.

At the top shear force and bending moment will be zero. At the base deflection and *B.M.* will be zero.

At $x = H$, $\quad -M = EI \dfrac{d^2 y}{dx^2} = 0$

At $x = H$, $\quad -F = EI \dfrac{d^3 y}{dx^3} = 0$

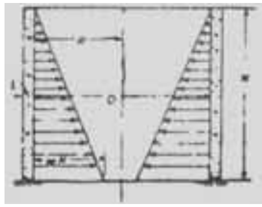
At $x = 0$, $\quad EIy = 0$

At $x = 0$, $\quad -M = EI \dfrac{d^2 v}{dx^2} = 0$

Thus four equations are obtained. These four equations can be solved for four constant. Hence values of *Pc* and *Pr* can be found.

Table 3C gives coefficients for ring tension and *B.M* at various heights and shear at the base.

CYLINDRICAL TANKS WITH HINGED BASE AND FREE TOP



Coefficients for Tension in Circular Rings
Triangular Load
T = Coefficient x wHR kg. per m.
Positive sign indicates tension.

Table 3A - Cylindrical tanks with hinged base and free top

H^2	Coefficient at Point									
D_t	0.0H	0.1H	0.2H	0.3H	0.4H	0.5H	0.6H	0.7H	0.8H	0.9H
0.4	+0.474	+0.440	+0.395	+0.352	+0.308	+0.264	+0.215	+0.166	+0.111	+0.007
0.2	+0.423	+0.402	+0.381	+0.358	+0.330	+0.297	+0.249	+0.202	+0.145	+0.076
1.2	+0.350	+0.355	+0.361	+0.362	+0.358	+0.343	+0.309	+0.255	+0.185	+0.098
1.6	+0.271	+0.302	+0.341	+0.369	+0.385	+0.385	+0.362	+0.314	+0.233	+0.124
2.0	+0.205	+0.260	+0.321	+0.373	+0.411	+0.434	+0.419	+0.369	+0.280	+0.151
3.0	+0.074	+0.179	+0.281	+0.375	+0.449	+0.506	+0.519	+0.479	+0.875	+0.210
4.0	-0.017	+0.137	+0.253	+0.267	+0.469	+0.545	+0.579	+0.553	+0.447	+0.256
5.0	-0.006	+0.114	+0.235	+0.356	+0.469	+0.562	+0.617	+0.606	+0.503	+0.294
6.0	-0.011	+0.103	+0.223	+0.343	+0.463	+0.566	+0.639	+0.643	+0.547	+0.327
8.0	-0.015	+0.096	+0.208	+0.324	+0.443	+0.564	+0.661	+0.697	+0.621	+0.386
10.0	-0.006	+0.095	+0.200	+0.311	+0.423	+0.552	+0.666	+0.730	+0.676	+0.433
12.0	-0.002	+0.097	+0.197	+0.302	+0.417	+0.541	+0.664	+0.750	+0.720	+0.477
14.0	0.000	+0.000	+0.197	+0.299	+0.408	+0.531	+0.659	+0.761	+0.752	+0.513
16.0	+0.202	+0.100	+0.198	+0.299	+0.403	+0.521	+0.650	+0.764	+0.776	+0.543

Coefficient of Point

	0.75H	0.80H	0.85H	0.90H	0.95H
20	+0.812	+0.817	+0.756	+0.603	+0.0344
24	+0.816	+0.839	+0.793	+0.647	+0.0377
32	+0.814	+0.861	+0.847	+0.721	+0.436
40	+0.802	+0.866	+0.880	+0.778	+0.483
48	+0.791	+0.864	+0.900	+0.820	+0.527
56	+0.781	+0.859	+0.911	+0.852	+0.563

3. Cylindrical tank with top slab and fixed base

At top deflection will be zero and slope will be as for the slope of top slab. These conditions will give two equations. At bottom, slope and deflection will be zero. Thus four equations are obtained which can be solved for constants C_1, C_2, C_3 and C_4. Hence values of P_0 and P_r can be calculated. *B. M.* and ring tension at various heights can be calculated.

4. Cylindrical tanks with Domes at top and bottom.

The slopes and deflections of the wall at top and bottom will be equal to slopes and deflections for the domes at top and bottom respectively. Thus four equations can be formed which can be solved for four constants. Knowing equation of elastic curve, values of P_0 and P_r can be calculated.

Ex. 2. Design the tank of Problem 17° 1 if the tank is fixed at the base and free at the top.

Sol. $D = 13$ m., $H = 4$ m.

Assume thickness of wall = 15 cm.

$$\frac{H^2}{Dt} = \frac{4^2}{13 \times 0.15} = 8.20$$

Coefficients for hoop tension and *B.M* for various heights are found by interpolation from Table 17.2. These coefficients are given in Table 17.4.

Maximum hoop tension occurs at 0.6 *H* from top

Maximum hoop tension

$$= 0.5783 \text{ x } w\frac{HD}{2}$$

$$= 0.5783 \text{ x } 1000 \text{ x } \frac{4 \times 13}{2}$$

$$= 14{,}736 \text{ kg}$$

TABLE 4

Depth	Coefficient for hoop tension	Depth	Coefficient for *B.M.*
0°0*H*	- 0°011	0°1*H*	- 0°0000
0°1*H*	+ 0°1034	0°2*H*	+ 0°00009
0°2*H*	+ 0°2170	0°3*H*	+ 0°00019
0°3*H*	+ 0°3338	0°4*H*	+ 0°00076
0°4*H*	+ 0°4424	0°5*H*	+ 0°00151
0°5*H*	+ 0°5348	0°6*H*	+ 0°00271
0°6*H*	+ 0°5783	0°7*H*	+ 0°00371
0°7*H*	+ 0°5359	0°8*H*	+ 0°00289
0°8*H*	+ 0°3869	0°9*H*	- 0°0021
0°9*H*	+ 0°1538	1*H*	- 0°01436
H	-		

Steel area required = $\frac{14,736}{1000}$ = 14.736 cm^2

Provide 12 mm ϕ bars at 14 cm. centers on both faces

A_1 = 16.16 cm^2

The reinforcement is provided for 0.8*H* to 0.3*H* from top. In the remaining portion provide 12 mm ϕ bars at 20 cm centers.

Tensile stress in concrete

$$= \frac{14,736}{100 \times 15 + (m - 11) \times 16.1}$$

$$= \frac{14,736}{1500 + 12 \times 16.16}$$

$$= \frac{14,736}{1693.92} = 8.7 kg / cm^2$$

Safe

Maximum position B.M. (tension outside) occurs at 0.7*H* from top. Maximum +ve *B.M.* per metre height

= coefficient x wH^3

= 0.0037 x 1000 x 4^3

= + 237.4 kg. m.

Maximum – ve *B.M.* (tension inside)

= 0.01436 x 1000 x 4^3

= -919.1 kg. m.

Effective depth of 15 – 4 = 11 cm. is provided

Area of steel required for +ve *B.M.* on outer face

$$= \frac{23,740}{0.84 \times 11 \times 1000} = 2.57 \text{ cm}^2$$

Minimum percentage of steel to be provided

$$= 0.3 - \frac{0.1 \times 5}{35}$$

= 0.3 – 0.014 = 0.286.

Minimum steel area required

$$= \frac{0.286}{100} \text{ x } 15 \text{ x } 100 = 4.290 \text{ cm}^2$$

Minimum reinforcement on one face = 2.145 cm^2

Provide 8 mm ϕ bars at 20cm. centers.

Area of steel required for – ve *B.M.* on inner face

$$= \frac{91,910}{0.84 \times 11 \times 1000} = 9.65 \text{ cm}^2$$

Provide 12 mm bars at 110 mm center to center

Shear force

Maximum shear at base of wall

$$= 0.1724\ wH^2$$

$$= 0.1724 \times 1000 \times 4 \times 4 = 2758.4 \text{ kg}$$

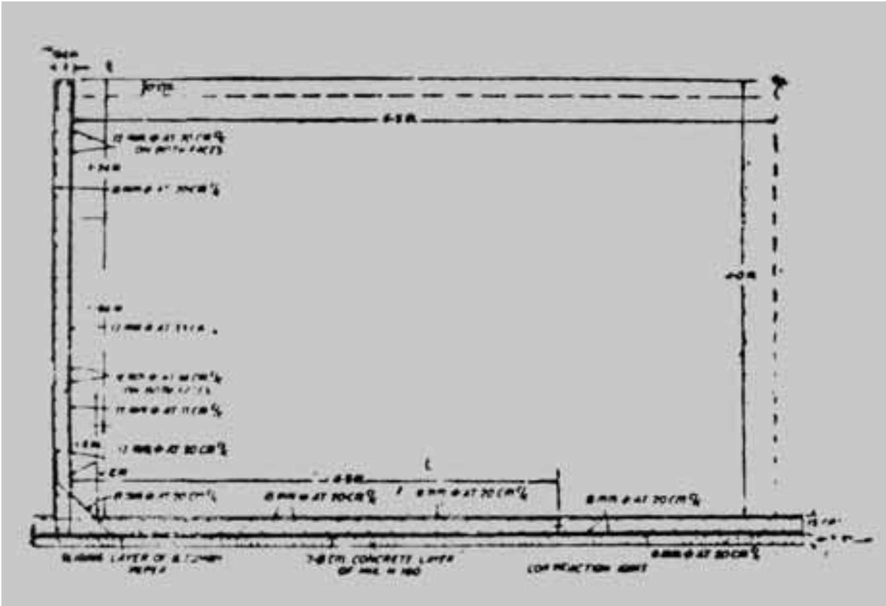


Fig. 7

Shear stress $= \dfrac{2758.4}{0.84 \times 11 \times 100} = 2.99 \text{ kg / cm}^2$

Safe.

Check for Bond

Bond stress $= \dfrac{2758.4}{0.84 \times 11 \times \dfrac{100}{11} \times 3.76}$

$= 8.73 \text{ kg/ cm}^2$

Safe.

Base. Provide 15 cm thick base slab with 8 mm ϕ bars at 20 cm centers both ways at top and bottom.

8. **Approximate method of design of circular tanks with fixed base.** In the approximate method of design of circular tanks it is assumed that some portion of the tank at base acts as cantilever and thus some load at bottom is taken by the cantilever effect. Load in the top portion is taken by the hoop tension caused in the top portion. The cantilever effect will depend on the dimensions of the tank and thickness of the wall for between 6 to 12, the cantilever portion may be assumed at H/3 or 1 m from base whichever is more. For H^2/D_t

between 12 to 30, the cantilever portion may be assumed as H/4 or 1 m from base, whichever is more.

In Fig. 8 *AB* is the height of tank and *ABC* pressure diagram. *ADB* is taken as pressure causing hoop tension and *DBC* is taken as cantilever load. The maximum *B.M.* occurs at the base.

The steel for hoop tension is provided on both faces. For the bottom portion BD reinforcement for hoop tension is provided in addition to steel required for bending.

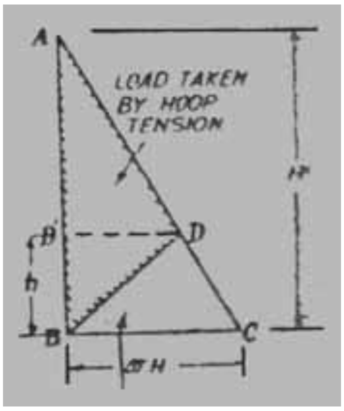


Fig.8

Ex. 3 Design the water tank of problem 1 by approximate method.

Sol. $D = 13$ m $\qquad H = 4$ m

Assume thickness of wall $t = 15$ cm

$$\frac{H^2}{Dt} = \frac{4\times 4}{13\times 0.15} = 8.276$$

It is assumed that bottom $\frac{H}{3}$ i.e. $\frac{4}{3}$ m acts as cantilever.

Maximum hoop tension = $\frac{pD}{2}$

$$= \frac{8000}{3}\times\frac{13}{2}$$

= 17,333 kg. per meter height.

Area of steel required

$$= \frac{17,333}{1000} = 17.33 \text{ cm}^2$$

Provide 12 mm ϕ bars at 13 cm centers on both faces

Maximum *B.M.* = $\frac{1}{2}\times 4000\times\frac{4}{3}\times\frac{1}{3}\times\frac{4}{3}$

$$= \frac{32,000}{27} = 1185\,kg.m$$

Effective depth = 15-4 = 11 cm

Area of steel required

$$= \frac{1185 \times 100}{0.84 \times 11 \times 1000} = 12.84 cm^3$$

Provide 12 mm ϕ bars at 8 cm center.

Distribution steel

$$\% \text{ reinforcement} = \frac{0.3 - 0.1 \times (15 - 10)}{(45 - 10)} = 0.286$$

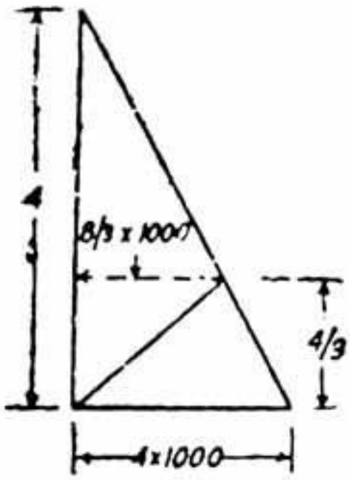


Fig. 9

$$\text{Steel area} = \frac{0.286}{100} \times 15 \times 100$$

$$= 4.29 \text{ cm}^2$$

Provide 8 mm ϕ bars at 22 cm. centers on each face.

9. **Rectangular tank**. Rectangular tanks are provided when small capacity tanks are required. For small capacities circular tanks prove uneconomical as the formwork for circular tanks is very costly. The rectangular tanks should be preferably square in plan from point of view of economy. It is desirable that longer side should not be greater than twice the smaller side.

In rectangular tanks moments are caused in two directions. The exact analysis is rather difficult and such tanks are designed by approximate methods.

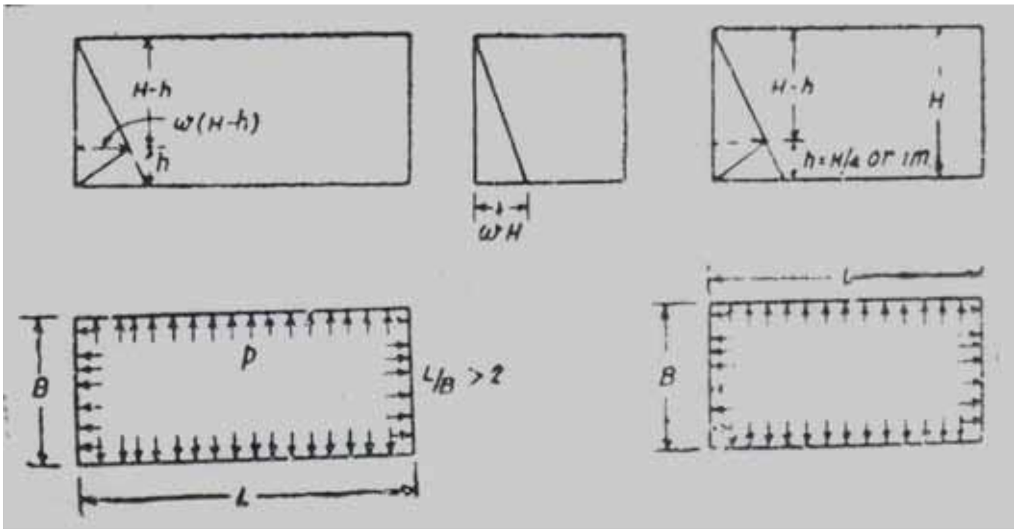
For rectangular tanks in which ratio of length to breadth is less than 2, tank walls are designed as continuous frame subjected to pressure varying from zero at to top to maximum at H/4 or 1 m., from base, whichever is more. The bottom portion H/4 (or) 1 m whichever is more is designed as cantilever. In addition to bending, walls are subjected to direct tension caused by the hydrostatic pressure on the walls. The section is to be designed for direct tension and bending. Bending moments in the walls are found by moment distribution. Direct tension in long walls

$$= \frac{w(H - h) \times B}{2}$$

and direct tension in short walls

$$= \frac{w(H-h)\times L}{2}$$

For rectangular tanks in which ratio of length to breadth is greater than 2, the long walls are designed as cantilevers and short



(b) Fig. 10 (a)

walls as slabs supported on long walls. Bottom portion of short walls H/4 or 1 m whichever is more, is designed as cantilever.

Maximum *B.M.* in long walls at base

$= \frac{1}{2}$ w H x H x $\frac{H}{3}$ = $\frac{wH^3}{6}$ In the short walls maximum *B.M.* occurs at support and is given by $\frac{w(H-h)B^2}{12}$ *B.M.* at center of short walls is taken as $\frac{w(H-h)B^2}{16}$. For bottom portion of short wall, which is designed as cantilever maximum *B.M.* is given by $\frac{wH^3}{6}$ or $\frac{wH\times 1}{6}$ whichever is greater. In addition to *B.M.* short walls and long walls are subjected to direct tension. Direct tension on long walls is given by $\frac{w(H-h)\times B}{2}$ .For short walls it is assumed that end one meter width of long wall contributes to direct tension on the short walls. Direct tension on short wall is *w(H-h)*.

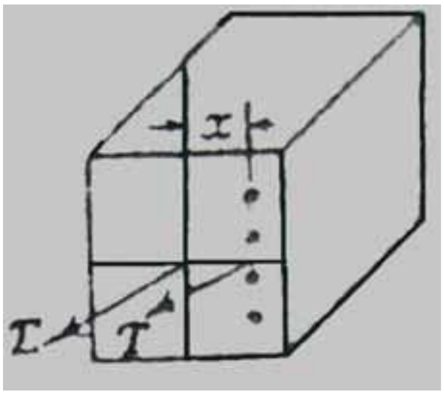


Fig. 10 (c)

Design of section for tension. It is assumed that entire tension is taken by steel. Let T be tension. Net $B.M. = M - T \times x$. Steel reinforcement is provided for $B.M.$ of $M - T \times x$ and direct tension T as shown in Fig. 10 (c).

Ex. 4 Design a rectangular tank for a capacity of 80,000 liters.

Sol. Provide height of 3.5 m for tank with free board of 15 cm.

Effective height = 335 cm.

Volume of tank to be required

= 80,000 liters = 80,000,000 c.c

Area of tank to be provided

$$= \frac{80.000,000}{335} = 238,900 \text{ cm.}^2$$

Provide length of 600 cm. and breadth of 400 cm.

$$\frac{L}{B} = \quad = 1.5 < 2$$

The wall of tanks are to be designed as continuous slab.

$$\frac{H}{4} = \frac{3.50}{4} = 0.875 \text{ m}$$

Bottom 1 m. of tank will be designed as cantilever.

Pressure at depth of 2.5 m.

$p = wh = 2.5 \times 1000$

$= 2,500 \text{ kg/m}^2$

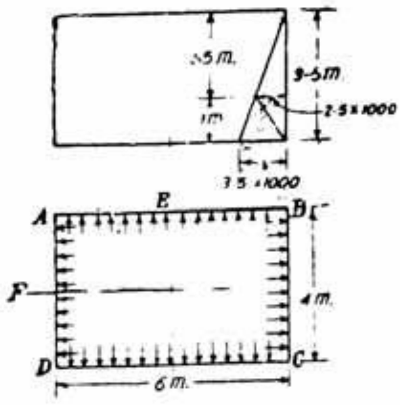


Fig. 11

Moments in the walls are found by moment distribution. As the frame is symmetrical about both axes moment distribution is done for one quarter of tank only.

Joint	A	
Member	AB	AD
Distribution Factors	0.4	0.6
Fixed Moment	-3p	+ 4/3 p
Balancing	+ 2/3 p	+ p
Final	- 7/3 P	+ 7/3p

Moment at support = $\frac{7}{3}p = \frac{7}{3}$ x 2500 = 5833 kg. m.

B.M. at center of long span

$$= \frac{2500 \times 6^2}{8} - 5833 = 11{,}250 - 5833$$

= 5417 kg. m.

B.M. at center of shorter span

$$= \frac{2500 \times 4^2}{8} - 5833$$

= 5000 – 5833 = - 833 kg. m.

Maximum *B.M.* = 5833 kg. m

Effective depth required

$$= \sqrt{\frac{5833 \times 100}{14.11 \times 100}} = 20.32 \text{ cm.}$$

Provide overall depth of 25 cm. with effective depth of 21.5 cm.

Direct tension in long wall

$$= \frac{2500 \times 4}{2} = 5000 \text{ kg.}$$

Direct tension in the short wall

$$= \frac{2500 \times 6}{2} = 7500 \text{ kg.}$$

Design of section

c = 70, m = 13, t = 1000 kg/cm^2 on water face

$$k = \frac{1}{1 + t / cm} = \frac{1}{1 + \frac{10.0}{70 \times 13}}$$

$$= \frac{1}{2.097} = 0.48$$

$$j = 1 - \frac{k}{3} = 1\text{-}0.16 = 0.84$$

$$Q = ½\ ckj$$

$$= ½ \text{ x } 70 \text{ x } 0.48 \text{ x } 0.84 = 14.11$$

Considering effect of bending only, effective depth required

$$= \sqrt{\frac{5833 \times 100}{14.11 \times 100}} = 20.32 \text{ cm.}$$

Provide overall depth of 25 cm. with effective depth 21.5 cm.

Net moment = $M - T$ x x

Area of steel

$$= \frac{M - T \times x}{0.84d \times 1000} + \frac{T}{1000}$$

$$= \frac{5833 \times 100 - 5000(21.5 - 12.5)}{0.84 \times 215 \times 1000} + \frac{5000}{1000}$$

$$= 29.77 + 50 = 34.77 \text{ cm}^2.$$

Provide 20 mm. ϕ bars at 8 cm. centers. Area of steel provided = 39.28 cm^2.

Steel at center of span

Tension occurs away from water face, c = 70, t = 1250, m = 13, n = 0.42d, j = 0.86d, Q = 12.64

Area of steel

$$= \frac{541{,}700 - 5000(21.5 - 12.5)}{0.86 \times 21.5 \times 1250} + \frac{5000}{1000}$$

$$= 21.49 + 5.0$$

$$= 26.49 \text{ cm}^2.$$

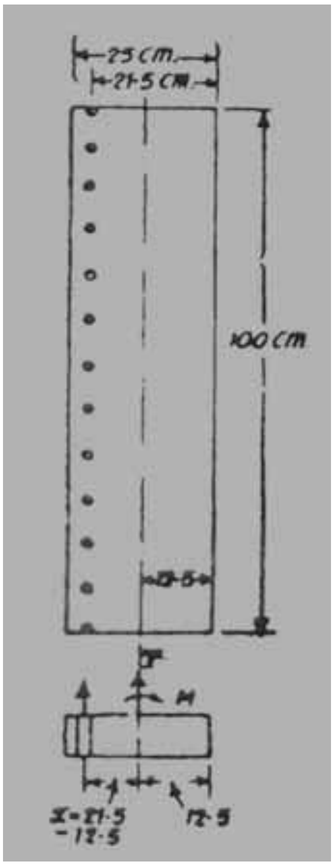


Fig. 12

Half the bars from inner face at support are bent in outer face providing area of $\frac{39.28}{2}$ = 19.64 cm^2.

Remaining area to be provided

$= 26.49 - 19.64 = 6.85$ cm^2.

Additional reinforcement of 16 mm. ϕ bars at 16 cm. centers is provided.

At the center of short span *B.M.* is of negative sign, however nominal reinforcement is provided on that face.

Cantilever Moment

Cantilever moment = 3.5 x 1000 x ½ x 1 x $\frac{1}{3}$ = 583.3 kg.m.

Area of steel required

$$= \frac{583.3 \times 100}{0.84 \times 21.5 \times 1000}$$

= 3.23 cm^2.

Provide 8 mm. ϕ bars at 15 cm. centers.

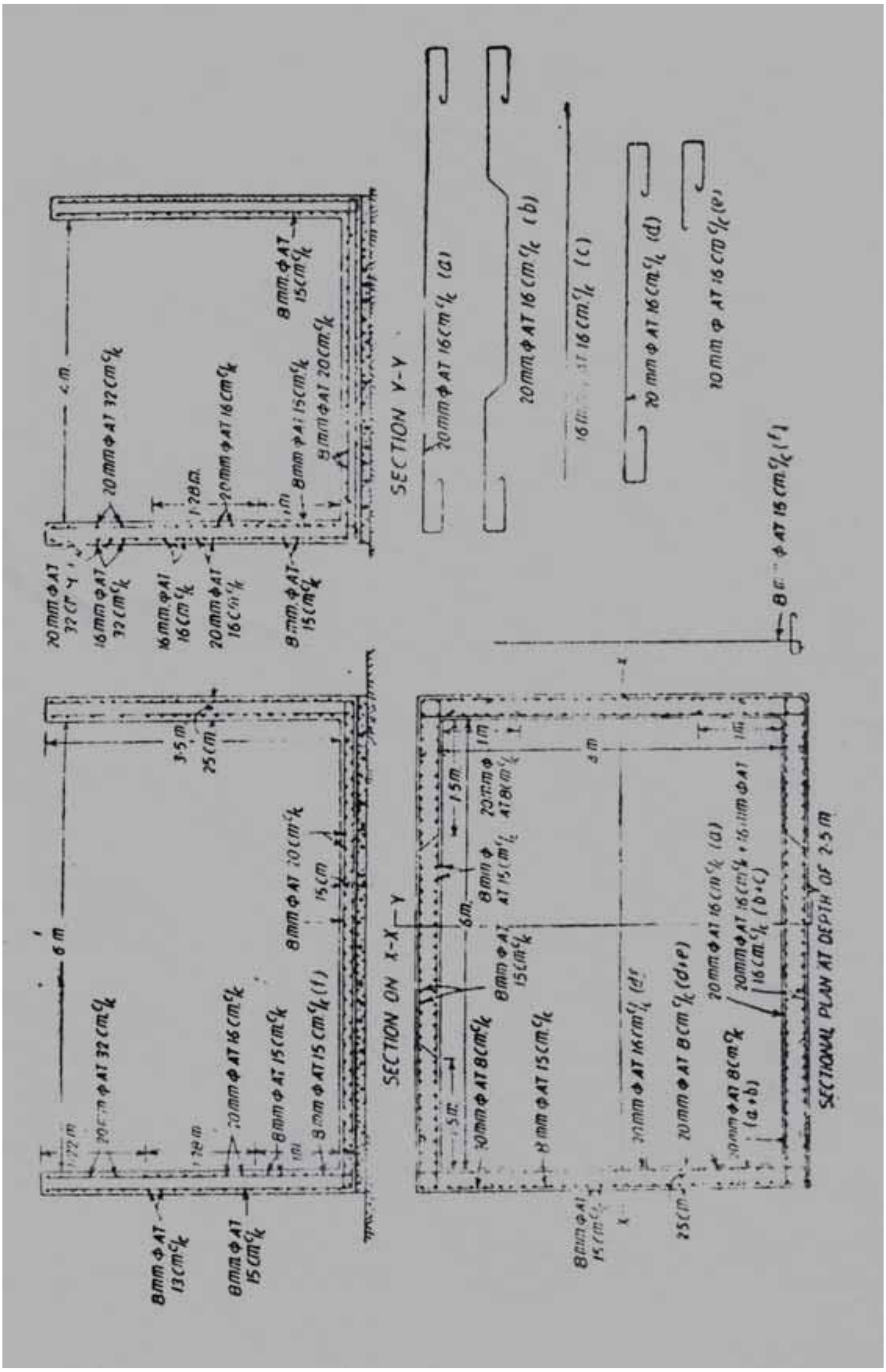


Fig. 13

Distribution steel

$$\text{Distribution steel} = 0.3 - \frac{0.1\times(25-10)}{(45-10)}$$

$$= 0.3 - 0.043$$

$$= 0.257\ \%$$

$$\text{Area of steel} = \frac{0.257}{100} \text{ x } 25 \text{ x } 100$$

$$= 6.425 \text{ cm}^2.$$

Area on each face = 3.212 cm^2.

Provide 8 mm ϕ bars at 15 cm. centers.

Base Slab

Provide 15 cm. thick slab with 8 mm ϕ bars at 20 cm. centers both ways at top and bottom.

10. Underground Tanks. The design principles of underground tanks are same as for tanks resting on the ground. The walls of the underground tanks are subjected to internal water pressure and outside earth pressure. The section of wall is designed for water pressure and earth pressure acting separately as well as acting simultaneously.

Whenever there is possibility of water table to rise, soil becomes saturated and earth pressure exerted by saturated soil should be taken into consideration.

Ex. 8.5. Design an underground reservoir 12 m. x 4 m. x 4 m. deep. The long walls will be designed as cantilevers and the top portion of the short walls will be designed as slab supported by long walls. Bottom one metre of short walls will be designed as cantilever slab.

Design of long wall

1. **Pressure of saturated soil acting from outside and no water pressure from inside.** Earth pressure at base will be due to water pressure plus due to submerged weight of soil.

$$p = 1000 \text{ x } 4 + (1600 - 1000) \text{ x } \tfrac{1}{2} \text{ x } 4$$

$$= 4000 + 800 = 4800 \text{ kg/m}^2.$$

Maximum *B.M.* at base of long wall

$$= 4800 \text{ x } \frac{4}{2}\times\frac{4}{3}$$

$$= 12{,}800 \text{ kg.m.}$$

$$= 1{,}280{,}000 \text{ kg. cm.}$$

Effective depth required

$$= \sqrt{\frac{12.80,000}{14.11 \times 100}}$$

$$= 30.11 \text{ cm.}$$

Provide overall depth of 35 cm. with effective depth of 31 cm.

$$\text{Area of steel} = \frac{1,210,000}{0.84 \times 1000 \times 31}$$

$$= 48.2 \text{ cm}^2.$$

Provide 20 mm ϕ bars at 6 cm. centers on outside face. Area of steel provided = 52.36 cm^2.

Reinforcement is curtailed in the same manner as in the second case.

Direct compression in long walls. Direct compression is caused in long walls because of earth pressure acting on short walls which act as slab supported on long walls.

Direct compression at 1 m above base

$$= 3600 \times \frac{4}{2}$$

$$= 7200 \text{ kg.}$$

This will be taken by wall and the distribution steel provided.

2. **Water pressure acting from inside and no earth pressure acting from outside.**

Maximum water pressure at base

$$= 1000 \times 4 = 4000 \text{ kg/m}^2$$

$$\text{Maximum } B.M. = 4000 \times \frac{4}{2} \times \frac{4}{3}$$

$$= \frac{32,000}{3} = 10,667 \text{ kg.m}$$

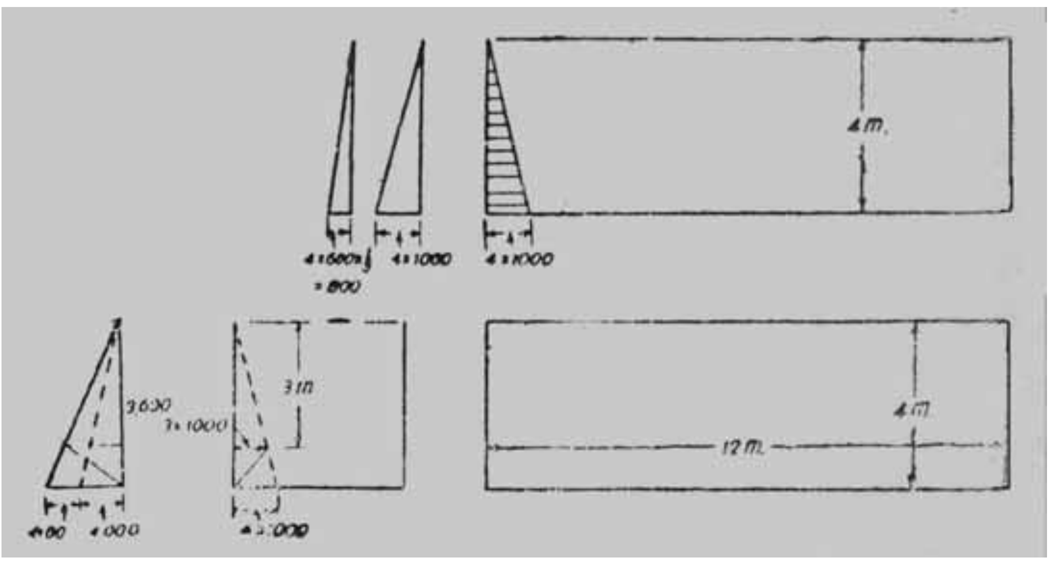


Fig. 14

Area of steel required

$$= \frac{10.667 \times 100}{0.84 \times 1000 \times 31}$$

$= 40.85 \text{ cm}^2$.

Provide 20 mm ϕ bars at 7 cm. centers on inside face. Area of steel provided.

$= 44.86 \text{ cm}^2$.

Curtailment of reinforcement. Let A_{th} be the reinforcement required at depth h. The *B.M.* at any depth is proportional to h^2.

$$\therefore \qquad \frac{A_{th}}{A_t} = \frac{h^3}{H^3}$$

$$h = \sqrt[3]{\frac{A_{th}}{A_t}} \times H$$

Depth where half the bars can be curtailed, $A_{th} = \frac{1}{2} A_t$.

$$\therefore \qquad h = \sqrt[3]{\tfrac{1}{2}} \times 4$$

$= 3.175$ m

Taking bond length as 20d, the bars are curtailed at 1.25 from base. 20 mm ϕ at 12 cm. *c/c* are provided on outside and 20 mm ϕ at 14 cm. *c/c* inside.

Depth where only ¼th reinforcement is required $A_{th} = \frac{1}{4} A_t$.

$$h = \sqrt[3]{\tfrac{1}{2}} \times 4$$

$= 2.52$ m

¼th reinforcement i.e. 20 mm ϕ at 24 cm. centers are provided on outside and 20 mm ϕ at 28 cm. *c/c* inside at 2 m. and above from base.

Distribution steel

% of distribution steel

$$= 0.3 - \frac{0.1 \times (35 - 10)}{(45 - 10)}$$

$= 0.3 - 0.06 = 0.24$

$$\text{Area of steel} = 0.24 \times \frac{35 \times 100}{100}$$

$= 7.68 \text{ cm}^2$.

Area to be provided on each face

$= 3.84 \text{ cm}^2$.

Provide 8 mm ϕ bars at 13 cm centers.

Direct tension in long walls. Direct tension is caused in long walls because of water pressure acting on short walls which act as slab supported on long walls.

Direct tension at 1 m above base

$$= 3 \times 1000 \times \frac{4}{2} = 6000 \text{ kg.}$$

Area of steel required

$$= \frac{6000}{1000} = 6 \text{ cm}^2.$$

Area of distribution steel provided is 7.6 cm^2.

Distribution steel will take direct tension.

Design of short wall

1. Water pressure acting from inside and no earth pressure acting from outside.

Bottom one metre acts as cantilever and remaining 3 m acts as slab supported on long walls.

Water pressure at depth of 3 m

$$= 1000 \times 3 = 3000 \text{ kg/m}^2.$$

Maximum *B.M.* is assumed as

$$= \frac{wl^2}{12} \text{ at supports.}$$

$$B.M. \text{ at support} = \frac{wl^2}{12} = \frac{3000 \times 4^2}{12}$$

$$= 4000 \text{ kg.m}$$

Direct tension from end one metre of long wall

$$= 3000 \times 1 = 3000 \text{ kg.}$$

$$\text{Net } B.M. = M - T \times x = 4000 - 3000 \times 0.12$$

$$= 3640 \text{ kg.m.}$$

$$\text{Area of steel} = \frac{3640 \times 100}{0.84 \times 31 \times 1000} + \frac{3000}{1000} \text{ (Tension inside)}$$

$$= 13.97 + 3$$

$$= 16.97 \text{ cm}^2.$$

Provide 12 mm ϕ bars at 6 cm centers in one metre length. As the bending moment is proportional to the depth of water, reinforcement will vary linearly with depth of water. At depth of 2 m., 12 mm bars at 12 cm. centers are provided. At depth of 1 m., 12 mm ϕ bars at 18 cm centers are provided.

B.M at center of span

$$= \frac{wl^2}{12} = \frac{3000 \times 4^2}{16}$$

= 3000 kg m.

Net $B.M. = M - T$ x x = 3000 – 3000 x 0.12

= 2640 kg. m.

$$\text{Area of steel} = \frac{2640 \times 100}{0.86 \times 31 \times 1000} + \frac{3000}{1000}$$

= 10.14 + 3 = 13.14 cm^2

Actually the section is doubly reinforced as steel is to be proded on front face for the bending moment produced due to outside pressure.

12 mm ϕ bars are provided at 6 cm centers.

A_t provided = 18.85 cm^2.

The reinforcement is reduced linearly towards top. At depth of metres 12 mm ϕ bars are provided at 12 cm. centers. At depth of 1 metre 12 mm ϕ bars are provided at 18 cm. centers.

Design of bottom one metre

$B.M.$= 1000 x 4 x ½ x ½

= 667 kg.m

$$\text{Area of steel} = \frac{667 \times 100}{0.84 \times 31 \times 1000}$$

= 2.56 cm^2

Minimum reinforcement of 8 mm ϕ at 13 cm centers is provided.

Distribution steel of 8 mm ϕ at 13 cm centers is provided on both faces.

2. Pressure of Saturated soil acting from outside and no water pressure from inside.

Direct compression due to cantilever action of one metre length of long wall = 3600 kg.

Pressure p = 3600 kg/ m^2

$$B.M. \text{ at support} = \frac{pl^2}{12} = \frac{3600 \times 4 \times 4}{12}$$

= 4800 kg. m

$$\text{B.M at center of span} = \frac{3600 \times 4 \times 4}{12} = 3600 \text{ kg.m}$$

Area of steel required, considering bending only,

$$A_t = \frac{4800 \times 100}{0.86 \times 1000 \times 31}$$

$= 18.43\ cm^2$ (Tension outside)

Provide 12 mm ϕ bars at 6 cm centers,

A_t provided = 18.85 cm^2

Actually the section is doubly reinforced and area of steel required will be less. The effect of direct compressive force is not considered as it is very small and compressive reinforcement is provided.

Area of steel at center of span

$$= \frac{3600 \times 100}{0.84 \times 1000 \times 31}$$

$= 13.81\ cm^2$.

Provide 12 mm ϕ bars at 6 cm centers.

Design of bottom one metre

$B.M =$ ½ x 4,800 x ½

= 800 kg.m

Area of steel $\frac{800 \times 100}{0.84 \times 1000 \times 31} = 3.07\ cm^2$.

Provide 8 mm ϕ bars at 13 cm centers.

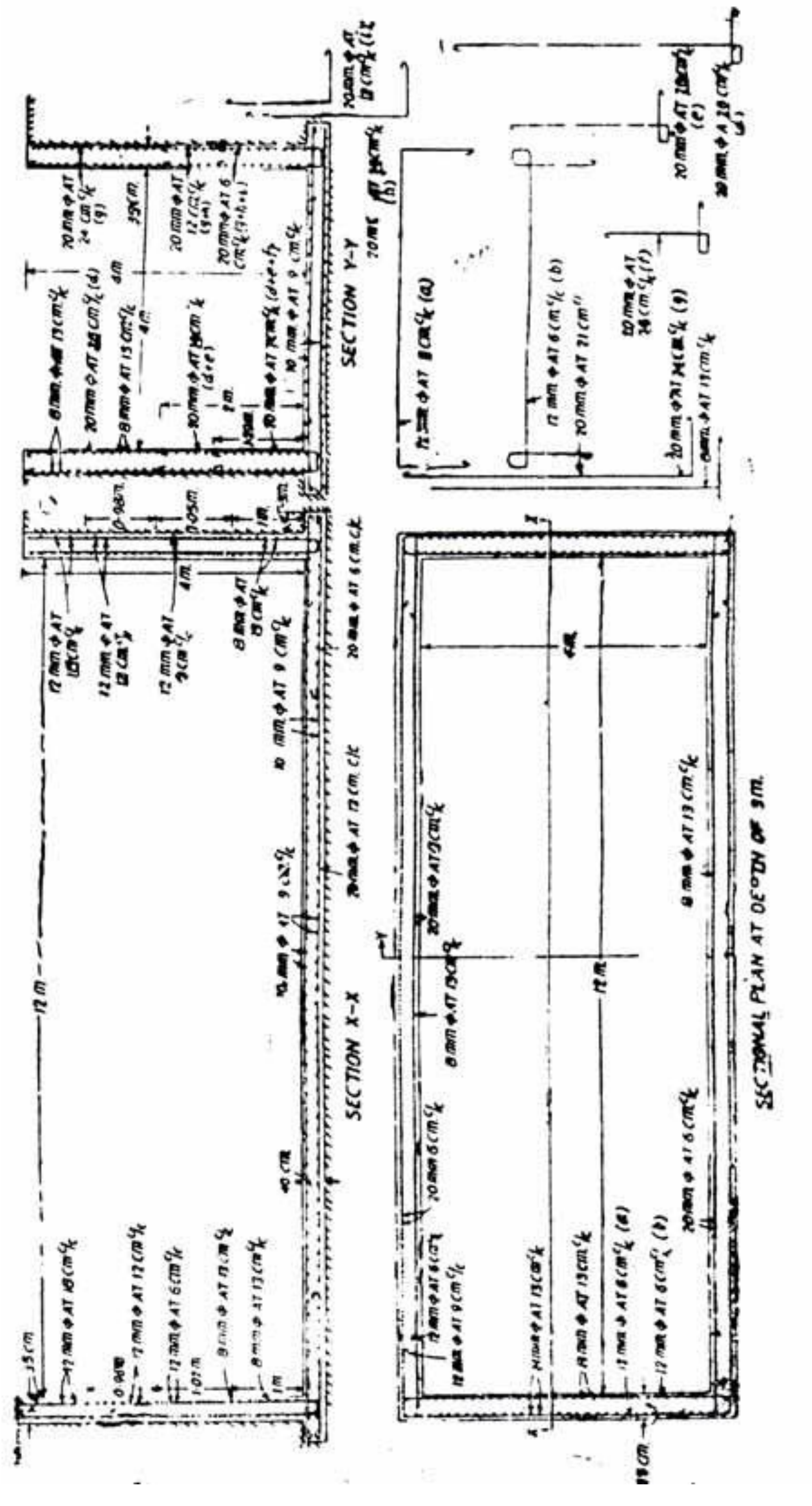


Fig. 15

Base slab

Check Against Uplift

Projection of 0.3 m is provided beyond the face of the walls to add to stability against uplift.

Weight of vertical walls

Long walls = 2 x 12.7 x 0.35 x 4 x 2400 = 85,344 kg.

Short walls = 2 x 4 x 4 x 0.35 x 2400 = 26,880 kg.

Base slab = 13.3 x 5.30 x 0.4 x 2400 = 67,674 kg.

Weight of earth on projection = 2(13.3 + 4.7) x 4 x 0.3 x 1600

= 69,120 kg.

Uplift pressure due to pressure of water at bottom of tank

= 5.3 x 13.3 x 4.4 x 1000

= 249,014 kg.

Total downward weight

= 85,344 + 26,880 + 67,670 + 69,120

= 249,014 kg.

Frictional resistance required

= 310,200 – 249,012

= 61,188 kg.

Pressure of submerged earth and water at depth of 4.4 m.

$= 1000 \times 4.4 + \frac{1}{3}(1600\text{-}1000) \times 4.4$

$= 4400 + 880 = 5280 \text{ kg/m}^2$

Total pressure per one metre length of walls

= ½ x 4.4 x 5280

= 11,616 kg.

As the soil is saturated, the angle of friction of submerged soil will be low. Assuming coefficient of friction as 0.15, frictional resistance per metre length of wall

= 0.15 x11616

= 1742.4 kg.

Total frictional resistance of four sides

= 2(5.3 + 13.3) x 1742 .4

= 64,820 > 61,188 Safe.

Design of Base Slab

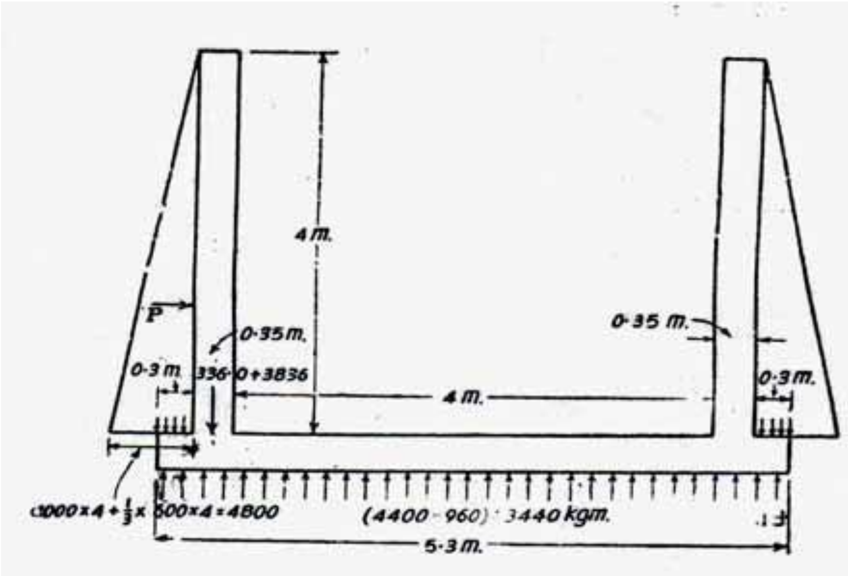


Fig. 16

Consider one metre length of slab.

Upward pressure of water per sq. metre

$= 4.4 \times 1000 = 4400$ kg/m^2.

Self weight of slab $= 1 \times 1 \times 0.4 \times 2400 = 960$ kg/m^2.

Net upward pressure $= 4400 - 960 = 3440$ kg/m^2.

Weight of wall per metre run

$= 0.35 \times 4 \times 2400 = 3360$ kg.

Weight of earth on projection

$= 1600 \times 4.0 = 6400$ kg/m^2.

Net unbalanced force

$= 3440 \times 5.3 - 2(3360 + 6400 \times 0.3)$

$= 18{,}232 - 10{,}560$

$= 7672$ kg.

Reaction on each wall

$= \dfrac{7672}{2} = 3836$ kg.

B.M. at edge of cantilever portion

$$= \frac{3440 \times 0.3^2}{2} + 4800 \text{ x } \frac{4}{2}\left(\frac{4}{3} + 0.2\right) - 6400 \text{ x } \frac{0.3^2}{2}$$

$= 154.8 + 14{,}720 – 288$

$= 14{,}586.8$ kg. m.

B.M at center of span

$$= \frac{3440}{2} \times \left(\frac{5.3}{2}\right)^2 + 4800 \times \frac{4}{2} \times \left(\frac{4}{3} + 0.2\right) - (3360 + 3836) \left(\frac{5.3}{2} - 0.3 - \frac{0.35}{2}\right) - 6400 \text{ x } 0.3 \times \left(\frac{5.3}{2} - 0.15\right)$$

$= 12{,}080 + 14{,}720 – 15{,}650 – 4{,}800$

$= 6530$ kg.m.

Effective depth required

$$= \sqrt{\frac{14586.8 \times 100}{14.11 \times 100}}$$

$= 32.15$ cm

Provide overall depth of 40 cm. with effective depth of 35 cm.

Area of steel required at support

$$= \frac{14{,}586.8 \times 100}{0.84 \times 1000 \times 35}$$

$= 49.58 \text{ cm}^2$.

Provide 20 mm ϕ at bars at 12 cm. centers.

Reinforcement at centre of span

$$= \frac{6350 \times 100}{0.84 \times 1000 \times 35}$$

$= 21.60 \text{ cm}^2$.

Provide 20 mm ϕ bars at 12 cm. centres.

Minimum reinforcement

$$= 0.3 – 0.1 \text{ x } \frac{30}{35}$$

$= 0.3 – 0.09 = 0.21$

Reinforcement $= \dfrac{0.21}{100} \times 40 \times 100$

$= 8.4 \text{ cm}^2$

Provide 10 mm ϕ bars at 9 cm *.c/c* as distribution steel at bottom and 10 mm ϕ bars at 9 cm. *c/c* both ways at top.

Maximum *S. F* at edge

$= 3360 + 3836 + 6400 \times 0.3 - 3440(0.3 + 0.35)$

$= 3360 + 3836 + 1920 - 2236$

$= 6880$ kg.

Shear stress $= \dfrac{6880}{0.84 \times 100 \times 35}$

$= 2.24 \text{ kg/cm}^2$. Safe.

11. Overhead Tanks These tanks may be rectangular or circular. The tanks are supported on staging which consists of masonry tower or a number of columns braced together. The tank walls are designed in the same way as the walls of tanks resting on the ground. The base slab of circular tanks is designed as circular slab supported on masonry or circular beam at the end. The slab of rectangular tanks is designed as two-way slab if length is less than twice the breadth the slab is designed as one-way slab. The base slab is subjected to bending moment at the end to direct tension, caused by the water pressure acting on vertical walls.

For large tanks base slab is supported on series of beams supported on columns.

The staging consists of a number of columns braced together at intervals. The columns are assumed to be fixed at the braces as well as to elevated tank, therefore, effective length of column is taken as distance between bracings.

The wind force acting on the tank and staging produces tension on the windward side columns and compression on the leeward side columns. The force in any column is proportional to its distance from C.G. of the column group.

Let '*P*' be the total wind force acting at height '*h*' from the base and r_1, r_2........... be the distances of the columns from the C.G. of the column group, measured parallel to the direction of the wind.

Force F_1 in column 1 at distance r_1 from C.G. of the column group is given by

$$F_1 = \frac{phr_1}{r_1^2 + r_2^2 + ...}$$

$$= \frac{phr_1}{\Sigma r^2}$$

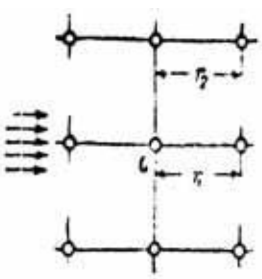


Fig. 17

In the design it is assumed that horizontal shear taken by inner columns is twice that taken by outer columns.

The bracings are designed for B.M. and shear. Same reinforcement is put at top and bottom as the may blow from one side or the other.

Circular tanks are sometimes provided with inclined columns. In such cases the vertical component of the force to each column is found as given above. The horizontal shear in each column is given by deducting the sum of horizontal components of the forces in the columns from the wind force and dividing by number of columns.

The moments in the inclined braces meeting at a column can be found as follows-

The axes of moments in column above and below the brace will be at right angles to the direction of the wind. The axes of moments in the two braces will be at right angles to their axes. By completing the triangle of moments, the moments, the moments in the braces can be found.

Let O be the column and OA and OB be the braces meeting at column O. Oa is drawn perpendicular to OA and OB is drawn perpendicular to OB and ab is drawn at right angles to the direction of wind, ab gives moment in the column Oa and Ob will give the moments in braces OA and OB respectively. For moment to be maximum in OA, the wind should blow at right angles to the other brace OB. In such a case triangle of moments will be right angled triangle and side Oa will be hypotenuse.

Foundation for elevated tanks. The foundation for elevated tank columns may be combined foundation in the form of raft or independent footing may be provided for each column.

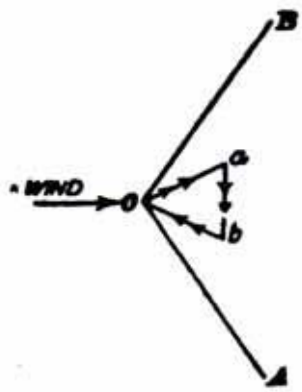


Fig. 18

Ex. 6. Design a circular tank having diameter of 6 m. and height of 3 m. The tank is supported on masonry tower.

Sol. The tank is covered with domed roof. Rise of 1 m. is provided for the dome.

Radius of dome is given by

$$3 \times 3 = 1 \times (2R - 1)$$

$\therefore \quad R = 5$ m.

Thickness of 10 cm. is provided for dome.

Self-load per square metre

$= 10 \times 24 = 240$ kg.

Live load per square metre

$= 150$ kg.

Total load = 390 kg/m^2.

Hoop stress at any angle θ is given by

$$\frac{wR}{t} \times \frac{(\cos^2\theta + \cos\theta - 1)}{(1+\cos\theta)}$$

Meridional stress

$$= \frac{(wR(1-\cos\theta))}{t \sin^2}$$

$\sin\phi = \frac{3}{5} = 0.6,$ $\qquad \phi = 36° \, 52'$

$\cos\phi = \frac{4}{5}$

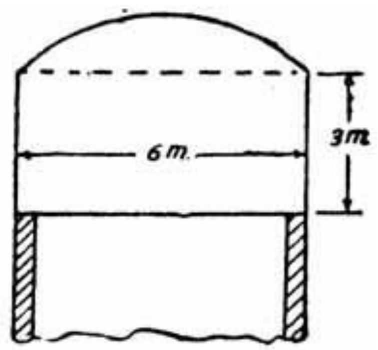


Fig. 9 (a)

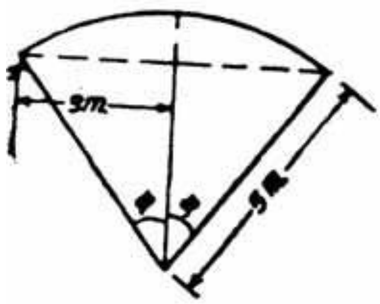


Fig. 9 (b)

$$\text{Meridional stress} = \frac{390 \times 5}{0.1} \frac{(1-\cos\theta)}{\sin^2\theta} \times 10^{-4} \text{ kg/cm}^2$$

$$= 1.95 \times \frac{(1-\cos\theta)}{\sin^2\theta}$$

Hoop stress = $\dfrac{390 \text{ x } 5\,(\cos^2\theta + \cos\theta - 1)}{0.1(1+\cos\theta)} x10^{-4}\ kg\ /cm^2$

$$= \frac{1.95\,(\cos^2\theta + \cos\theta - 1)}{(1+\cos\theta)} kg\ /cm^2.$$

Maximum meridional stress occurs at

$\theta = \phi = 36°52'$.

Meridional stress = 1.95 x $\dfrac{(1- 0.8)}{0.6 \text{ x } 0.6}$

=1.08 kg/ cm^2.

Maximum hoop stress occurs at

$\theta = 0°$

Hoop stress = 1.95 x $\dfrac{(1+1-1)}{1+1}$

= 0.975 kg/cm^2.

Stresses are very small. Provide nominal reinforcement of 8 mm ϕ bars at 20 cm. centres both ways.

To resist the horizontal component of the meridional thrust at the end of done a ring is provided. The ring beam will be subjected to hoop tension.

Design of Ring Beam. Meridional thrust per 1 cm. length

= 1.08 x 10 x 1 = 10.8 kg

Horizontal component of meridional thrust

= 10.8 cos ϕ

= 10.8 x 0.8.

Hoop tension = $\dfrac{pd}{2}$ =10.8 x 0.8 x $\dfrac{600}{2}$

= 2592 kg.

Area of steel required

= $\dfrac{2592}{1400}$ =1.85 c m^2 .

Provide 4 bars of 10 mm ϕ. Area provided

= 3.14 cm^2.

6 mm ϕ stirrups at 20 cm. centres are provided.

20 cm x 15 cm. section of ring beam is provided.

Stress in concrete. Equivalent area

= 20 x 15 + 12 x 3.14

= 300 + 37.68

= 337.68 cm^2.

Tensile stress = $\dfrac{2592}{337.68}$ = 7.68 kg / cm^2. Safe.

Design of Cylindrical Walls. As the dome is designed on membrane theory, the tank wall is assumed free at top.

H = 3 m., D = 6 m. Assume thickness of tank wall = 15 cm.

$$\frac{H^2}{Dt} = \frac{3^2}{6 \times 0.15} = 10$$

From table 17.2.

Maximum hoop tension

= 0.608 wHR

= 0.608 x 1000 x 3 x 3

= 5472 kg.

Maximum hoop tension occurs at depth of

0.6 H = 1.8 m. from top.

Maximum - ve *B.M.* = 0.0122 wH^2

= 0.0122 x 1000 x 27

= 329.4 kg. m.

Maximum -ve *B.M.* occurs at base.

Maximum +ve *B.M.* = 0.0029 x 1000 x 27

= 78.3 kg. m.

Maximum + ve *B.M.* occurs at depth of

0.7 H = 2.1 m. from top.

Maximum shear at base

= 0.158 wH^{2ss}

= 0.158 x 1000 x 9

= 1422 kg.

Steel required for hoop tension

$$= \frac{5472}{1000}$$

= 5.472 cm^2.

Provide 8 mm ϕ bars at 18 cm. centres on both faces.

Minimum steel required

$$= 0.3 - \frac{0.1 \times (15 - 10)}{(45 - 10)}$$

= 0.286%

Steel required $= \frac{0.286 \times 15 \times 100}{100} = 4.29\ cm^2$.

Provide 8 mm ϕ bars at 18 cm. centres throughout the height.

Steel required for - ve *B.M.*

Effective depth = 15 - 4 = 11 cm.

$$A_t = \frac{329.4 \times 100}{0.84 \times 11 \times 1000} = 3.55 \text{ cm}^2$$

Provide 8 mm ϕ bars at 14 cm centres. Every third bar will be stopped at 1 m. from base, the remaining bars will be taken upto top to serve as distribution steel. On the other face 8 mm ϕ bars at 21 cm. centres are provided. The reinforcement will resist +ve B.M.

Design of bottom slab. The bottom slab will be treated as having edges clamped. Assume 22 cm. thickness of slab.

Self load of slab = 0.22 x 1 x 1x 2400 = 528 kg/m^2.
Load of water = 3 x 1000 = 3000 kg/m^2.
Total load = 3000 + 528 = 3528 kg/m^2.

In circular slab of radius '*a*' and uniformly loaded with load of intensity '*q*'.
Circumferential moment

$$= \frac{q}{16} a^2 .$$

Radial moment +ve $= \frac{q}{16} a^2$.

- ve $= \frac{2q}{16} a^2$.

Maximum radial negative moment

$$= \frac{2 \times 3528 \times 3^2}{16} = 3969 \text{ kg. m.}$$

Effective depth required

$$= \sqrt{\frac{3969 \times 100}{14.11 \times 100}} \text{ (tension on water side).}$$

= 16.8 cm

Provide overall depth of 22 cm. with effective depth of 18 cm.

$$A_t = \frac{3969 \times 100}{0.84 \times 18 \times 1000} = 26.2 cm^2$$

Provide 16 mm ϕ bars at 7 cm. centres 1.2 m from edge.

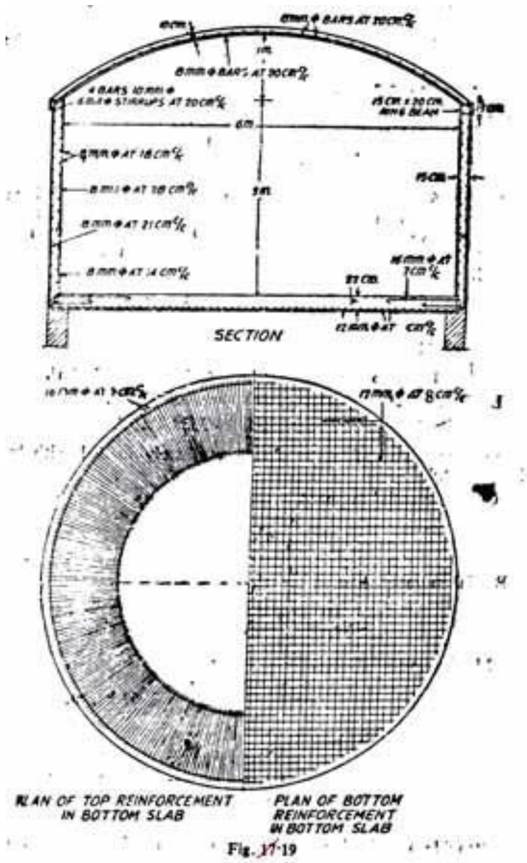


Fig. 19

+ ve radial moment $= \dfrac{q}{16} a^2$.

$$= \frac{3528 \times 3^2}{16} = 1984.5 \text{ kg. m.}$$

Circumferential moment

$= 1984.5$ kg. m.

Area of steel $= \dfrac{1984.5 \times 100}{0.84 \times 18 \times 1000}$ (tension outside) $= 13.0$ cm^2

Provide 12 mm ϕ bars at 8 cm. centres both way throughout.

12. Elevated Rectangular Tanks. Design of elevated tanks open at top is similar to the tanks resting on the ground. If the tank is covered at the top the vertical walls are considered to be supported at the top. In such cases if the ratio of width of tank wall to height of tank is between 0.5 to 2, the coefficients for moments in two directions vertical and horizontal can be obtained from Table 5. The walls are designed for these moments. In case the width of the tank wall is greater than twice the height of tank it will be designed spanning vertically, simply supported at top and fixed at the base. In case the width of tank wall is less than half the height of tank, it is designed spanning horizontally and fixed at the junctions.

The roof slab is designed as two-way slab if the length does not exceed twice the breadth of the tank. In case the length of tank is more than twice the breadth of the tank, the slab is designed oneway spanning along the shorter span.

The design of bottom slab depends on the system of columns and beams provided.

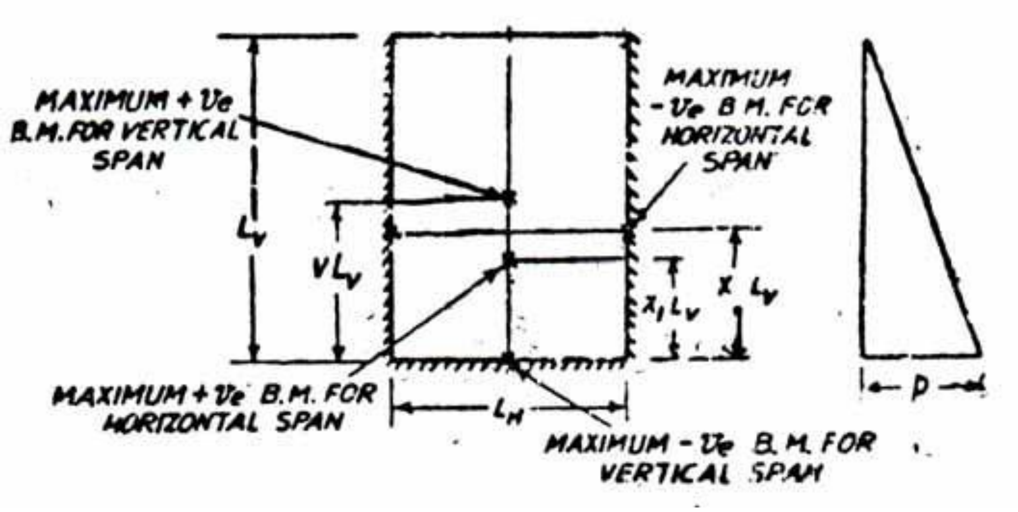


Fig. 20

B.M. = kpl^2

where l is the span,

k is coefficient given in Table 5

and p is pressure, $p = w \times Lv$.

For $\frac{L_H}{L_V} \geq 2$, whole load will act along the vertical span and

maximum -ve *B.M.* = $\frac{pLv^2}{15}$ and + ve *B.M.* = $\frac{pLv^2}{33.5}$

Table 5

$\frac{L_H}{L_V}$	X	X_1	V	Max. -ve B.M. for vertical span	Max. +ve B.M. for vertical span	Max -ve B.M for horizontal span	Max. +ve B.M. for horizontal span
0.5	0.18	0.40	0.25	0.007	0.006	0.050	0.034
1.0	0.33	0.47	0.40	0.026	0.013	0.026	0.012
1.5	0.42	0.48	0.45	.043	0.027	0.013	0.004
2.0	0.45	0.49	0.48	0.055	0.032	0.002	0.001

For $\frac{L_H}{L_V} < 0.5$, whole load will act along the horizontal span and maximum -ve

$B.M. = \frac{pL_H^{\,2}}{12}$ and maximum + ve $B.M. = \frac{pL^2_H}{16}$

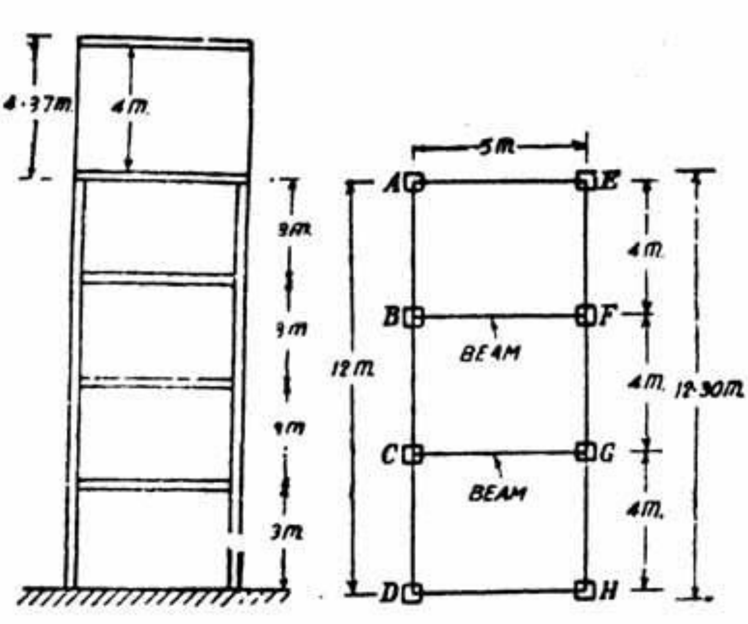


Fig. 21

Ex. 17.7. Design an elevated rectangular tank 12 m. x 5 m. x 4m high. The bottom of tank is 12 m. above ground level. The tank is covered at top.
Bearing capacity of soil is 15,000 kg/m^2.

Sol. The tank will be supported on 8 columns, spaced at 4 m. centres as shown in Fig. 17.21. The columns are braced together at intervals of 3 m.

Concrete mix used is *M* 200.

Design of Roof Slab. Roof slab is designed for a live load of 150 kg/m^2.
Assume thickness of roof slab as 15 cm.

Self load = 0.15 x 1 x 1 x 2400 = 360 kg/m^2.

Live load = 150 kg/m^2.

Total load = 510 kg/m^2.

As the ratio of length to breadth of slab is greater than 2, slab will span in shorter direction.

$$\text{Maximum } B.M. = \frac{wl^2}{8} = \frac{510 \text{ x } 5^2}{8}$$

= 1593.75 kg. m.

= 159,375 kg. cm.

For mix. M 200, C = 70, t = 1400, m = 13, Q = 12.10.

Effective depth $= \sqrt{\dfrac{159.375}{12.1 \text{x} 100}} = 11.48$ cm.

Provide overall depth of 15 cm. with effective depth of 13 cm.

$$A_t = \frac{159{,}375}{1400 \text{ x } 0.87 \text{ x } 13}$$

$= 10.07 \text{ cm}^2$.

Provide 12 mm ϕ bars at 11 cm. centre.

$$\text{Distribution steel} = \frac{0.15 \times 15 \times 100}{100}$$

$= 2.25 \text{ cm}^2$.

Provide 6 mm ϕ bars at 12 cm centres.

Design of Long Walls. As the length of long wall is greater than twice the height of the wall, the wall will be designed as spanning vertically,

p = 4 x 1000 = 4000 kg/m^2

Maximum *B.M.* at base

$$= \frac{pl^2}{15} = \frac{4000 \text{ x } 4^2}{15} = \frac{64{,}000}{15}$$

= 4266.66 kg. m. = 426,666 kg. cm.

The tension will be on water side

$c = 70$, $t = 1000$, $Q = 14.11$, $j = 0.84$.

Effective depth $= \sqrt{\dfrac{426.666}{14.11 \text{ x } 100}}$. = 17.39 cm

Provide overall depth of 22 cm. with effective depth of 18 cm.

Area of steel required

$$= \frac{426666}{0.84 \text{ x } 1000 \text{ x } 18} = 28.05 \text{ cm}^2.$$

Provide 20 mm. ϕ bars at 11 cm. centres.

A provided $= 28.56 \text{ cm}^2$.

Maximum + ve *B.M.*

$$= \frac{pLv^2}{33.5} = \frac{4000 \text{ x } 4^2}{33.5}$$

= 1911 kg. m.

= 191,100 kg. cm.

Area of steel required $= \dfrac{191{,}100}{0.84 \text{ x } 1000 \text{ x } 18} = 12.60 \text{ cm}^2$.

Provide 12 mm ϕ bars at 9 cm. centres.

Distribution Steel

Distribution steel $= \left(0.3 - \frac{0.1 \times 12}{35}\right)\% = 0.266\ \%$

Steel area $= \frac{0.266 \times 22 \times 100}{100} = 5.852\ cm^2$.

Half the reinforcement is provided on each face. Provide 8 mm ϕ at 17 cm. centres on each face. There will be direct load on the long wall due to self weight of the roof. It is assumed that whole load from roof is transferred to long walls.

Load per metre length of wall

$= 510 \times \frac{5}{2} + 528 \times 4$

$= 1275 + 2112 = 3{,}387$ kg.

Direct load is very small, its effect is neglected. Actually reinforcement is provided on both faces vertically as well as horizontally,

There will be direct tension in long walls due to pressure on short walls but its effect is negligible.

Design of Short Walls

$$\frac{L_H}{L_V} = \frac{5}{4} = 1.25.$$

Coefficients for *B.M.* are obtained from Table 17.5 by linear interpolation.

$p = 4 \times 1000 = 4000\ kg/m^2$.

Maximum - ve *B M.* for vertical span

$= 0.0345 \times 4000 \times 4^2$

$= 2208$ kg. m.

Maximum + ve *B.M.* for vertical span

$= 0.02 \times 4000 \times 4^2$

$= 1280$ kg. m.

Maximum -ve *B.M.* for horizontal span

$= 0.0195 \times 4000 \times 5^2$

$= 1950$ kg. m.

Maximum +ve *B.M.* for horizontal span

$= 0.008 \times 4000 \times 5^2$

$= 800$ kg m.

Overall thickness of 22 cm. is provided.

Steel in Vertical Span

For -ve *B.M.* $= \frac{2208 \times 100}{0.84 \times 18 \times 1000}$ [tension on water face]

$= 14.6\ cm^2$.

Provide 12 mm. ϕ bars at 7cm. centre

For + ve *B.M.* $= \dfrac{1280 \times 100}{0.84 \times 18 \times 1000}$ [tension outside]

$= 8.40\ cm^2$.

Provide 10 mm. ϕ bars at 9 cm. centres.

Steel in Horizontal Span.

For -ve *B.M.* $= \dfrac{1950 \times 100}{0.84 \times (18 - 0.6 - 0.6) \times 1000}$

[tension on water side]

$= 14.0\ cm^2$.

Provide 12 mm. ϕ bars at 8 cm. centres.

For + ve *B.M.* $= \dfrac{800 \times 100}{0.84 \times 16.8 \times 1000}$ (tension outside)

$= 5.62\ cm^2$.

Provide 12 mm. ϕ bars at 16 cm. centres.

There will be small amount of direct tension in short walls due to water pressure on long walls but its effect will be very small and is neglected.

Design of base slab The slab is built monolithic with beam on all the four edges. For findin bending moment coefficients it will be considered continuous on all four edges.

Bending moment coefficients are obtained by interpolation from Table 8.5

$$\frac{L_H}{L_V} = \frac{5}{4} = 1.25.$$

Assume 22 cm. thick slab.

Self load = 0.22 x 1 x 1 x 2400 = 528 kg/m^2.

Load of water 4 x 1000 = 4000 kg/m^2.

Total load = 4528 kg/m^2.

***B.M.* in Short Span**

-ve *B.M.* at continuous edge

$= 0.0475 \times 4528 \times 4^3$

= 3985 kg m.

+ ve *B.M.* at mid-span

$= 0.036 \times 4528 \times 4^2$

= 2610 kg. m.

***B. M.* in Long Span**

-ve *B.M.* at continuous edge

$= 0.033 \times 4528 \times 4^2$

= 2390 kg. m.

+ve *B.M.* at mid - span

$= 0.025 \times 4528 \times 4^2$

= 1812 kg. m.

Maximum *B.M.* = 3985 kg.m. (tension on water side0

c = 70, t = 1000, Q = 14.11, n = 0.48, j = 0.84

Effective depth required

$$= \sqrt{\frac{3985 \text{x} 100}{14.11 \text{x} 100}} = 16.85 \text{ cm.}$$

Provide overall depth of 22 cm. with effective depth of 18 cm.

Steel in Short Span

Steel required for -ve *B.M.* at continuous edge

$$= \frac{3985 \text{x} 100}{0.84 \text{x} 1000 \text{x} 18}$$

$= 26.3 \text{ cm}^2$.

Provide 16 mm. ϕ bars at 7 cm. centres.

*A*t provided $= 28.73 \text{ cm}^2$.

Steel for +ve *B.M.* $= \frac{2610 \times 100}{0.84 \times 1000 \times 18} = 17.28 \text{ cm}^2$.

Provide 16 mm. ϕ bars at 11 cm. centres.

Steel in Long Span

Effective depth = 18-0.8-0.8 = 16.4 cm.

Steel for +ve *B.M.* $= \frac{2390 \times 100}{0.84 \times 1000 \times 16.4} = 17.28 \text{ cm}^2$.

Provide 16 mm. ϕ bars at 11 cm. centres.

$A_t = 18.27 \text{ cm}^2$.

Steel for + ve *B.M.* $= \frac{1812 \times 100}{0.84 \times 1000 \times 16.4}$

$= 13.0 \text{ cm}^2$.

Half the bars from support are bent to the mid-span giving 16 mm. ϕ at 22 cm. centres and steel area of $\frac{18.27}{2}$.

Remaining steel required = $13.0 - \frac{18.27}{2} = 3.86 \text{ cm}^2$.

10 mm. ϕ bars at 22 cm. centres are provided.

Design of Beams

Design of Beams B_1

Load from slab = w = 4528 kg/m^2.

Self weight = 0.48 x 0.3 x 2400

= 345.6 kg/m.

The loading on the beam is shown in

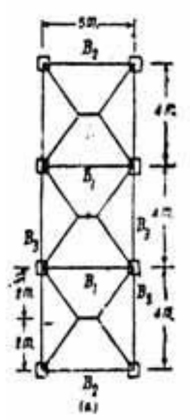


Fig. 22 (a).

As the beam is built monolithic with columns, partial fixity is assumed at the ends. *B.M.* taken for design purpose will be 0.8 times simply supported bending moment.

Max. *B.M.* = $0.8\left[6w \times 2.5 - \frac{4w}{2} \times 2\left(\frac{2}{3} + 0.5\right) - 4w \times 0.5 \times 0.25 + \frac{345.6 \times 5^2}{8}\right]$

= $0.8\left[15w - 4w \times \frac{7}{6} - 0.5w + 1080\right]$

= $0.8\,[9.83w + 1080]$

= 0.8 [9.83 x 4528 + 1080]

= 36,472 kg. m.

= 3,647,200 kg. cm.

Concrete mix used is *M* 200.

The beam is designed as *T*-beam.

Effective depth = 70-8 = 62 cm.

Assume lever arm = $62 - \frac{22}{2}$

= 51 cm.

Area of steel = $\frac{3{,}647{,}200}{51 \times 1400}$

= 51.2 cm^2.

Provide 4 bars of 32 mm. ϕ and 4 bars of 25 mm. ϕ .

Steel area provided = 38.17 + 19.63

= 51.80 cm^2.

Check for Stress

Flange width will be least of

(i) Centre to centre = 200 cm.

(ii) $\quad ^1/_3$ rd span $\dfrac{500}{3} = 167$

(iii) $\quad 12\, d_s + b_r = 12 \times 22 + 30 = 294$ cm.

Flange width = 167 cm.

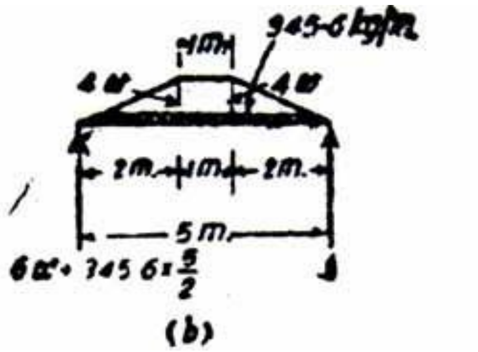


Fig. 22 (b)

Assume neutral axis to lie in the flange.

$$167 \times \frac{n^2}{2} = 13 \times 51.8\,(62-n)$$

$$n^2 + 8.06n = 500$$

$$(n + 4.03)^2 = 500 + 16.24$$

$$= 516.24$$

$$\therefore\ n = 22.72 - 4.03$$

$$= 18.69 \text{ cm.}$$

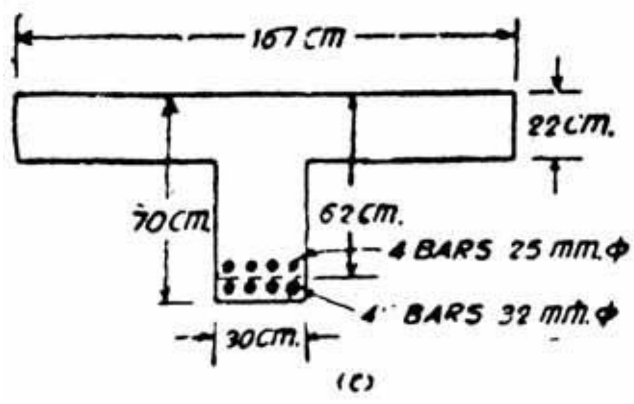


Fig 22 (c)

Maximum stress in steel

$$= \frac{3{,}647{,}200}{\left(62 - \dfrac{18.69}{3}\right) \times 51.8}$$

$$= 1263 \text{ kg/cm}^2.$$

Maximum stress in concrete

$$= \frac{t}{m} \times \frac{n}{d-n} = \frac{1263 \times 18.69}{13 \times 43.31}$$

= 42 kg/cm^2.

Safe

Maximum shear force

$$= 4528 \text{ x } 6 + 345.6 \text{ x } \frac{5}{2}$$

= 27,168 + 864 = 28,032 kg.

Shear stress $= \dfrac{28{,}032}{0.87\text{x}62\text{x}30}$

= 17.28 kg/cm^2.

Shear force corresponding to shear intensity of 7 kg/cm.^{2}

= 7 x 0.87 x 62 x 30 = 11,327 kg.

Shear at 2m. from support

$$= 6 \text{ x } 4528 - \tfrac{1}{2} \text{ x } 4 \text{ x } 4528 \text{ x } 2 + \frac{345.6\text{x}5}{2} - 345.6 \text{ x } 2$$

= 9056 + 172.8 = 9228.8 kg.

No shear reinforcement is required beyond this point. 4 bars of 25 mm. ϕ, two at a time, and 2 bars of 32 mm. ϕ are bent at intervals of 0.5 m.

Shear taken by two bars of 32 mm. ϕ

= 2 x 1250 x 8.04 x 0.707

= 14,200 kg

Net shear = 28,032 - 14,200

=13,832 kg.

Providing 12 mm. ϕ 2-legged stirrups,

pitch $= \dfrac{2\text{x}1.13\text{x}1250\text{x}0.87\text{x}62}{13{,}832}$

= 108 cm.

12 mm. ϕ 2-legged stirrups at 8 cm. centres are provided.

Shear at 0.5 m. from support

$$= 28{,}032 - \frac{0.5}{2} \text{x } w - \frac{345.6}{2}$$

= 28,032 - 1132 - 172.8

= 26,727.2 kg.

At this section two bars of 25 mm. ϕ bent up are effective,

Shear taken by 2 bars 25 mm. ϕ

= 2 x 1250 x 4.91 x 0.707

= 8700 kg.

Net shear for which stirrups are required

= 26,727.2 - 8700 = 18,027.2 kg.

Providing 12 mm. ϕ 2-legged stirrups at 8 cm. centres.

Shear at 1.5 m. from support

$$= 28{,}032 - \frac{3wx1.5}{2} - \frac{345.6x3}{2}$$

= 17,326 kg.

Provide 12 mm. ϕ 2-legged stirrups at 8 cm. centres upto 2 m. from support. In the remaining portion 12 mm. ϕ stirrups at 50 cm. centres are provided.

Design of Beam B2. The side wall will serve the purpose of beam. It is assumed that weight of roof is taken by long walls.

Self weight of wall per metre run = 2112 kg.

Weight of water and base slab will be half of that coming on beam $B1$.

Maximum $B.M$. at centre

$$= 0.8\left[\frac{2112 \times 5^2}{8} + 3 \times 4528 \times 2.5 - 2 \times 4528 \times \frac{1}{2} \times \left(\frac{2}{2 \times 5} + 0.5\right) - 2 \times 4528 \times 0.5 \times 0.25\right]$$

$$= 0.8\left[\frac{2112 \times 5^2}{8} + 3 \times 4528 \times 2.5 - 2 \times 4528 \times \frac{7}{6} - \frac{4528}{4}\right]$$

= 23,090.4 kg. m.

Overall depth = 422 cm.

Assume effective depth = 416 cm.

$$\text{Area of steel required} = \frac{23{,}090.4x100}{0.86x412x1250} = 4.9\ cm^2$$

B.M at center of AB

$$= 18{,}0486 - \frac{14{,}475.1}{2}$$

= 10, 811 kg. m.

B.M at center of BC = 18,048.6 - 14,475.1

= 3573.5 kg. m

Taking moments about B,

R_A x 4 + 14,475.1- 3387 x 4 x 2- ½ x 4 x 2 x 4528 x 2 = 0.

R_A = 6774 + 9056-3350

= 12,480 kg.

R_D = 12,480 kg.

R_B = R_C = ½ [3387 x 12 + 3 x 1/2 x 4 x 2 x 4528 -2 x 12,480]

= 20, 322 + 27,168 - 12,480

= 35,010 kg.

S.F. diagram is shown in Fig. 32

Maximum *B.M* = 14,475.1 kg. m.

Overall depth of wall = 422 cm.

Assume effective depth = 416 cm.

Area of steel required = $\frac{14,475.1x100}{0.84x412x1000}$

= 4.16cm^2

Provide 3 bars of 16mm.∅.

Area of steel required at center of *AB*

$$= \frac{10.811x100}{0.86x416x1250} = 2.5 \text{ cm}^2$$

Provide 3 bars of 12mm.∅.

Area of steel required at center of *BC*

$$= \frac{3573.5 \times 10010.811 \times 100}{0.86 \times 416 \times 1250} = 0.8\text{cm}^2$$

2 bars of 16mm. ∅are provided.

Shear

Maximum shear force = 19,180 kg.

$$\text{Shear stress} = \frac{19,180}{22 \times 0.87 \times 416}$$

= 2.1 kg/cm^2. Safe

Provide 8mm. ∅stirrups at 30cm. centers

Design of columns

Load on column *A* when tank full = Reactions from beams B_2 and B_3

= 18,864 + 12,480 = 31.344 kg

Provide 3 bars of 20mm. ∅.

$$\text{Shear force} = 3 \text{ x } 4528 + 2112 \text{ x } \frac{5}{2}$$

=13,584 + 5280 =18,864 kg.

The section will be safe as depth provided is quite considerable.

Provide 8 mm. ∅ 2- legged stirrups at 30cm. centres.

Design of beam B_2. The side walls will serve as girders.

Load of roof and wall = 3387 kg/m.

Load from bottom slab will be triangular as shown in Figm 17m23 (a).

The beam is symmetrical and symmetrically loaded. Moments are found by moment distribution by cutting the beam at center of span *BC* and taking k = ½.

Due to triangular loading fixed end moments are

$$\frac{5wl}{48} = \frac{5}{48} x2\,wx\frac{4}{2}\,x4$$

$$= \frac{5}{3}w = \frac{5}{3}x4528$$

= 7546.6 kg .m.

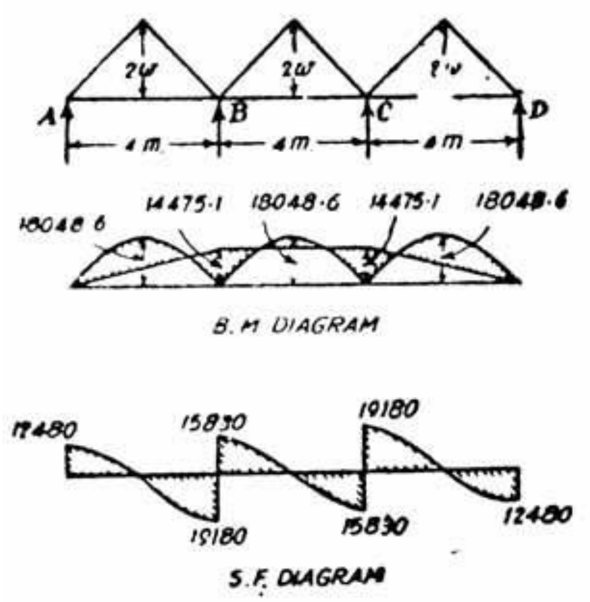


Fig 23

Due to uniformly distributed load fixed end moments are

$$\frac{wl^2}{12} = \frac{3387x4^2}{12} = 4516 \text{ kg. m.}$$

Total *F.E.M.* 7546.6 + 4516 =12,062.6 kg. m.

	A		3/5 B 2/5
F.E.M	-12, 062.6	+12, 062.6	-12, 062.6
Relese	+12, 062.6		
carry over		+6031.4	
Balance		+18,093.1	-12,062.6
		-3618.8	-2412.5
Final		+14,475.1	-14,475.1

Load on column *B* = 35.010 + 28,032

= 63,042 kg.

Total weight of water

= 12 x 5 x 4 x 1000 = 240,000 kg.

(For calculation of weight of water center line dimensions are taken as while calculating load on beams centre line dimensions were taken).

Columns *B*,*C*,*F* and *G* will take double the load of water taken by *A*, *E* , *D* and *H*.

Weight of water on column *A*

$$= \frac{240,000}{12} = 20,000kg.$$

Weight of water taken by column B

= 40,000 kg.

Load on column *B* when tank empty

= 31,344 - 20,000

= 11,344 kg.

Load on column B when tank empty

= 63,042-40,000

= 23,042 kg

Wind Forces

Intensity of wind pressure is assumed as 150 kg/m.^{2}

Wind force acting on water tank

= 4.37 x 12.22 x 150

= 8003 kg. acting at 14.185 from base

Wind Force on Staging

On columns = 4 x 0.3 x12 x150

= 2160 kg

On bracing = 3 x 0.3 x 12 x 150

= 1620 kg.

Total wind force on staging= 2160 + 1620 = 3780 kg. acting at 6m. from base.

Moment at base = 8003 x 14.185 + 3780 x 6

= 113,600 + 22,680

= 136.280 kg. m.

Σr^2 = 8 x 2.5 = 50

Force in column $= \frac{Mr}{\sum r^2} = \frac{136,280x2.5}{50}$

= 6814 kg. m.

There will be downward force of 6814 kg. in each of leeward columns and upward force on 6814 kg. in each of wind ward columns.

Self weight of column

= 0.3 x 0.4 x 12 x 2400 = 3456 kg

Maximum force in column

= 63,042 + 6814 + 3456

= 73,312 kg.

Maximum force in column

= 11,344 - 6814 = 5530 kg.

No upward load is produced in column when tank empty.

Design of column

Section of column is designed for dead load and checked for wind stresses.

Minimum dead load

$= 63{,}042 + 3456$

$= 66{,}498$ kg.

Section of 30 cm. x 40 cm. is provided.

$66{,}498 = (30 \times 40 - Ac) \times 50 + Ac \times 1300\ Ac.$

$\therefore\ Ac = \dfrac{6498}{1250} = 5.20\ \text{cm}^2$

4bars of 20 mm.ϕ are provide.

Area of steel provide

$= 12.57\ \text{cm}^2$

Minimum diameter of ties

$= \dfrac{20}{4} = 5.00$ mm

Provide 6 mm. ϕ ties at 30 cm. centres.

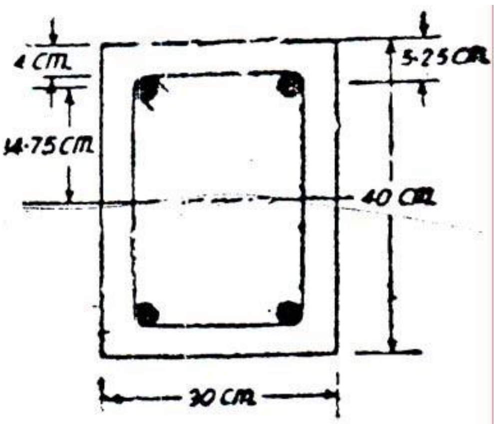


Fig. 24

Check for Stresses with Wind Blowing

Total horizontal force

$= 8003 + 3780 = 11{,}783$ kg.

Force taken by each column

$= \dfrac{11{,}783}{8} = 1473kg.$

Maximum $B.M$ = 1473 x 1.5

= 2210 kg.m.

Maximum load = 73,312 kg.

Eccentricity $= \dfrac{2210x100}{73.312} = 3.014$ cm.

As the eccentricity is very small no tension is induced.

Equivalent area of section

$= 30 \times 40 + 12 \times 4 \times 3.142$

$= 1200 + 150.84 = 1350.84$ cm^2.

Equivalent moment of inertia

$$= \frac{1}{12} x\ 30\ x\ 40^3 + 12\ x\ 4\ x\ 3.142\ x\ 14.75^2$$

$$= 160,000 + 32.790$$

$$= 192,790 \text{ cm}^4$$

Direct stress $= \dfrac{73342}{1350.84} = 54.28$ kg/cm^2.

Bending stress $= \dfrac{2210x100x15}{192,790} = 17.18$ kg/cm^2.

As the effect of wind is taken into consideration allowable stresses can be increased by $33\frac{1}{3}$%.

Allowable direct stress

$$= \frac{4}{3} \times 50 = 66.67 \text{ kg/cm}^2$$

Allowable bending stress

$$= \frac{4}{3} \times 70 = 93.33$$

$$\frac{54.28}{66.67} + \frac{17.18}{93.33} = 08139 + 0.1814$$

$$= 0.998 < 1 \text{ Safe.}$$

Design of Braces

Maximum B.M in the brase

= 2 x 2210 = 4420 kg. m.

= 442,000 kg. cm.

30 cm. x 40 cm. brace is provided.

Effective depth = 40 - 4 = 36 cm.

Moment of resistance of section

= $Qbd^2 = 12.1 \text{ x } 30 \text{ x } 36^2$

= 470,400 kg. cm.

Actual B M. is 442,000 kg. cm.

Area of steel required

$$= \frac{442,000}{0.87x36x1400} = 10.18 \text{ cm.}^2$$

Provide 4 bars of 20 mm. ϕ both at top and bottom as the wind may below from either side.

Maximum shear force

$$= \frac{2x1210}{5} = 884 \text{ kg.}$$

Shear force is very small. Sher stress is negligible.

Provide 6 mm ϕ stirrups at 30 cm. centres.

Design of foundation . For foundation concrete mix of M 150 is used.

Maximum load	= 66,498 kg.
Assume self load	= 6, 600 kg.
Total load	= 73, 098 kg.

Area of footing required

$$= \frac{73,098}{15,000} = 4.87m.^2$$

Provide 2.2 m. x 2.3 m. footing

Net upward pressure on footing $= \frac{66,498}{2.2x2.3} = 13,140 \text{ kg/m}^2$

Maximum $B.M$ $= 2.2 \times 0.95 \times \frac{13,140x0.95}{2}$

$= 13,040$ kg. m.

Effective depth required

$$= \sqrt{\frac{13,040 \times 100}{8.7 \times 40}} = 61.22 \text{ cm.}$$

$$= \frac{66,498}{2 \cdot 2 \times 2 \cdot 3} = 13,140 \text{ kg/m}^2.$$

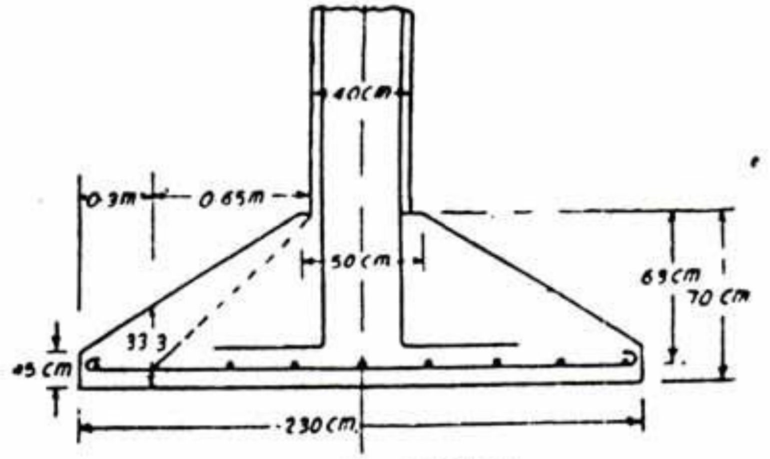


Fig. 17·25 (a)

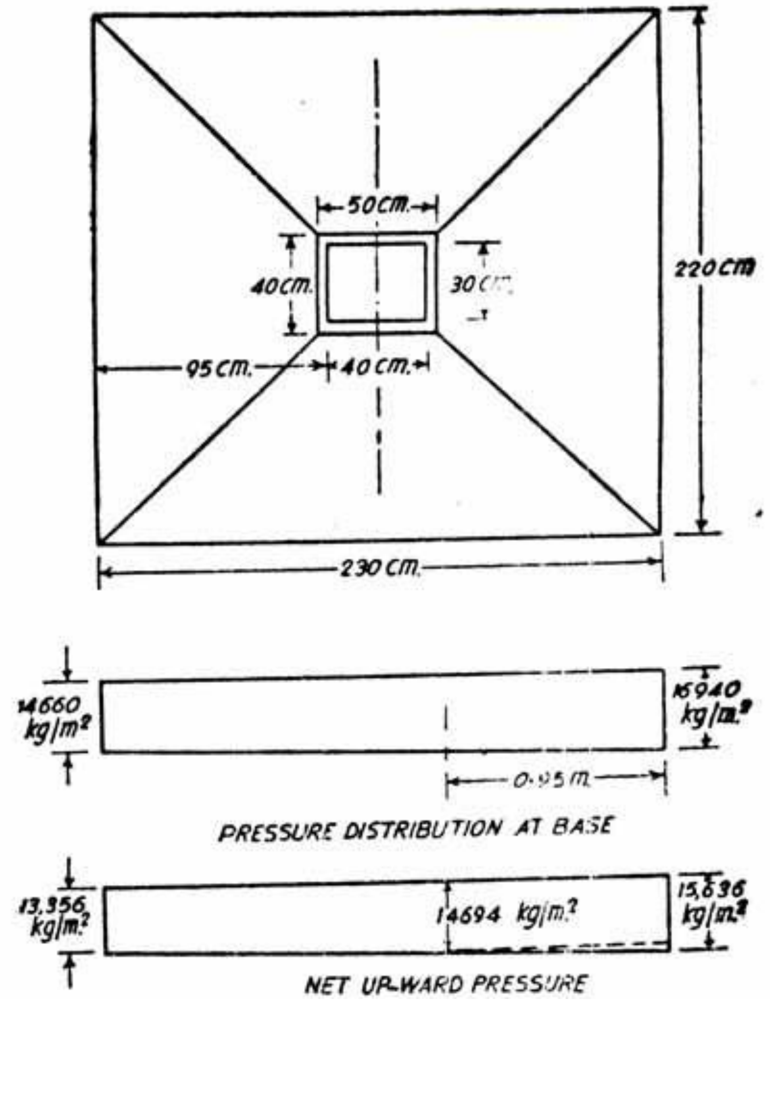


Fig. 17.24 (b)(c)

Depth required for punching

Punching force = 66,498 - 0.3 x 0.4 x 13,140

= 66,498 - 1,577 = 64,921 kg.

Effective depth required for punching

$$= \frac{64{,}921}{2\times(30+40)\times10} = 46{,}37 \text{ cm.}$$

Provide overall deoth of 70cm.with effective depth of 63 cm.

Area of steel required for punching

$$= \frac{13{,}040\times100}{0.87\times6.3\times1400} = 17 \text{ cm}^2$$

Provide 9 bars of 16 mm.φ in both directions.

Check for diagonal shear

Maximum shear = 0.3 x 2.2x13,140

= 8672.4 kg.

$$\text{Shear stress} = \frac{8672.4}{0.87x26.3xx160} = 2.37/\text{cm}^2$$

Safe.

Check for stresses due to wind load

Maximum vertical load = 73,312 kg.

Self load = 6600kg.

Total load = 79,912 kg

B.M = 2210 kg. m.

Pressure distribution on base is given by

$$p = \frac{79{,}912}{2.2\times2.3} \pm \frac{2210}{2.2\times2.3^2/6}$$

= 15,800± 1140

P_{max} = 16,940 kg/m^2

P_{min} = 14,660 kg/m^{2}.

Allowable pressure on ground considering wind effects

$$= \frac{5}{4} \text{x } 15{,}000 = 18{,}750 \text{ kg/m}^2.$$

Pressure at base due to self weight of footing

$$= \frac{6600}{2.2\times2.3} = 1304 \text{ kg/m}^2.$$

Net pressure on footing will very from 16,940-1304

= 15,636 kg/m^{2}. to 14,660 - 1304

= 13,356 kg/m^2

B.M at the edge of column

$$= \left\{14,694\times\frac{0.95\times0.95}{2}+\frac{(15,636-14,694)}{2}\times 0.95\right\} \text{ x } 2.2$$

$$= [\ 6630 + 283.3] \text{ x } 2.2$$

$$= 15,210 \text{ kg cm.}$$

As the effect of wind is taken into consideration, stresses can be increased by $33\frac{1}{3}$ %

Effective depth required

$$= \sqrt{\frac{15,210\times100}{\frac{4}{3}\times8.7\times40}}$$

$$= 54.1$$

Effective depth provided is 63 cm.

Hence O.K.

Area of steel required

$$= \frac{15,210\times100}{0.87\times63\times1400\times\frac{4}{3}} = 15.2\text{cm}^2$$

Area of steel provide is 17cm^2. Safe