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b. Deliver skill based quality education in Civil engineering

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A DETAIL DESIGN OF AN OVERHANGING T-BEAM ROOF OF 10 METER LONG AT A SCHOOL BUILDING OF AGARTALA

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Abstract

T-beam is common used structure in the civil engineering field more than a simple rectangular beam. Normally a T-beam is used in the slab as an intermediate beam. Inverted T-beam are also used as a cycle stand shade. But in the practical case, an overhanging T-beam roof is a rare case in construction site.

In this project, a detail design of an overhanging T-beam roof of 10 meter long at Agartala, is carried out. The hall measures 11m × 7m from inside and it has a wall of 425mm thick. The beam of the T-beam roof project out by 2.2m on one side of the room. The slab is monolithic casting with the beam and it extends to the full length of the beam. For suitable design it is assumed that the roof system carrying a super-imposed load of 2.1kN/m^2. The concrete grade and steel grade are used M20 and Fe415 respectively. The roof system is design as per Indian standard code IS 456:2000. The analysis part was done based on STAAD.Pro.V8i software.In the load combination seismic contribution was considered.

A **T-beam** (or **tee beam**[1]), used in construction, is a load-bearing structure of reinforced concrete, wood or metal, with a t-shaped cross section. The top of the t-shaped cross section serves as a flange or compression member in resisting compressive stresses. The web (vertical section) of the beam below the compression flange serves to resist shear stress and to provide greater separation for the coupled forces of bending.[2]

The T-beam has a big disadvantage compared to an I-beam because it has no bottom flange with which to deal with tensile forces. One way to make a T-beam more efficient structurally is to use an inverted T-beam with a floor slab or bridge deck joining the tops of the beams. Done properly, the slab acts as the compression flange.

T-beam

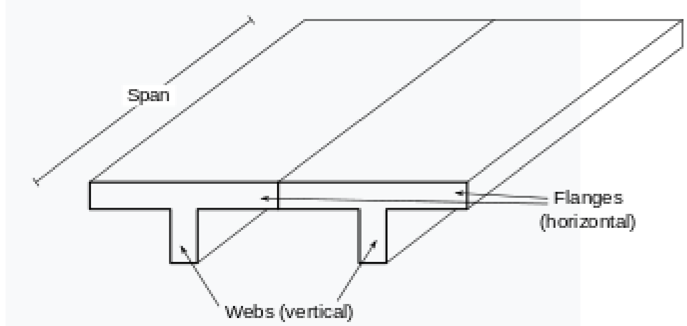


Diagram of two T-beams

History

A T-beam is a structural element able to withstand large loads by resistance in the beam or by internal reinforcements. In some respects, the T-beam dates back to the first time a human formed a bridge with a pier and a deck. After all, a T-beam is, in one sense, no more than a pillar with a horizontal bed on top, or, in the case of the inverted T-beam, on the bottom.[3] The upright portion carrying the tension of the beam is termed a web or stem, and the horizontal part that carries the compression is termed a flange. However, the materials used have changed over the years but the basic structure is the same. T-beams structures such as highway overpasses, buildings and parking garages, have extra material added on the underside where the web joins the flange to reduce the T-beam's vulnerability to shear stress.[4] However, when one investigates more deeply into the design of T-beams, some distinctions appear.

Designs

The T-beam, though simple in design, contains multiple design elements of interest. Unlike an I-beam, a T-beam lacks a bottom flange, which carries savings in terms of materials, but at the loss of resistance to tensile forces.[5] In parking garages, however, it is obvious that this lack of a bottom flange on a T-beam actually serves as an advantage in that the stem rests on shelf making the flange the upper deck. T- beam designs come in many sizes, lengths and widths depending on what the structure is and its compression tension needs. However, the simplicity of the T-beam is in question by some who would rightly test more than one complex structure; for example, a group of researchers tested pretension inverted T-beams with circular web openings,[6] with mixed but generally favorable results. Thus, in some cases, the extra time and effort invested in creating a more complex structure proves worthwhile. A simpler matter to consider is that of which material or materials make up the construction of T-beams.

Materials

Steel T-beams

Steel T-beams manufacturing process includes: hot rolling, extrusion, plate welding and pressure fitting. A process of large rollers connecting two steel plates by pinching them together called pressure fitting is a common process for non-load bearing beams. The reality is that for most roadways and bridges today, it is more practical to bring concrete into the design as well. Most T-beam construction is not with steel or concrete alone, but rather with the composite of the two, namely, reinforced concrete.[7] Though the term could refer to any one of a number of means of reinforcement, generally, the definition is limited to concrete poured around rebar. This shows that in considering materials available for a task, engineers need to consider the possibility that no one single material is adequate for the job; rather, combining multiple materials together may be the best solution. Thus, steel and concrete together can prove ideal.

Reinforced concrete T-beams[edit]

Concrete alone is brittle and thus overly subject to the shear stresses a T-beam faces where the web and flange meet. This is the reason that steel is combined with concrete in T-beams. A problem of shear stress can lead to failures of flanges detaching from webs when under load.[8] This could prove catastrophic if allowed to occur in real life; hence, the very real need to mitigate that possibility with reinforcement for concrete T-beams. In such composite structures, many questions arise as to the particulars of the design, including what the ideal distribution of concrete and steel might be: "To evaluate an objective function, a ratio of steel to concrete costs is necessary".[9] This demonstrates that for all aspects of the design of composite T-beams, equations are made only if one has adequate information. Still, there are aspects of design that some may not even have considered, such as the possibility of using external fabric-based reinforcement, as described by Chajes et al., who say of their tested beams, "All the beams failed in shear and those with composite reinforcement displayed excellent bond characteristics. For the beams with external reinforcement, increases in ultimate strength of 60 to 150 percent were achieved".[4] When it comes to resistance to shear forces, external reinforcement is a valid option to consider. Thus, overall, the multiple important aspects of T-beam design impress themselves upon the student of engineering.

Issues

An issue with the T-beam compared to the I-beam is the lack of the bottom flange. In addition, this makes the beam not as versatile because of the weaker side not having the flange making it have less tensile strength.

Concrete beams are often poured integrally with the slab, forming a much stronger "T" – shaped beam. These beams are very efficient because the slab portion carries the compressive loads and the reinforcing bars placed at the bottom of the stem carry the tension. A T-beam typically has a narrower stem than an ordinary rectangular beam. These stems are typically spaced from 4'-0" apart to more than 12'-0". The slab portion above the stem is designed as a one-way slab spanning between stems.

Double-T beams

Main article: Double tee

A double-T beam or double tee beam is a load-bearing structure that resemble two T-beams connected to each other. Double tees are manufactured from prestressed concrete using pretensioning beds of about 200-foot (61 m) to 500-foot (150 m) long. The strong bond of the flange (horizontal section) and the two webs (vertical members) creates a structure that is capable of withstanding high loads while having a long span. The typical sizes of double tees are up to 15 feet (4.6 m) for flange width, up to 5 feet (1.5 m) for web depth and up to 80 feet (24 m) or more for span length.[10]

Basic concept

Bridge is life line of road network, both in urban and rural areas. With rapid technology growth the conventional bridge has been replaced by innovative cost effective structural system. One of these solution present two structural RCC systems that are T-Beam Girder and box Girder Bridge.

Box girders, have gained wide acceptance in freeway and bridge systems due to their structural efficiency, better stability, serviceability, economy of construction and pleasing aesthetics. Box girder design is more complicated as structure is more complex as well as needed sophisticated from work. In the place of Box Girder if we talk about T-Beam Girder geometry is simple and does not have sophisticated in construction.

Bridge design is an important as well as complex approach of structural engineer. As in case of bridge design, span length and live load are always important factor. These factors affect the conceptualization stage of design. The effect of live load for various span are varied. In shorter spans track load govern whereas on larger span wheel load govern. Selection of structural system for span is always a scope for research. Structure systems adopted are influence by factor like economy and complexity in construction. The 25 m span as selected for this study, these two factor are important aspects. In 25 m span, codal provision allows as to choose two structural systems i.e. T-Beam Girder and Box Girder. This study investigates these two structural systems for span 25 m and detail design has been carried out with IRC loadings. The choice of economical and constructible structural system is depending on the result.

II. LITERATURE REVIEW

Design of difference structural system for different span has been the subject of considerable, experimental and analytical research.

A important research has been published for Box Girder Bridge by Chu, K. H. (1971) [1] analysed simply supported curved box girder bridges by using finite element method. Schlaich, J. (1982), [2] describe the Concrete Box-Girder Bridges. Sami M. Fereig (1994), [3] has been carried out a Preliminary design of precast prestressed Concrete Box Girder Bridge. M. Qaqish (2008),[4] presents the analysis of two continuous spans Box Girder Bridge. The first method based on one dimensional model according to AASHTO specifications 2002 and the second method is based on three dimensional finite element analyses. M. Qaqish (2008), presents the Comparison between Computed Bending Moments by AASHTO Specifications and Finite Element Method of Two Continuous Spans of Voided Slab Bridge. GokhanPekcan (2008), presents Seismic Response of Skewed RC Box-Girder Bridges.

Many methods are used in designing T-Beam Girder Bridge such as AASHTO specifications, grillage and finite element methods. Chan and O’Connor (1990 a),[5] describe further field studies on the bridge referred to above and reported values for the impact fraction I, consistent with the values obtained previously. In a companion paper, the same authors Chan and O’Connor (1990 b), [6] present a vehicle model in which each axle load includes a dynamic load component that varies sinusoidally at the first natural frequency of the bridge. Wang and Huang (1992), [7] studied the dynamic and impact characteristics of continuous

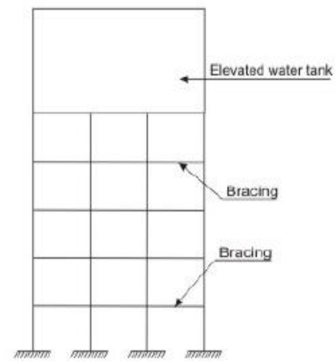
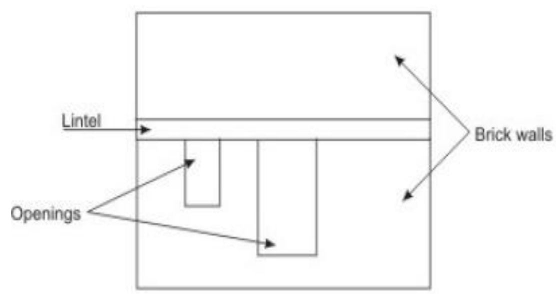


Fig. 5.10.1(a): Bracings of elevated water tank



g. 5.10.1 (b)

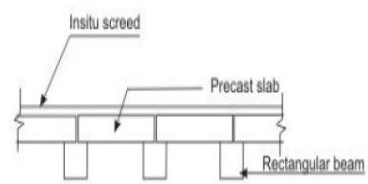


Fig. 5.10.1(c) Precast slab on rectangular beams

Reinforced concrete slabs used in floors, roofs and decks are mostly cast monolithic from the bottom of the beam to the top of the slab. Such rectangular beams having slab on top are different from others having either no slab (bracings of elevated tanks, lintels etc.) or having disconnected slabs as in some pre-cast systems (Figs. 5.10.1 a, b and c). Due to monolithic casting, beams and a part of the slab act together. Under the action of positive bending moment, i.e., between the supports of a continuous beam, the slab, up to a certain width greater than the width of the beam, forms the top part of the beam. Such beams having slab on top of the rectangular rib are designated as the flanged beams - either *T* or *L* type depending on whether the slab is on both sides or on one side of the beam (Figs. 5.10.2 a to e). Over the supports of a continuous beam, the bending moment is negative and the slab, therefore, is in tension while a part of the rectangular beam (rib) is in compression. The continuous beam at support is thus equivalent to a rectangular beam (Figs. 5.10.2 a, c, f and g).

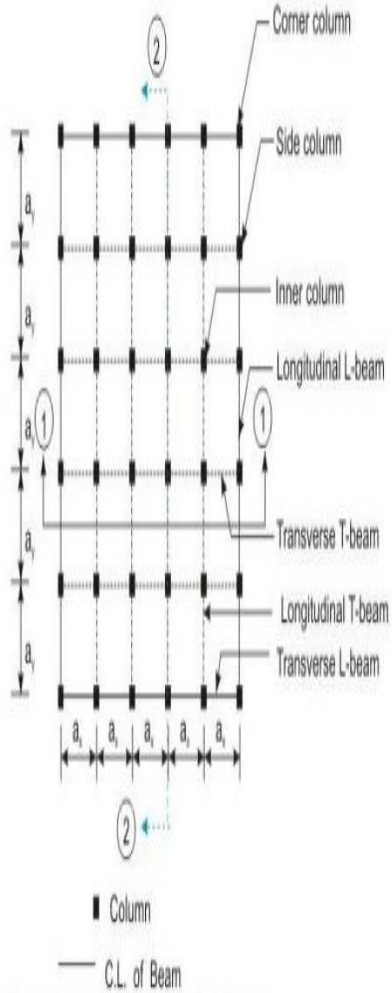


Fig. 5.10.2 (a): Key plan

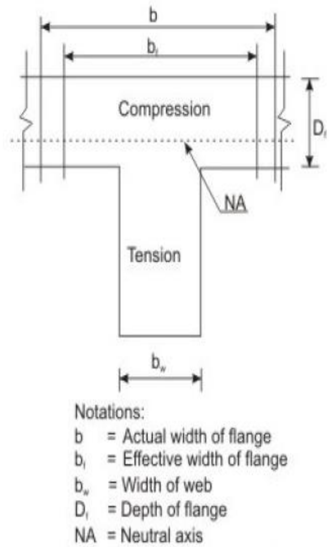


Fig.5.10.2 (e): Detail at 4 (T-beam)

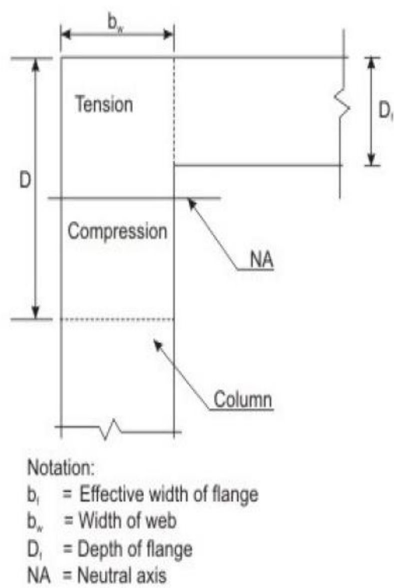


Fig. 5.10.2 (f): Detail at 5 (rectangular beam)

The actual width of the flange is the spacing of the beam, which is the same as the distance between the middle points of the adjacent spans of the slab, as shown in Fig. 5.10.2 b. However, in a flanged beam, a part of the width less than the actual width, is effective to be considered as a part of the beam. This width of the slab is designated as the effective width of the flange.

Effective width

IS code requirements

The following requirements (cl. 23.1.1 of IS 456) are to be satisfied to ensure the combined action of the part of the slab and the rib (rectangular part of the beam).

(a) The slab and the rectangular beam shall be cast integrally or they shall be effectively bonded in any other manner.

(b) Slabs must be provided with the transverse reinforcement of at least 60 per cent of the main reinforcement at the mid span of the slab if the main reinforcement of the slab is parallel to the transverse beam (Figs. 5.10.3 a and b).

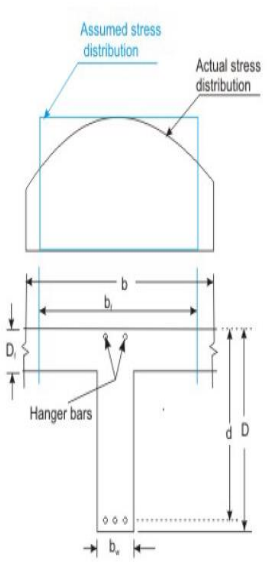


Fig. 5.10.4: Variation of compressive stress

The variation of compressive stress (Fig. 5.10.4) along the actual width of the flange shows that the compressive stress is more in the flange just above the rib than the same at some distance away from it. The nature of variation is complex and, therefore, the concept of effective width has been introduced. The effective width is a convenient hypothetical width of the flange over which the compressive stress is assumed to be uniform to give the same compressive.

force as it would have been in case of the actual width with the true variation of compressive stress.

5.10.2.2 IS code specifications

Clause 23.1.2 of IS 456 specifies the following effective widths of *T* and *L*-beams:

(a) For *T*-beams, the lesser of

(i) $b_f = l_o/6 + b_w + 6\,D_f$

(ii) b_f = Actual width of the flange

(b) For isolated *T*-beams, the lesser of

(ii) b_f = Actual width of the flange

(c) For *L*-beams, the lesser of

(i) $b_f = l_o/12 + b_w + 3\,D_f$

(ii) b_f = Actual width of the flange

(d) For isolated *L*-beams, the lesser of

(ii) b_f = Actual width of the flange

where b_f = effective width of the flange,

l_o = distance between points of zero moments in the beam, which is the effective span for simply supported beams and 0.7 times the effective span for continuous beams and frames,

b_w = beadth of the web,

D_f = thickness of the flange,

and b = actual width of the flange.

Four Different Cases

The neutral axis of a flanged beam may be either in the flange or in the web depending on the physical dimensions of the effective width of flange b_f, effective width of web b_w, thickness of flange D_f and effective depth of flanged beam *d* (Fig. 5.10.4). The flanged beam may be considered as a rectangular beam of width b_f and effective depth *d* if the neutral axis is in the flange as the concrete in tension is ignored. However, if the neutral axis is in the web, the compression is taken by the flange and a part of the web.

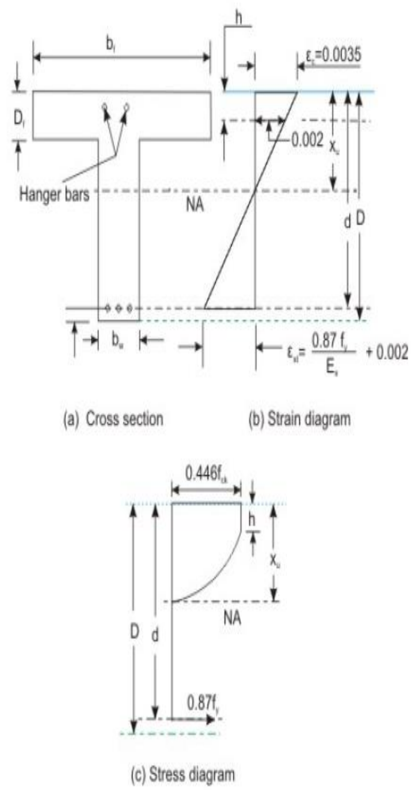


Fig. 5.10.5: A typical T-beam section

All the assumptions made in sec. 3.4.2 of Lesson 4 are also applicable for the flanged beams. As explained in Lesson 4, the compressive stress remains constant between the strains of 0.002 and 0.0035. It is important to find the depth *h* of the beam where the strain is 0.002 (Fig. 5.10.5 b). If it is located in the web, the whole of flange will be under the constant stress level of 0.446 f_{ck}. The following gives the relation of D_f and *d* to facilitate the determination of the depth *h* where the strain will be 0.002.

From the strain diagram of Fig. 5.10.5 b:

$$\frac{0.002}{0.0035} = \frac{x_u - h}{x_u}$$

or $\quad \dfrac{h}{x_u} = \dfrac{3}{7} = 0.43$

(5.1)

when $x_u = x_{u,\max}$, we get

$h = \dfrac{3}{7} x_{u,\max} = 0.227\,d,\ 0.205\,d$ and $0.197\,d$, for Fe 250, Fe 415 and Fe 500, respectively. In general, we can adopt, say

$$h/d = 0.2 \tag{5.2}$$

The same relation is obtained below from the values of strains of concrete and steel of Fig. 5.10.5 b.

$$\frac{\varepsilon_{st}}{\varepsilon_c} = \frac{d - x_u}{x_u}$$

or
$$\frac{d}{x_u} = \frac{\varepsilon_{st} + \varepsilon_c}{\varepsilon_c} \tag{5.3}$$

Dividing Eq. 5.1 by Eq. 5.3

$$\frac{h}{d} = \frac{0.0015}{\varepsilon_{st} + 0.0035} \tag{5.4}$$

Using $\varepsilon_{st} = (0.87\, f_y / E_s) + 0.002$ in Eq. 5.4, we get *h/d* = 0.227, 0.205 and 0.197 for Fe 250, Fe 415 and Fe 500 respectively, and we can adopt *h/d* = 0.2 (as in Eq. 5.2).

Thus, we get the same Eq. 5.2 from Eq. 5.4,

$$h/d = 0.2 \tag{5.2}$$

It is now clear that the three values of *h* are around 0.2 *d* for the three grades of steel. The maximum value of *h* may be D_f, at the bottom of the flange where the strain will be 0.002, if D_f/d = 0.2. This reveals that the thickness of the flange may be considered small if D_f/d does not exceed 0.2 and in that case, the position of the fibre of 0.002 strain will be in the web and the entire flange will be under a constant compressive stress of 0.446 f_{ck}.

On the other hand, if D_f is > 0.2 *d*, the position of the fibre of 0.002 strain will be in the flange. In that case, a part of the slab will have the constant stress of 0.446 f_{ck} where the strain will be more than 0.002.

Thus, in the balanced and over-reinforced flanged beams (when), the ratio of *D*max , *uuxx*= *f* /*d* is important to determine if the rectangular stress block is for the full depth of the flange (when D_f/d does not exceed 0.2) of for a part of the flange (when D_f/d > 0.2). Similarly, for the under-reinforced flanged beams, the ratio of D_f/x_u is considered in place of D_f/d. If D_f/x_u does not exceed 0.43 (see Eq. 5.1), the constant stress block is for the full depth of the flange. If D_f/x_u > 0.43, the constant stress block is for a part of the depth of the flange.

Based on the above discussion, the four cases of flanged beams are as follows:

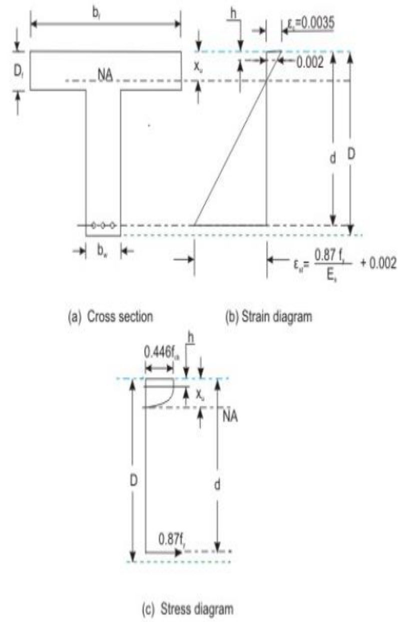


Fig. 5.10.6: T-beam, case (i), when $x_u < D_f$

(i) Neutral axis is in the flange ($x_u < D_f$), (Fig. 5.10.6 a to c)

(ii) Neutral axis is in the web and the section is balanced ($x_u = x_{u,max} > D_f$), (Figs. 5.10.7 and 8 a to e)

It has two situations: (a) when D_f/d does not exceed 0.2, the constant stress block is for the entire depth of the flange (Fig. 5.10.7), and (b) when D_f/d > 0.2, the constant stress block is for a part of the depth of flange (Fig. 5.10.8).

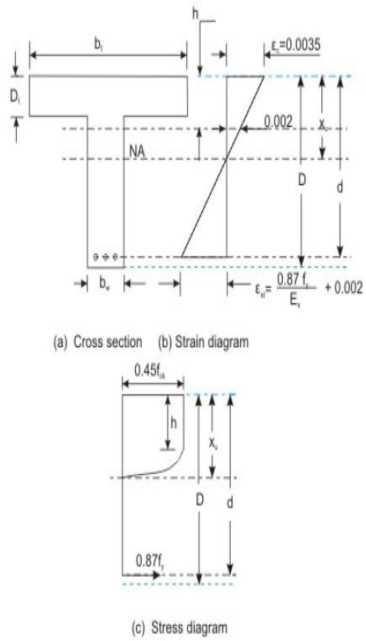


Fig. 5.10.9: T-beam, case (iii a), when D_f/x_u <= 0.43 and under-reinforced $x_u > D_f$

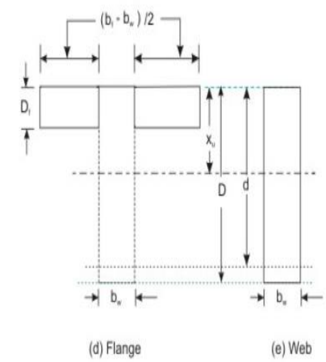


Fig. 5.10.9: T-beam, case (iii a), when D_f/x_u <= 0.43 and under-reinforced $x_u > D_f$

(iii) Neutral axis is in the web and the section is under-reinforced ($x_{u,max} > x_u > D_f$), (Figs. 5.10.9 and 10 a to e)

This has two situations: (a) when D_f/x_u does not exceed 0.43, the full depth of flange is having the constant stress (Fig. 5.10.9), and (b) when D_f/x_u > 0.43, the constant stress is for a part of the depth of flange (Fig. 5.10.10).

(iv) Neutral axis is in the web and the section is over-reinforced ($x_u > x_{u,max} > D_f$), (Figs. 5.10.7 and 8 a to e)

As mentioned earlier, the value of x_u is then taken as $x_{u,max}$ when $x_u > x_{u,max}$. Therefore, this case also will have two situations depending on D_f /d not exceeding 0.2 or > 0.2 as in (ii) above. The governing equations of the four different cases are now taken up.

5.10.4 Governing Equations

The following equations are only for the singly reinforced *T*-beams. Additional terms involving $M_{u,lim}$, M_{u2}, $A_{sc,}$ A_{st1} and A_{st2} are to be included from Eqs. 4.1 to 4.8 of sec. 4.8.3 of Lesson 8 depending on the particular case. Applications of these terms are explained through the solutions of numerical problems of doubly reinforced *T*-beams in Lessons 11 and 12.

5.10.4.1 Case (i): When the neutral axis is in the flange ($x_u < D_f$), (Figs. 5.10.6 a to c)

Concrete below the neutral axis is in tension and is ignored. The steel reinforcement takes the tensile force (Fig. 5.10.6). Therefore, *T* and *L*-beams are considered as rectangular beams of width b_f and effective depth *d*. All the equations of singly and doubly reinforced rectangular beams derived in Lessons 4 to 5 and 8 respectively, are also applicable here.

5.10.4.2 Case (ii): When the neutral axis is in the web and the section is balanced ($x_{u,max} > D_f$), (Figs. 5.10.7 and 8 a to e)

(a) When D_f /d does not exceed 0.2, (Figs. 5.10.7 a to e)

As explained in sec. 5.10.3, the depth of the rectangular portion of the stress block (of constant stress = 0.446 f_{ck}) in this case is greater than D_f (Figs. 5.10.7 a, b and c). The section is split into two parts: (i) rectangular web of width b_w and effective depth *d*, and (ii) flange of width (b_f - b_w) and depth D_f (Figs. 5.10.7 d and e).

Total compressive force = Compressive force of rectangular beam of width b_w and depth *d* + Compressive force of rectangular flange of width (b_f - b_w) and depth D_f.

Thus, total compressive force $C = 0.36 f_{ck} b_w x_{u,\,max} + 0.45 f_{ck}(b_f - b_w) D_f$ (5.5)

(Assuming the constant stress of concrete in the flange as 0.45 f_{ck} in place of 0.446 f_{ck}, as per G-2.2 of IS 456), and the tensile force

$T = 0.87 f_y A_{st}$ (5.6)

The lever arm of the rectangular beam (web part) is (d - 0.42 $x_{u,\,max}$) and the same for the flanged part is (d - 0.5 D_f).

So, the total moment = Moment due to rectangular web part + Moment due to rectangular flange part

or $M_u = 0.36 f_{ck} b_w x_{u,\,max} (d - 0.42 x_{u,\,max}) + 0.45 f_{ck}(b_f - b_w) D_f(d - D_f/2)$

or $M_u = 0.36(x_{u,\,max}/d)\{1 - 0.42(x_{u,\,max}/d)\} f_{ck} b_w d^2 + 0.45 f_{ck}(b_f - b_w) D_f(d - D_f/2)$

(5.7)

Equation 5.7 is given in G-2.2 of IS 456.

(b) When D_f /d >0.2, (Figs. 5.10.8 a to e)

In this case, the depth of rectangular portion of stress block is within the flange (Figs. 5.10.8 a, b and c). It is assumed that this depth of constant stress (0.45 f_{ck}) is y_f, where

$y_f = 0.15 x_{u,\,max} + 0.65 D_f$, but not greater than D_f (5.8)

The above expression of y_f is derived in sec. 5.10.4.5.

As in the previous case (ii a), when D_f /d does not exceed 0.2, equations of *C*, *T* and M_u are obtained from Eqs. 5.5, 6 and 7 by changing D_f to y_f. Thus, we have (Figs. 5.10.8 d and e)

$$C = 0.36 f_{ck} b_w x_{u,\,max} + 0.45 f_{ck}(b_f - b_w) y_f$$

(5.9)

$T = 0.87 f_y A_{st}$ (5.10)

The lever arm of the rectangular beam (web part) is (d - 0.42 $x_{u,\,max}$) and the same for the flange part is (d - 0.5 y_f). Accordingly, the expression of M_u is as follows:

$M_u = 0.36(x_{u,\,max}/d)\{1 - 0.42(x_{u,\,max}/d)\} f_{ck} b_w d^2 + 0.45 f_{ck}(b_f - b_w) y_f(d - y_f/2)$

(5.11)

5.10.4.3 Case (iii): When the neutral axis is in the web and the section is under-reinforced ($x_u > D_f$), (Figs. 5.10.9 and 10 a to e)

(a) When D_f / x_u does not exceed 0.43, (Figs. 5.10.9 a to e)

Since D_f does not exceed 0.43 x_u and h (depth of fibre where the strain is 0.002) is at a depth of 0.43 x_u, the entire flange will be under a constant stress of 0.45 f_{ck} (Figs. 5.10.9 a, b and c). The equations of C, T and M_u can be written in the same manner as in sec. 5.10.4.2, case (ii a). The final forms of the equations are obtained from Eqs. 5.5, 6 and 7 by replacing $x_{u,\,max}$ by x_u. Thus, we have (Figs. 5.10.9 d and e)

$$C = 0.36\, f_{ck}\, b_w\, x_u + 0.45\, f_{ck} (b_f - b_w)\, D_f \quad (5.12)$$

$$T = 0.87\, f_y\, A_{st} \quad (5.13)$$

$$M_u = 0.36(x_u/d)\{1 - 0.42(x_u/d)\}\, f_{ck}\, b_w\, d^2 + 0.45\, f_{ck} (b_f - b_w)\, D_f (d - D_f/2) \quad (5.14)$$

(b) When $D_f / x_u > 0.43$, (Figs. 5.10.10 a to e)

Since $D_f > 0.43\, x_u$ and h (depth of fibre where the strain is 0.002) is at a depth of 0.43 x_u, the part of the flange having the constant stress of 0.45 f_{ck} is assumed as y_f (Fig. 5.10.10 a, b and c). The expressions of y_f, C, T and M_u can be written from Eqs. 5.8, 9, 10 and 11 of sec. 5.10.4.2, case (ii b), by replacing $x_{u,max}$ by x_u. Thus, we have (Fig. 5.10.10 d and e)

$$y_f = 0.15\, x_u + 0.65\, D_f, \text{ but not greater than } D_f \quad (5.15)$$

$$C = 0.36\, f_{ck}\, b_w\, x_u + 0.45\, f_{ck} (b_f - b_w)\, y_f \quad (5.16)$$

$$T = 0.87\, f_y\, A_{st} \quad (5.17)$$

$$M_u = 0.36(x_u/d)\{1 - 0.42(x_u/d)\}\, f_{ck}\, b_w\, d^2 + 0.45\, f_{ck} (b_f - b_w)\, y_f (d - y_f/2) \quad (5.18)$$

ARRANGEMENT OF LIVE LOAD

22.4.1 a) Consideration may be limited to combinations of:

1) Design dead load on all spans with full design live load on two adjacent spans; and

2) Design dead load on all spans with dull design live load on alternate spans.

22.4.1 b) When design live load does not exceed three-fourths of the design dead load, the load arrangement may be design dead load and design live load on all the spans.

Note: For beams continuous over support 22.4.1 (a) may be assumed.

22.4.2 **Substitute Frame:** For determining the moments and shears at any floor or roof level due to gravity loads, the beams at that level together with columns above and below with their far ends fixed may be considered to constitute the frame.

22.4.3 For lateral loads, simplified methods may be used to obtain the moments and shears for structures that are symmetrical. For unsymmetrical or very tall structures, more rigorous methods should be used.

MOMENT AND SHEAR COFFICIENTS FOR CONTINUOUS BEAMS

Unless more exact estimates are made, for beams of uniform cross-section which support substantially uniformly distributed load over three or more spans which do not differ by more than 15 percent of the longest, the bending moments and shear forces used in design may be obtained using the coefficients given in Tables below.

For moments at supports where two unequal spans meet or in case where the spans are not equally loaded, the average of the two values for the negative moment at the support may be taken for design.

Where coefficients given in Table below are used for calculation of bending moments, redistribution referred to in 22.7 shall not be permitted.

22.5.2 **BEAMS OVER FREE END SUPPORTS**

Where a member is built into a masonry wall which develops only partial restraint, the member shall be designed to resist a negative moment at the face of the support of W1/24 where W is the total design load and 1 is the effective span, or such other restraining moment as may be shown to be applicable. For such a condition shear coefficient given in Table below at the end support may be increased by 0.05.

BENDING MOMENT COFFICIENTS

	Span Moments		Support Moments	
Types of Load	Near Middle of End Span	At Middle of interior span	At Support next to the end support	At Other Interior Supports
Dead load and imposed load (fixed)	$+\frac{1}{12}$	$+\frac{1}{16}$	$(-)\frac{1}{10}$	$(-)\frac{1}{12}$
Imposed load (not fixed)	$+\frac{1}{10}$	$+\frac{1}{12}$	$(-)\frac{1}{9}$	$(-)\frac{1}{9}$

Note: For obtaining the bending moment, the coefficient shall be multiplied by the total design load and effective span.

CRITICAL SECTIONS FOR MOMENT AND SHEAR

22.6.1 For monolithic construction, the moments computed at the face of the supports shall be used in the design of the members at those sections. For non-monolithic construction the design of the member shall be done keeping in view 22.2.

22.6.2 **Critical Section for Shear**

The shears computed at the face of the Support shall be used in the design of the member at that section except as in 22.6.2.1

22.6.2.1 When the reaction in the direction of the applied shear introduces compression into the end region of the member, sections located at a distance less than d from the face of the support may be designed for the same shear as that computed at distance d.

REDISTRIBUTION OF MOMENTS

22.7 Redistribution of moments may be done in accordance with 37.1.1 for limit state method and in accordance with B-1.2 for working stress method. However, where simplified analysis using coefficients is adopted, redistribution of moments shall not be done.

EFFECTIVE DEPTH

23.0 Effective depth of a beam is the distance between the centroid of the area of tension reinforcement and the maximum compression fibre, excluding the thickness of finishing material not placed monolithically with the member and the thickness of any concrete provided to allow for wear. This will not apply to deep beams.

CONTROL OF DEFLECTION

23.2 The deflection of a structure or part thereof shall not adversely affect the appearance or efficiency of the structure or finishes or partitions. The deflection shall generally be limited to the following:

a) The final deflection due to all loads including the effects of temperature, creep and shrinkage and measured from the as-cast level of the supports of floors, roofs and all other horizontal members, should not normally exceed span/250.

b) The deflection including the effects of temperature, creep and shrinkage occurring after erection of partitions and the application of finishes should not normally exceed span/350 or 20mm whichever is less.

23.2.1 For beams, the vertical deflection limits may generally be assumed to be satisfied provided that the span to depth ratio are not greater than the value obtained as below:

a) Basic values of span to effective depth ratios for spans up to 10m:

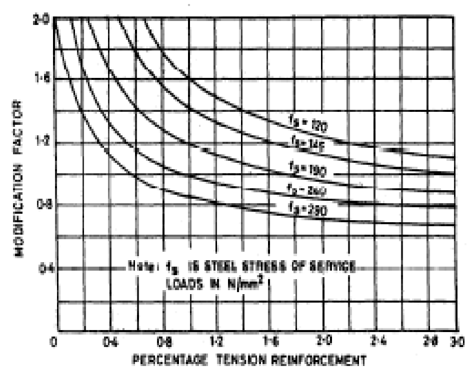
Cantilever	7
Simply supported	20
Continuous	26

b) For spans above 10m, the values in (a) may be multiplied by 10/span in metres, except for cantilever in which case deflection calculations should be made.

c) Depending on the area and the type of steel for tension reinforcement, the value in (a) or (b) shall be modified as per Fig. 4

d) Depending on the area of compression reinforcement, the value of span to depth ratio be further modified as per Fig. 5

e) For flanged beams, the value of (a) or (b) be modified as per Fig. 6 and the reinforcement percentage for use in fig. 4 and 5 should be based on area of section equal to bf d.



$f_s = 0.58 f_y \dfrac{\text{Area of cross-section of steel required}}{\text{Area of cross-section of steel provided}}$

Fig. 4 Modification Factor for Tension Reinforcement

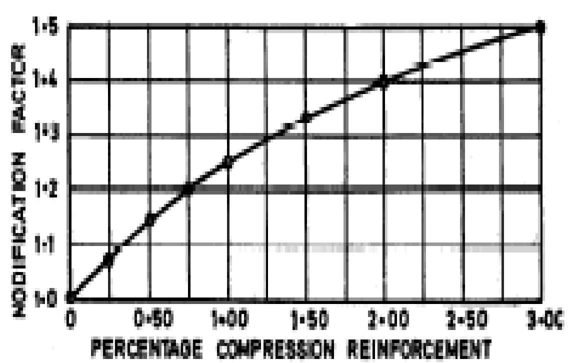


Fig. 5 Modification Factor for Compression Reinforcement

26.5 REQUIREMENT OF REINFORCEMENT FOR STRUCTURAL MEMBER

26.5.1 Beams

26.5.1.1 Tension reinforcement

(a) **Minimum reinforcement**:- The minimum area of tension reinforcement shall not be less than that given by the following:-

$$\frac{A_s}{bd} = \frac{0.85}{f_y}$$

where

As = minimum area of tension reinforcement.

b = breadth of beam or the breadth of the web of T-beam.

d = effective depth, and

fy = characteristic strength of reinforcement in M/mm2

(b) **Maximum reinforcement**:- the maximum area of tension reinforcement shall not exceed 0.04bD.

26.5.1.2 Compression reinforcement

The maximum area of comparison reinforcement shall not exceed 0.04 bd. Comparison reinforcement in beams shall be enclosed by stirrups for effective lateral restraint.

26.5.1.3 Side face reinforcement

Where the depth of the web in the beam exceeds 750mm, side face reinforcement shall be provided along the two faces. The total area of such reinforcement shall be not less than 0.1 % of the web area and shall be distributed on the equally on the two face at spacing not exceeding 300mm or web thickness whichever is less.

26.5.1.4 Transverse reinforcement in beam for shear torsion

The transverse reinforcement in beam shall be taken around the outer most tension & compression bars. In T-beams and I-beams, such reinforcement shall pass around longitudinal bars located close to the outer face of the flange.

26.5.1.5 Maximum spacing of shear reinforcement

Maximum spacing of shear reinforcement means long by axis of the member shall not exceed 0.75 d for vertical stirrups and d for inclined stirrups at 45" where d is the effective depth on the section under consideration. In no case shall be spacing exceed 300mm.

26.5.1.6 Minimum shear reinforcement

Minimum shear reinforcement in the form of stirrups shall be provided such that:

$$\frac{A_{sv}}{bs_v} > \frac{0.4}{0.87\, f_y}$$

Where

Asv = total cross-sectional area of stirrups legs effective in shear.

Sv = stirrups spacing along the length of the member

B = breadth of the beam or breadth of the web of flange beam, and

fy = characteristic strength of the stirrups reinforcement in N/mm2 which shall not taken greater than 415 N/mm2

Where the maximum shear stress calculated is less than half the permissible value in member of minor structure importance such as lintels, this provision need not to be complied with.

26.5.1.7 Distribution of torsion reinforcement

When a member is designed for torsion torsion reinforcement shall be provided as below:

a) the transverse reinforcement for torsion shall be rectangular closed stirrups placed perpendicular to the axis of the member. The spacing of the stirrups shall not exceed the list of x1, x1+y1/4 and 300 mm, where x1, y1 are respectively the short & long dimensions of the stirrup.

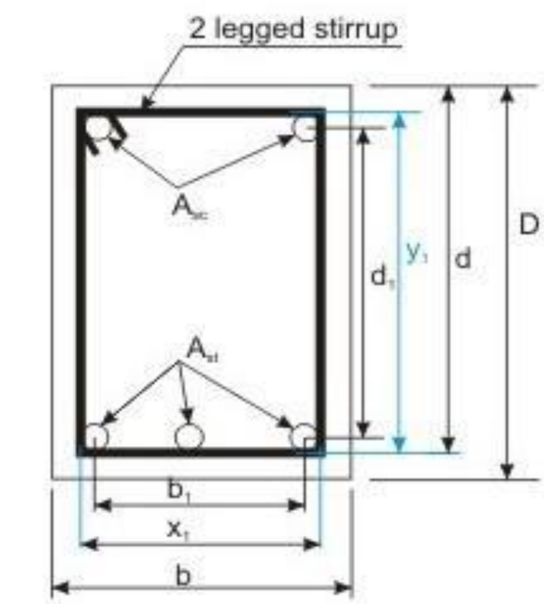


Fig. 6.16.2: Stirrups in beams

b) Longitudinal reinforcement shall be place as closed as is practicable to the corner of the cross section & in all cases, there shall be atleast one longitudinal bar in each corner of the ties. When the cross sectional dimension of the member exceed 450 mm additional longitudinal bar shall be provided

5.10.4.4 Case (iv): When the neutral axis is in the web and the section is over-reinforced ($x_u > D_f$), (Figs. 5.10.7 and 8 a to e)

For the over-reinforced beam, the depth of neutral axis x_u is more than $x_{u,\,max}$ as in rectangular beams. However, x_u is restricted up to $x_{u,max}$. Therefore, the corresponding expressions of *C*, *T* and M_u for the two situations (a) when D_f / *d* does not exceed 0.2 and (b) when D_f / *d* > 0.2 are written from Eqs. 5.5 to 5.7 and 5.9 to 5.11, respectively of sec. 5.10.4.2 (Figs. 5.10.7 and 8). The expression of y_f for (b) is the same as that of Eq. 5.8.

(a) When D_f/d does not exceed 0.2 (Figs. 5.10.7 a to e)

The equations are:

$C = 0.36\, f_{ck}\, b_w\, x_{u,\,max} + 0.45\, f_{ck}(b_f - b_w)\, D_f$ (5.5)

$T = 0.87\, f_y\, A_{st}$ (5.6)

$M_u = 0.36(x_{u,\,max}/d)\{1 - 0.42(x_{u,\,max}/d)\}\, f_{ck}\, b_w\, d^2 + 0.45\, f_{ck}(b_f - b_w)\, D_f(d - D_f/2)$

(5.7)

(b) When D_f/d >0.2 (Figs. 5.10.8 a to e)

$y_f = 0.15\, x_{u,\,max} + 0.65\, D_f$, but not greater than D_f (5.8)

$C = 0.36\, f_{ck}\, b_w\, x_{u,\,max} + 0.45\, f_{ck}(b_f - b_w)\, y_f$

(5.9)

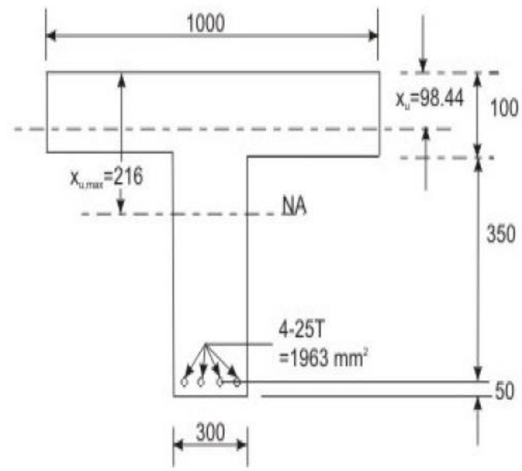
$T = 0.87\, f_y\, A_{st}$ (5.10)

$M_u = 0.36(x_{u,\,max}/d)\{1 - 0.42(x_{u,\,max}/d)\}\, f_{ck}\, b_w\, d^2 + 0.45\, f_{ck}(b_f - b_w)\, y_f(d - y_f/2)$

(5.11)

It is clear from the above that the over-reinforced beam will not have additional moment of resistance beyond that of the balanced one. Moreover, it will prevent steel failure. It is, therefore, recommended either to re-design or to go for doubly reinforced flanged beam than designing over-reinforced flanged beam.

Designing of the overhanging T-beam



b_f = 1000 mm, D_f = 100 mm, b_w = 300 mm, cover = 50 mm, *d* = 450 mm and A_{st} = 1963 mm^2 (4- 25 T). Use M 20 and Fe 415.

Step 1: To determine the depth of the neutral axis x_u

Assuming x_u in the flange and equating total compressive and tensile forces from the expressions of *C* and *T* (Eq. 3.16 of Lesson 5) as the *T*-beam can be treated as rectangular beam of width b_f and effective depth *d*, we get:

$$x_u = \frac{0.87\, f_y\, A_{st}}{0.36\, b_f\, f_{ck}} = \frac{0.87\,(415)\,(1963)}{0.36\,(1000)\,(20)} = 98.44 \text{ mm} < 100 \text{ mm}$$

So, the assumption of x_u in the flange is correct.

$x_{u,\,max}$ for the balanced rectangular beam = 0.48 *d* = 0.48 (450) = 216 mm.

It is under-reinforced since $x_u < x_{u,max}$.

Step 2: To determine *C*, *T* and M_u

From Eqs. 3.9 (using $b = b_f$) and 3.14 of Lesson 4 for *C* and *T* and Eq. 3.23 of Lesson 5 for M_u, we have:

$C = 0.36\, b_f\, x_u\, f_{ck}$ (3.9)

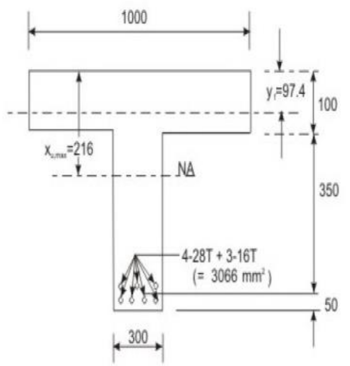
= 0.36 (1000) (98.44) (20) = 708.77 kN

$T = 0.87\, f_y\, A_{st}$ (3.14)

= 0.87 (415) (1963) = 708.74 kN

$$M_u = 0.87 f_y A_{st} d \left(1 - \frac{A_{st} f_y}{f_{ck} b_f d}\right) \qquad (3.23)$$

$$= 0.87 (415) (1963) (450) \left\{1 - \frac{(1963)(415)}{(20)(1000)(450)}\right\} = 290.06 \text{ kNm}$$



Step 3: Determination of $x_{u,lim}$ for singly reinforced flanged beam

Here, D_f/d = 120/600 = 0.2, so y_f is not needed.

$$M_{u,lim} = 0.36\,(x_{u,max}/d)\,\{1 - 0.42\,(x_{u,max}/d)\}\, f_{ck}\, b_w\, d^2 + 0.45\, f_{ck}\,(b_f - b_w)\, D_f\,(d - D_f/2)$$

= 0.36(0.46) {1 – 0.42(0.46)} (30) (300) (600) (600)
+ 0.45(30) (900) (120) (540)
= 1,220.20 kNm

$$A_{st,lim} = \frac{M_{u,lim}}{0.87 f_y\, d\, \{1 - 0.42\,(x_{u,max}/d)\}}$$

$$= \frac{(1220.20)(10^6)}{(0.87)(500)(600)(0.8068)} = 5{,}794.6$$

Step 4: Determination of M_{u2}

Total A_{st} = 6,509 mm², $A_{st,lim}$ = 5,794.62 mm²

A_{st2} = 714.38 mm² and A_{sc} = 1,030 mm²

It is important to find out how much of the total A_{sc} and A_{st2} are required effectively. From the equilibrium of C and T forces due to additional steel (compressive and tensile), we have:

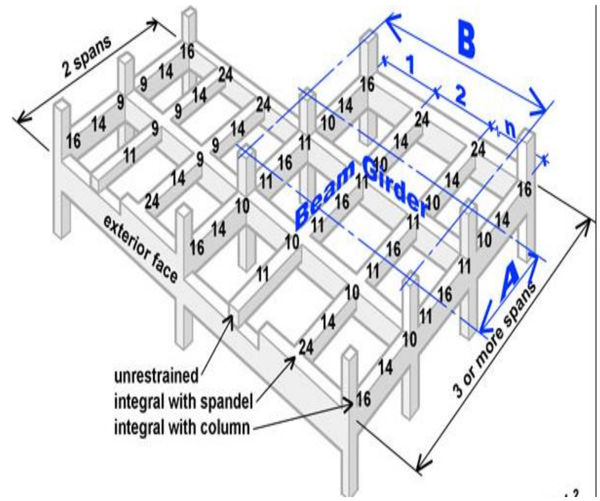
$(A_{st2})\,(0.87)\,(f_y) = (A_{sc})\,(f_{sc})$

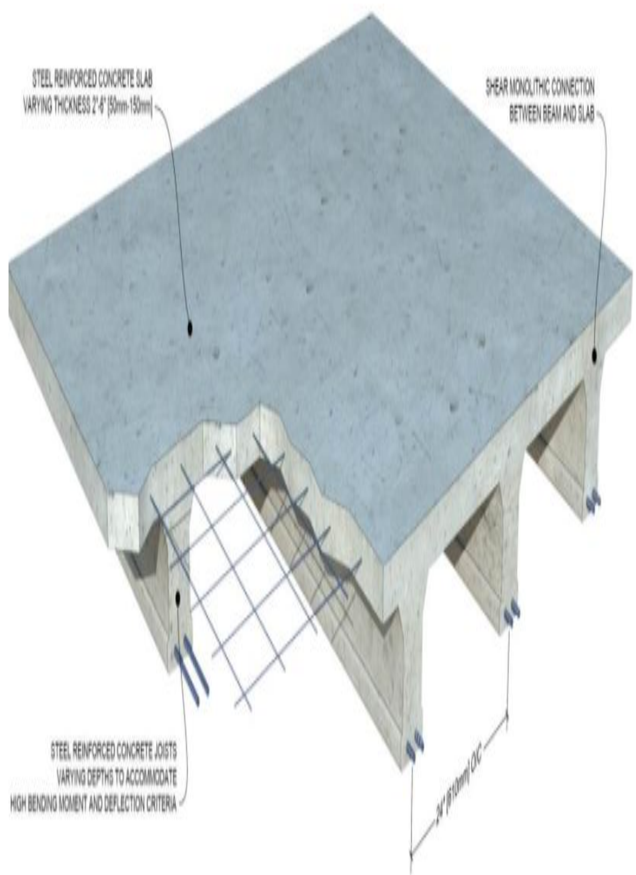
If we assume A_{sc} = 1,030 mm²

Total moment of resistance = $M_{u,lim} + M_{u2}$ = 1,220.20 + 167.81 = 1,388.01 kNm

Total A_{st} required = $A_{st,lim} + A_{st2}$ = 5,794.62 + 714.38 = 6,509.00 mm², (provided A_{st} = 6,509 mm²)

A_{sc} required = 880.326 mm² (provided 1,030 mm²).





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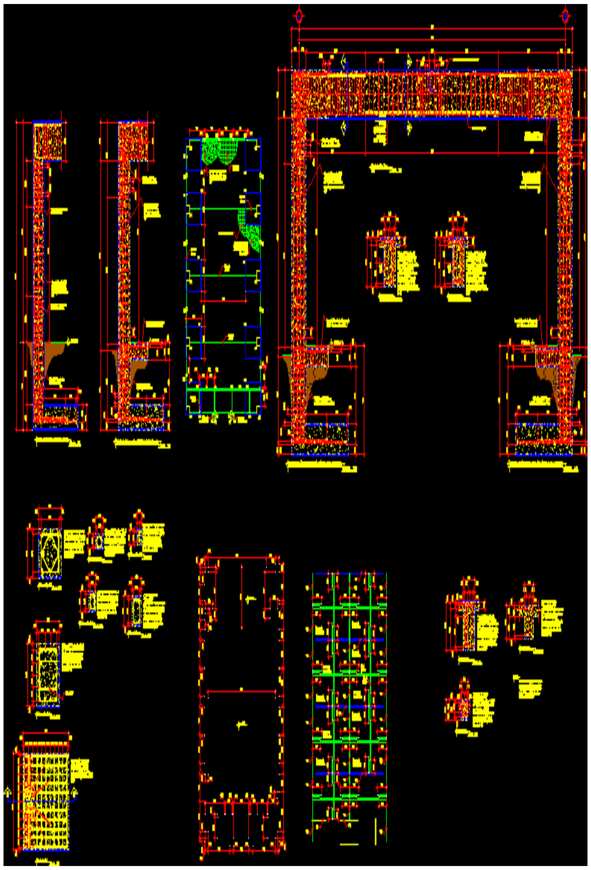

Conclusion

Maximum sagging (creating tensile stress at the bottom face of the beam) and hogging (creating tensile stress at the top face) moments are calculated for all active load cases at each of the above mentioned sections. Each of these sections are designed to resist both of these critical sagging and hogging moments. Where ever the rectangular section is inadequate as singly reinforced section, doubly reinforced section is tried.

Design for Shear:
Shear reinforcement is calculated to resist both shear forces and torsional moments. Shear capacity calculation at different sections without the shear reinforcement is based on the actual tensile reinforcement provided by STAAD program. Two-legged stirrups are provided to take care of the balance shear forces acting on these sections.
The default design output of the beam contains flexural and shear reinforcement provided along the length of the beam.
Columns are designed for axial forces and biaxial moments at the ends. All active load cases are tested to calculate reinforcement. The loading which yield maximum reinforcement is called the critical load. Column design is done for square section. Square columns are designed with reinforcement distributed on each side equally for the sections under biaxial moments and with reinforcement distributed equally in two faces for sections under uni-axial moment.

The comparative study was conducted based on the analytical modeling of simply supported RC overhanging T-beam slab by rational method . Based on this study Courbon's method gives the average result with respect BM values in the longitudinal girder as compared to GuyonMassonet method. whereas Guyonmassonet method underestimates the BM values when compared with Courbon's method.

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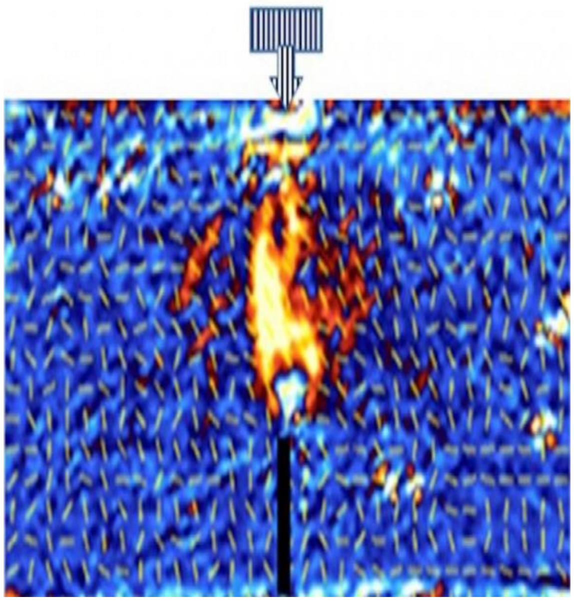
USING THE PRINCIPLES OF LIGHT, UNIVERSITY OF LEEDS SCIENTISTS HAVE DISCOVERED A NEW WAY TO MEASURE THE STRENGTH OF MODERN FORMS OF CONCRETE -- GIVING INDUSTRY A BETTER WAY TO UNDERSTAND WHEN IT COULD FRACTURE.

By – **Rabi Das**
(Lecturer Civil Engineering Department) with the following students
Dip Majumder (2nd yr, DCE) & Saikat Chakraborty (3rd yr, DCE)



A picture from a photonic camera showing how using the coating can create a candle-like 'flame' highlighting shear stress distribution in a sample concrete beam.

Their approach was based on applying a complex light-refracting coating, designed to display stress positions, to the surface of concrete beam samples.

The epoxy coating is 'birefringent' -- it has the ability to split light waves in different directions in relation to the amount of stress acting in those directions, and reflecting back to a photonic camera. The camera then takes a picture showing where the stress levels are most extreme before cracks or fractures occur.

While the coating itself is not new, this research project was the first time it had been used to measure shear stress and assess concrete toughness against fractures.

Dr Joseph Antony from the School of Chemical and Process Engineering at the leading Russell Group UK university, who led the study together with researchers at the University of Qatar, said: "There are other methods to measure stress and strain levels in the engineering sector, but we do not believe any of them can measure shear strain directly with high precision, which is most relevant to assess the failure strength of materials.

"The photonic method we developed can directly measure sheer strain, even on opaque materials. Until now, photonic and optical methods of measurement have only been associated with transparent materials."

The results using the new method compared favourably with conventional methods of stress testing, which have relied on combined experimental and numerical or analytical approaches.

The rise of composite concretes now used extensively in the construction industry prompted the team to look for new ways to study the material's strength.

Concrete has traditionally been made with cement, gravel and sand but has changed significantly in recent decades. It can now include numerous waste products including plastic pellets, in order to reduce the levels of natural materials used and to recycle waste products.

Dr Antony added: "Our study was aimed at developing a method by which plastic or polymer waste materials, in this case from Qatar, could be used as valuable ingredients in developing new engineering products.

"By working with industries which recycle the waste products into micron sized particles, we had direct insight into how they are used, meaning our study could be much more informed by industry requirements."

Finding a new way to show industry the precise toughness of these new forms of composite concrete meant there could be more reliance on their use as a building material.

Dr Antony explained how concrete made with waste plastic products had shown superior qualities to traditional ingredients, but his team wanted to ensure it could sustain service loads without fracturing.

He added: "We believe this new photonic or optical approach to fracture testing could be applied not only to develop sustainable manufacturing using materials that would otherwise be discarded as waste, but also in other diverse engineering designs including mechanical, civil, materials, electronics and chemical engineering applications."

The research was funded by the Qatar National Research Fund, and is published in *Scientific Reports*.

ROBOTICS AND AUTOMATION TECHNOLOGY IN CONSTRUCTION

By – Somedeb Saha
(Lecturer Civil Engineering Department)

Construction is one of the major industries around the world. Construction industry is labour-intensive and construction works performed in dangerous or risky situations because of many problems associated with labours like education, skill, experience, human tendency, strike, etc. Hence, the importance of construction automation has grown rapidly. Applications and activities of automation in this industry started in the early 90s aiming to optimize equipment operations, improve safety, enhance perception of workspace and furthermore, ensure quality environment for construction works. Construction productivity on large projects, including road construction, has been constant or declining since the 1970s. This has been coupled with a dramatic increase in construction labor cost and shortage in funding for new road construction and maintenance. At the same time, highway construction costs have been increasing, even after correcting for general inflation. One viable solutions is partial or full automation in road construction.

Overview Of Automation In Construction

For rapid construction with less risk and good quality there has been more and more use of machines as well as equipments in the construction industry. Human efforts and risks are reduced by using machines, robots, etc. at appropriate places. Since India has second largest man - power in the world, automation is not replacements of the human-power but is an important supplement that caters to the need of mega-construction and fast-track construction. Nowadays, in India, the human power is replaced by new technologies of automation because of unskilled labours, they do not give good quality work as compared to automation. Automation increases the productivity of the construction project, reduces the duration and laborious work, and increases the construction safety, increases the quality of work as compared to unskilled workers.

Civil Infrastructures

In the field of road construction, several projects had been developed over the last decade. They were mainly focused in the development of the new generation of semi-autonomous road pavers and asphalt compactors. The EU projects CIRC and latter OSYRIS had as the main objectives, based in the GPS and laser data, the semi-autonomous guidance of the machines and the quality control of pavers and roller processes by controlling the speed, temperature, layer thickness, travelled distance, etc. The coordination of several machines in order to improve productivity is also the objective of the project.

Product of OSYRIS

In the field of earthwork the research is centered in the introduction of new control techniques to existing machinery like excavators, bulldozers, draglines, etc. One of the major exponents of this research area is the control by CSIRO of the 100-m tall walking crane used in surface coal mining. The swing cycle of the dragline accounts for about 80 percent of time taken. The automatic swing cycle improves the efficiency of the machine, taking in mind that the bucket which weighs around 40 tones when empty and up to 120 tones when full, acts as a large pendulum and requires operator skill to control well. The torque-force control during the excavation is also improving the productivity of the processes. The University of Sydney project developed an automated excavator that accounts for interaction forces in analyzing the required bucket motion therefore seems promising. As the bucket comes in contact with its environment, the contact force must be regulated such that it remains within a specific range by using specific control strategy.

The Earthwork machines such as excavator, grader, asphalt paver, etc. should consist of the Laser – based sensor systems installed on it. By using this system, we get the required field data with the help of the laser sensors which acts alongside the working of machines. The field data is then displayed on the LCD screen to the operator, with the help of which he can make necessary adjustments in the machine before commencement of work.

· The earthwork machines used for the construction of bituminous roads should be fully equipped with required features and should be fully functional.

ASPHALT PAVER MACHINE

The asphalt paver machine should have the depth adjustment knob working, with the help of which the required depth of the asphalt layer to be laid on the road can be adjusted.

· Laying of tack coat/prime coat should be done using the machines such as Bitumen Pressure Distributor.

Power broom machine or the Road sweeper should be used for the road cleaning before the laying of tack coat and prime coat, which will remove all the unwanted waste materials from the construction site.

· Paver should regulate the grade and the cross slope of the pavement.

· To know the underground profile before excavation, Ground Penetrating Radar System (GPRS) should be used to avoid cutting of live wire cables and pipelines if any. Using this machine, we get to know the underground profile of the area. Then the excavation work can be done easily without disturbing other things.

BRICK PAVER

House Building

The last decade has witnessed the development of several robots for automatic assembly of buildings. An effort had been done in the brick laying masonry and the development of robotic prefabrication of façade and wall elements. The EU project ROCCO developed a large-range (10 m reach) and high payload (up to 500 kg) hydraulic 6 DOF robot for brick

6-DOF ROBOT Brick Assembler

Assembly. The robot is equipped with auto-tracking laser telemeter in the tip in order to perform prices (up to 5 cm) brick assembly. In this way the control system avoid important arm flexion. The robot performs the assembly sequence obtained by the planning software and needs an initialization process in order to know the bricks pallet position.

During the last few years a tendency to develop wearable robots for different applications immerged. First this type of robot were thought of from a military point of view, and that is to provide soldiers with powered exoskeletons to allow them handle heavy loads and resist longer periods without being exhausted. The main limitation of these robots is their power supply, but in the construction site this should not be a serious problem, since the robot can be iambically connected to a power source while being wearied by an operator.

A wearable/exoskeletons robot is able to endow the operator with more strength beyond his natural limits and allow him/her to handle heavy objects during their construction activities such as carpentry or fitting ceiling boards as they require large muscular power. The prototype developed in is an example of such application.

Ground Penetrating Radar System

<u>Safety Of Operators And Machines</u>

Thousands of construction workers are injured or killed in construction accidents each year. Researches and development efforts have been made in the last few years to look into new ways of improving the security and developing methods and reliable systems to detect possible failures and to avoid any harm to the workers, machines and installations.

There are two basic security levels: machine and human ones. Machine level refers to the failures in the machinery, possible erroneous operation, bad condition of the components, etc. As far as the human level is concerned, the objective is to prevent the operatives from suffering the accidents. The strategy to adopt consist in the definition of different safe and prohibited zones around the workers and the sources of danger, so that in the moment in which these areas comes into contact a danger situation is triggered and warning is generated. There are several actions to be done in this situation such as advising the worker thought the voice instructions, halting a machine movement via central computer among others. The proposed prototype systems records all the detected risk situations for later examination and is able to be used for monitoring of some activities of the site as it records the position of workers and automated machines continuously .

Safety Device- A Halting Machine

Advantages of Automation]

The project success from the project management's view point is achieved when the project is completed with the lowest possible cost, the highest quality, no accidents, etc. In other words, success means bringing each of the project performance indicators such as cost, schedule, quality, safety, labour productivity, materials consumption or waste, etc. to an optimum value. Applying automation and robotics in construction is addressed from the perspective of raising projects performance to serve the client and the environment. Automation and robotics systems in construction industry may achieve the following advantages:

· Uniform quality with higher accuracy than that provided by skilled workers.

· Improving work environment as conventional manual work is reduced to a minimum, so the workers are relieved from uncomfortable work positions.

· Eliminating complaints about noise and dust concerning works such as removal, cleaning or preparation of surfaces.

· Increasing productivity and work efficiency with reduced costs.

· Higher safety for both workers and the public through develop.

Use of ROBOTICS in infrastructure project

Conclusion

- **The importance of implementing automation technologies is the need of today's infrastructure project and construction firms in order to increase the productivity and good quality of work.. To achieve satisfactory road construction work with necessary quality and safety, we must follow the automation adopted by developed countries.**
- **As the machines used for construction work are not up to mark, they need to be upgraded with the newer machines with all the extra features in it.**
- **The work procedure, materials used, standard of quality etc. all should be done according to the recommended design.**
- **There should be a different committee for stricter supervision on the constructional works by their higher authorities to avoid use of inferior quality materials, inefficient machines, improper work procedures etc. all that are being followed nowadays.**

- **The operators shall be mandatorily trained under renowned construction companies (like VOLVO, CATERPILLAR, JCB, etc.) to gain proper knowledge and increase skills. They give information and training to the labour/operators/workers etc. to operate these automated machines and follow various techniques for speedy construction work.**

ROLLER

- **Necessary diversions, pathways, proper barrications and sign boards should be used before starting the work for safety purposes.**
- **All labours/operators/workers need to be fully equipped with Personal Protective Equipments (PPEs) before the commencement of any constructional activity.**

There is a wider scope for automation in construction sector and its utilization of automation is growing rapidly since last decades

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A DETAIL ESTIMATION OF A WELL FOUNDATION WITH OUTER DIAMETER 7M ON THE RIVER GANGA

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Introduction

Well foundations is a type of deep foundation which is generally provided below the water level for bridges. Well have been in use for foundation of bridges and other structures since Roman and Mughal periods. There are different type of well available based on its shape. Single circular well shape is the most common well, because of its easy construction and design process. A circular well has the minimum perimeter of a given dredge area. Since the perimeter is equidistant at the points from the centre of dredge hole, the sinking is more uniform than the other shape. Although the choice of a particular shape of well depends on the size of pier, the consideration of tilt or shift during sinking, the care plus cost of sinking and the vertical or horizontal acted forces.

In this project, a detail estimation of a well foundation with outer diameter 7m on the river Ganga, is carried out. Where the height of the well from top to bottom is 16m.

Even though well foundations are the costliest among all other foundations they are extensively used for bridges, marine structures, abutments in lakes, rivers and seas, breakwaters, other shore protection works, large water-front structures such as pump houses, subjected to huge vertical and horizontal forces, isolated heavy structures such as chimneys, and even to large and heavy buildings. Wells are also used as foundations for the high voltage carrying electric towers.

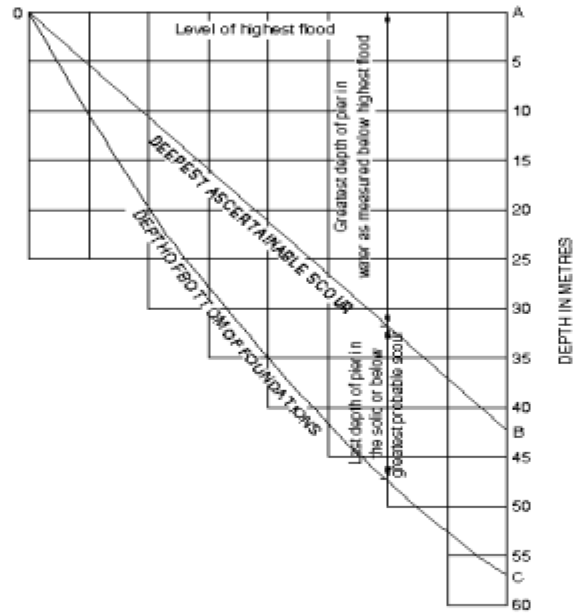
Wherever considerations of scour or bearing capacity require foundations being taken to a depth of more than 15 to 20 m, open excavation becomes costly and uneconomic as heavy timbering and shoring has to be provided. Progress will be slow, particularly where dewatering is involved. Another disadvantage of adopting the conventional type of footing is that the excavated material which is refilled around the structures is loose and hence easily scoured as compared to that of the natural ground.

The main aim of this study is to check the suitability of a developed computer program for the analysis, design and drafting of well foundation. Three approaches were used to for this purpose: (a) IRC method, (b) Beams on elastic foundations using Winkler model, and (c) Continuum mechanics based on Mindlin Solution (Poulos and Davis, 1980). For this study, the well was assumed to be a rigid foundation and buckling in the well is neglected. Soil is assumed to be homogeneous and elastic. Results obtained from the three methods of analysis were compared to develop an understanding about the erelative merits of these methods.

WELL FOUNDATION

Depth Of Foundations The depth of deep foundations below the high flood level shall be determined as indicated in clause 6.10.1. For substructures in sandy strata the depth of foundations may be determined from Fig 1 which is based on Technical Paper No 153 PL: XIII. The choice of type and shape of well foundation will depend upon the soil, type, the size and shape of pier or abutment, depth of foundation and available construction material. Where major obstructions such as uneven rocky strata are likely to be encountered, provision for pneumatic sinking may be made. Small obstructions can be removed either with the help of divers or by chiselling. 1.2. Shape And Cross-Section Of Wells: The horizontal cross-section should satisfy the following requirements: (a) The dredge hole should be large enough to permit dredging. (b) The steining thickness should be sufficient to enable sinking without excessive kentledge and provide adequate strength against forces acting on the steining, both during sinking and service. The well steining should also be designed to withstand the earth pressures acting only on two opposite sides or only on diametrically opposite quadrants under conditions of sand blowing. The effect of heap of earth dumped near the well during sinking shall also be taken into account. (c) It should accommodate the base of the substructure and not cause undue obstruction to the flow of water. (d) The overall size should be sufficient to transmit the loads safely to the soil without exceeding its allowable bearing pressure.

FIG.1 DIAGRAM OF THE DEPTH OF BRIDGE PIERS IN WATER AND IN RIVER BED RESPECTIVELY.



Explanation Of The Diagram: The intention of the diagram is to offer something definite in place of the rather fortuious method now centrally practiced. OA-Represents highest known flood level OB-Represents deepest ascertainable scour.OC-Represents depth to which foundation should be sunk. Note: 1. The diagram applies only to sandy bottom. If the river bed is soft, a greater depth is necessary. Piers are always presumed to have enough stone around them to prevent local pier formed swirls from scooping pot-holes at pier base. 2. This diagram is based upon Technical paper No 153 PI: XIIIA. (e) It shall allow rectification of the tilt and shift of the well without damaging the well. The shapes normally used are circular, double D. Dumb-bell, hexagonal or octagonal, square, rectangular and any of the above shapes with multiple dredge holes. 1.3. Allowable Bearing Pressure And Modulus Of Sub-Grade Reaction 1.3.1. The allowable bearing pressure may be determined in cohesion less soils on the basis of the penetration test results as given in IS: 3955 and reproduced below: Q = 9.8 {5.4 N2 B + 16(100+N2) D} in Newton/m2 [Q = 5.4 N2 B +

16(100+N2) D in Kg/m2 where, Q = Bearing capacity of soil under the well foundation in N/m2 (Kg/m2) N = Number of blows per 30cm in the standard penetration test. B = smaller dimension of the well cross-section in metre. D = Depth of foundation below scour level in metre. The capacity worked out by the above formula is applicable only for safety against shear failure. For well foundations, settlement governs the allowable bearing capacity in most cases. The permissible value of settlement is generally kept within 25mm and the allowable bearing pressure qa for such settlement can be obtained approximately by the following equation: qa=9.8 x 5/6 (1+ .3/2B)2 for B > 1.2m - in Newton/m2 [qa=5/6 (1+ .3/2B)2 for B > 1.2m - in kg/m2] qa= N approximately irrespective of B qa= 9.8 x 1.367 N for B U 1.2 m – in Newton/m2 [qa= 1.367N t/m2 for B U 1.2 m – in kg/m2] Where, N=corrected standard penetration resistance (No of blows per 30 cm) If larger settlement can be tolerated, the allowable bearing pressure could be increased accordingly. For clayey strata settlement should be worked out for full load based on consolidation test results. For wells constructed in cohesion less soils where full settlement due to dead load will take place by the time construction is completed and the necessary adjustments in the final level can be made before erection of girder, dead load due to well and the substructure can be ignored. In such cases, settlement shall be evaluated only for superstructure, live load and loss of friction in the well due to scour. 1.3.2. The passive pressure and skin friction shall be taken only for soil below the level of scour. In seismic areas relief due to skin friction should be ignored. The average value of skin friction may be adopted as per following equation.

$$F = 9.8\left(\frac{1}{2}K_a\gamma\left(Z - \frac{2C\sqrt{K_a}}{\gamma}\right)\tan\frac{2}{3}\phi\right) \quad in$$

N/m^2

$$\left[F = \left(\frac{1}{2}K_a\gamma\left(Z - \frac{2C\sqrt{K_a}}{\gamma}\right)\tan\frac{2}{3}\phi\right) in\ kg/m^2\right]$$

Where,

F = Skin friction in N/m^2 (kg/m^2)

K_a = Active earth pressure coefficient.

Loading 1.4.1 Wells shall be designed to resist the worst condition due to possible combination of the following loads, as may be applicable, with due regard to their direction and point of application. (a) Vertical Loads: i) Self-weight of well. ii) Buoyancy iii) Dead load of superstructure, substructure.iv) Live load, and v) Kentledge during sinking operation (b) Horizontal Forces: i) Braking and tractive effort of moving vehicles. ii) Forces on account of resistance of bearings. iii) Forces on account of water current or waves. iv) Centrifugal force, if the bridge is situated on a curve. v) Wind forces or seismic forces. vi) Earth pressure. vii) Other horizontal and uplift forces due to provision of transmission line tower (broken wire condition) etc. 1.5 Tilt And Shifts As far as possible wells shall be sunk without any tilt and shift. A tilt of 1 in 100 and shift of D/40 subject to a minimum of 150 mm shall be taken into account in the design of well foundation (D is the width or diameter of well). If greater tilts and shifts occur, their effects on bearing pressure on soil, steining stresses, change in span etc. should be examined individually. 1.6 Cutting Edges Cutting edge shall be properly anchored to the well curb. When there are two or more compartments in a well the bottom of the cutting edge of the intermediate walls may be kept about 300 mm above the cutting edge of the outer wall to prevent rocking. 1.7 Well Curb It should transmit the superimposed load to the bottom plug without getting overstressed and it should offer minimum resistance to sinking. The slope to the vertical of the inner faces of the curb shall preferably be not more than 30 degrees. In sandy strata, it may be upto 45 degrees. An offset on the outside (about 50 mm) may be provided to ease sinking. The curb shall invariably be of reinforced concrete with a minimum reinforcement of 70 kg/m3 excluding bond rods. In case blasting is anticipated, the inner face of the curbs shall be protected by steel plates or any other means to sufficient height.

Well Steining Well steining shall be built of masonry or cement concrete not weaker than M-100 grade. Sufficient bond rods shall be provided to bond the units of the steining during the progress of construction. Bond rods shall be distributed evenly on both faces of steining and tied up by providing adequate horizontal hoop reinforcement. For masonry steining and for concrete steining of small thickness, bond rods may be provided in one row in the centre only and tied up by providing plates or hoop reinforcement. 1.9 Bottom Plug A bottom plug shall be provided for all wells and its top shall be kept 300 mm above the top edge of the inclined face of the curb. The concrete used for the bottom plug when placed under dry conditions shall generally be of 1:3:6 proportions and it shall be placed gently in one operation. When the concrete is placed under water, the quantity of cement shall be increased by 10% and it shall be placed by tremie or skip boxes under still water condition. 1.10 Top Plug A 300 mm thick plug of cement concrete 1:3:6 shall be provided over the hearting which shall normally be done with sand. Sometimes only water is filled to reduce the weight. 1.11 Well Cap The bottom of the well cap shall, as far as possible, be located 300 mm above low water level. All the longitudinal bars from the well steining shall be anchored into the well cap. The well cap shall be designed as a slab resting on the well. 1.12 Pneumatic Sinking Of Wells Where boring data indicate pneumatic sinking, it will be necessary to decide the method of such sinking and location of air lock. 1.12.1. The side wall and roof of the working chamber shall be designed to withstand the maximum air pressure envisaged with the use of pneumatic sinking equipment. The design air pressure for design shall be higher than the pressure due to the depth of water above the bottom of the well. 1.12.2. In case the concrete steining is used and the tension in concrete exceeds three-eighths of the modulus of rupture, the section of the steining shall be changed to keep the tensile stress within this limit or mild steel reinforcement shall be provided suitably over the width of the steining. The following further points shall be kept in view. (i) Extra hoop reinforcement, if required to be provided, shall overlap at least one bond length below the section from where MS plates are provided for protection against blasting or other reason. (ii) The pneumatic platform and the weight of the steining and kent ledge, if any, shall be sufficient to resist the uplift of air from inside. (iii) If at any section of steining the uplift pressure is more than the total weight acting downwards, then the platform and the steiningcan be weighed down by kentledge and also anchored to the steining, if necessary. (iv) The well steining shall also be checked at different sections for any possible rupture against the uplift force and upto the height at which the uplift force is balanced by the self weight of the steining and any superimposed load on it.

6 BASIC ELEMENTS OF A WELL FOUNDATION

Suryakanta | February 22, 2016 | Foundation, Geotechnical | 1 Comment

WELL FOUNDATION

Well foundations have their origin in India & have been used for hundreds of years for providing deep foundation to important buildings and bridges. Well foundations were freely used during the Moghal Period for bridges across the major rivers. Moghal monuments including TajMahal are built on well foundations. Well foundations provide a solid & massive structure. This foundation has maximum sectional modulus for a given cross-sectional area. Wells can resist large horizontal forces & vertical loads even when the unsupported length is large in scourable river beds. A well foundation is monolithic and relatively rigid in its structural behavior.

BASIC ELEMENTS OF A WELL FOUNDATION

A well foundation is a type of foundation which is generally built in parts at the surface and sunk to its final position, where it forms the permanent foundation. **Fig-1** shows a typical section of a circular well foundation.

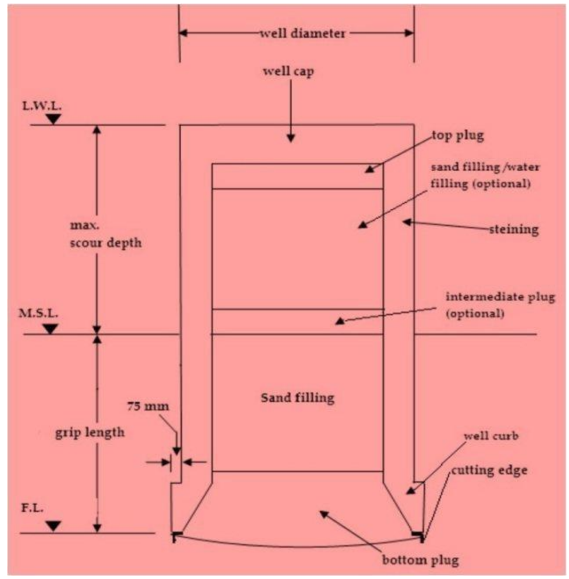


Fig-1 Section of Well Foundation

1. WELL-CAP

It is a RCC slab laid at the top of the well steining to transmit the loads and moments from the pier to the well or wells below. Shape of well cap is same as that of well with a possible overhand of 150 mm all-around to accommodate lengthy piers. It is designed as a two-way slab with partial fixedity at supports. The top of the well cap is usually kept at the bed level in case of rivers with seasonal flow or at about the low water level in case of perennial rivers. **Thickness of well cap** is usually between **1500 mm to 2000 mm**.

2. STEINING

It is the main body of the well which transfers load to the base of the foundation. Steining is normally of reinforced concrete. Minimum grade of concrete used in steining is **M20** with cement content not less than 310 kg/m^3. To facilitate well sinking an **off-set of 75 mm to 100 mm** is provided in well steining at its junction with the well curb.

The **thickness of well steining should not be less than 500 mm** nor less than that given below.

t = KD*(L$^{1/2}$)

Where,

t = minimum thickness of concrete steining, **m**,

D = external diameter of circular well or dumb bell shaped well or smaller plan dimension of twin D well, **m**,

L = depth of well in **m** below L.W.L. or top of well cap whichever is greater,

K = a constant depending on the nature of subsoil and steining material (taken as **0.30** for **circular well** and **0.039 for twin – D well** for concrete steining in sandy strata and 10% more than the corresponding value in the case of clayey soil).

3. WELL CURB

It is the wedge shaped RCC ring beam located at the lower portion of the well steining provided to facilitate sinking. Well curb carries cutting edge for the well and is made up of reinforced concrete using controlled concrete of grade **M25**. The cutting edge usually consists of a mild steel equal angle of side 150 mm. In case blasting in anticipated, the outer face of the well curb should be protected with 6 mm thick steel plate and the inner face should have 10 mm thick plate up to the top of the curb and 6 mm plate further up to a height of 3 m above the top of the curb.

4. BOTTOM PLUG

After the well is sunk to the required depth, the base of the well is plugged with concrete. This is called the bottom plug. It acts like an inverted dome supported by the steining on all the sides and transmits the load to the subsoil and acts as a raft against soil pressure from below. Minimum grade of concrete used in bottom plug is M15. Thickness of bottom plug should not be less than the half of dredge-hole diameter nor less than the value calculated using following formula.

$$t^2 = \frac{3W}{8\pi f_c}(3 + \vartheta)$$

Where,

W = total bearing pressure at the base of well,

f_c = flexural strength of concrete in bottom plug, , and,

ϑ = Poisson's ratio for concrete, **0.18 to 0.20**.

5. TOP PLUG

The top plug is an unreinforced concrete plug, generally provided with a thickness of about 600 mm beneath the well cap to transmit the loads from the pier to the steining. Minimum grade of concrete used in top plug is M15.

The space inside the well between the bottom of the top plug and the top of bottom plug is usually filled with clean sand, so that the stability of the well against overturning is increased. While this practice is good in case of wells resting on sand or rock, the desirability of sand filling for wells resting on clayey strata is doubtful, as this increases the load on the foundation and may lead to greater settlement. In the latter case, the sand filling is done only for the part of well up to scour level, and remaining portion is left free.

6. INTERMEDIATE PLUG

As discussed above, for wells resting on clayey strata, it is not preferable to fill the space inside the well completely with sand. In such cases, sand filling is not done or sand is filled up to the scour level. A concrete plug covering the filling is usually provided, known as intermediate plug. Usually, thickness of intermediate plug is taken as 500 mm.

Estimate of a Well Foundation

For preparing the estimate of a Well Foundation, followings things should be understand properly:-

- Basic concept of Well Foundation
- Understanding the drawings of the Well Foundation
- Method of measurement of various items of work involved.
- Rate of items o

- f work.
- Specifications

1) Basic Concept of Well Foundation

Definition

Well foundation is a type of deep foundation which is generally constructed for the foundation of bridges, for irrigation water supply, and for drinking water purposes. Sinking operation of well is done from the surface of either land or water to some desired depth.

Well is also called "Wells".

Types of Wells:

- Box wells
- Open wells
- Pneumatic wells

Common types of well shapes

1. Single circular
2. Twin circular
3. Dumb well
4. Double-D
5. Twin Hexagonal
6. Twin octagonal
7. Rectangular

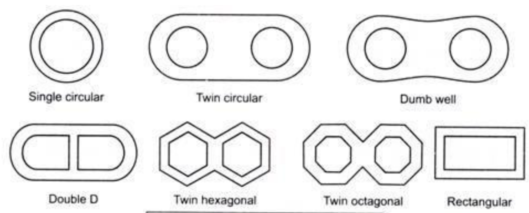


FIG. 11.30 Different shapes of wells

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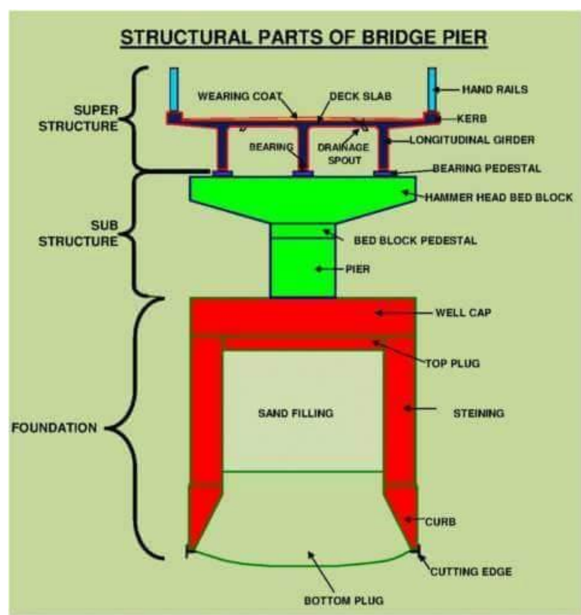
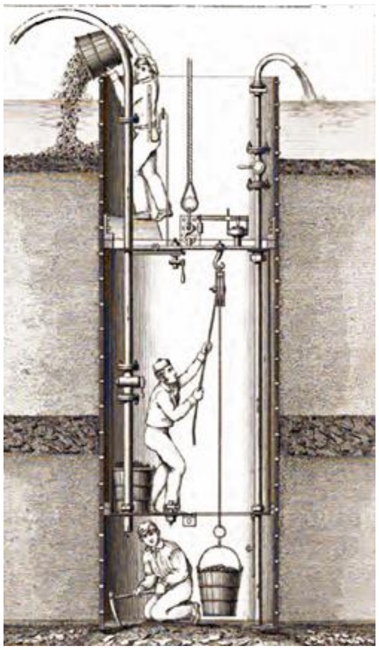
Some figures that help to understand the concept of the Well Foundation:





Section of well foundation

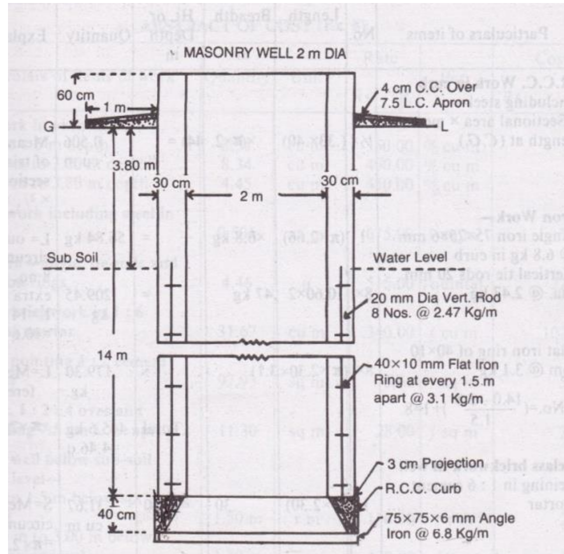
Three Dimensional View of Well Foundation

2) Understanding the drawings of the Well Foundation

To understand the drawings of the well foundation, visualization of each and every component of the well foundation is important

Let's explain the method of estimation of well foundation with an example:-

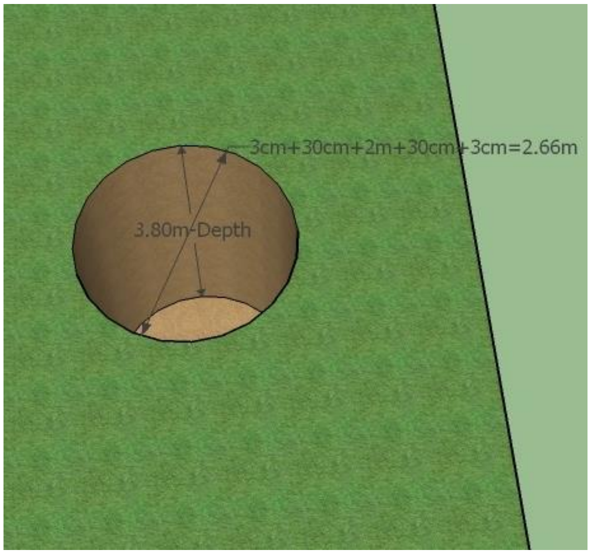
Q) Prepare a detailed estimate of a masonry well of 2m dia. And 14m deep exclusive the curb from the drawings given in figure below.The soil water level being 3.80m below G.L. The steining of well is of 30cm thick of 1st class brick masonry in 1:6 cement mortar. The inside and exposed surfaces shall be pointed with 1:2 cement mortar. The well should be raised 60cm above the G.L. and an apron of 4cm c.c. lime concrete 1m wide shall be provided all round the well. The curb shall be of R.C.C. with 75*75*6mm angle iron cutting edge.



Solution:-

Item no.	Description of items of works	No.	Length (m)	Breadth (m)	Height or Depth (m)	Quantity (m^3)	Remarks
1	Earthwork in excavation a) Upto 1.5m depth	1	π∗2.6624π∗2.6624		1.5	8.34	
	b) 1.5m to 3m depth	1	π∗2.6624π∗2.6624		1.5	8.34	
	c) 3m to 3.80m depth	1	π∗2.6624π∗2.6624		0.8	4.45	

Three Dimensional view of the earthwork in excavation:-



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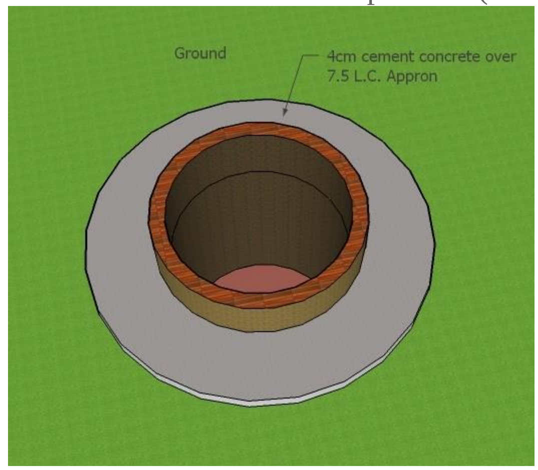
2)

Item no.	Description of items of works	No.	Length (m)	Breadth (m)	Height or Depth (m)	Quantity (m^3)	Remarks
2)	4cm C.C. over 7.5cm L.C. apron (floor)	1	3.14*3.60	-	1.00	11.30 sq.m	

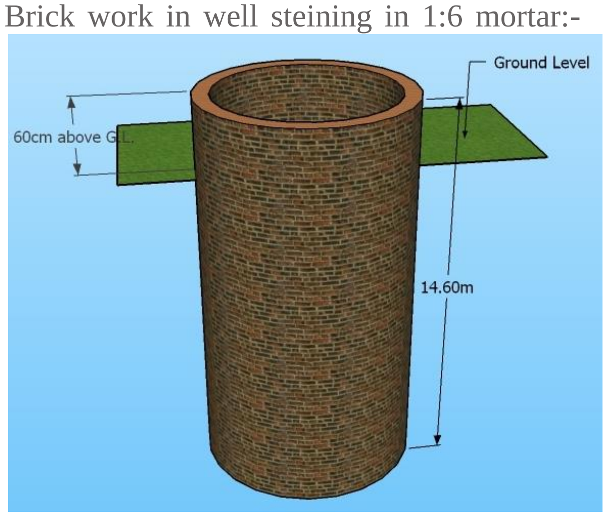
Three-Dimensional view of the 4cm C.C. over 7.5cm L.C. apron (floor):-



3)

Item no.	Description of items of works	No.	Length (m)	Breadth (m)	Height or Depth (m)	Quantity (m^3)	Remarks
3)	I-Class Brick work in well steining in 1:6 mortar	1	3.14*2.30	0.30	14.60	31.67	

Three-Dimensional view of the I-Class Brick work in well steining in 1:6 mortar:-



Caisson Foundation

- The foundation system of and the soils beneath the building prevent the complex from moving vertically.
- When a load is placed on soil, most soils settle. This creates a problem when the building settles but the utilities do not. Even more critical than settlement is differential settlement.
- This occurs when parts of your building settle at different rates, resulting in cracks, some of which may affect the structural integrity of the building. Conversely, in some rare instances soils may swell, pushing your building upwards and resulting in similar problems.
- Therefore, the foundation system must work in tandem with the soils to support the building.

WHAT IS CAISSONS?

- It's a prefabricated hollow box or cylinder
- It is sunk into the ground to some desired depth and then filled with concrete thus forming a foundation.
- Most often used in the construction of bridge piers & other structures that require foundation beneath rivers & other bodies of water
- This is because caissons can be floated to the job site and sunk into place
- Basically it is similar in form to pile foundation but installed using different way
- used when soil of adequate bearing strength is found below surface layers of weak materials such as fill or peat
- It's a form of deep foundation which are
- constructed above ground level, then sunk to the required level by excavating or dredging material from within the caisson
- A caisson foundation consists of concrete columns constructed in cylindrical shafts excavated under the proposed structural column locations
- Caissons are drilled to bedrock or deep into the underlying strata if a geotech eng. find the soil suitable to carry the building load
- It's created by auguring a deep hole in the ground
- Then, 2 or more 'stick' reinforcing bar are inserted into and run the full length of the hole and the concrete is poured into the caisson hole.
- The caisson foundations carry the building loads at their lower ends, which are often bell-shaped.

TYPES OF CAISSONS

- Box Caissons
- Excavated Caissons
- Floating Caissons
- Open Caissons
- Pneumatic Caissons
- Sheeted Caisson

ADVANTAGES

- Economics
- Minimizes pile cap needs
- Slightly less noise and reduced vibrations
- Easily adaptable to varying site conditions
- High axial and lateral loading capacity

DISADVANTAGES

- Extremely sensitive to construction procedures
- Not good for contaminated sites
- Lack of construction expertise
- Lack of Qualified Inspectors

Caisson Foundations

- A drilled pier is a deep foundation system that is constructed by placing fresh concrete and reinforcing steel into a drilled shaft.
- The shaft is constructed by rotary methods using either a self-contained drill unit or a crane mounted drill unit. The hole is advanced through soil or rock to the desired bearing stratum. Temporary or permanent steel casings may be used to maintain the sides of the drilled excavation if caving soils or water infiltration becomes a problem.
- Drilled shafts can be used to sustain high axial and lateral loads. Typical shaft diameters range from 18 to 144 inches.

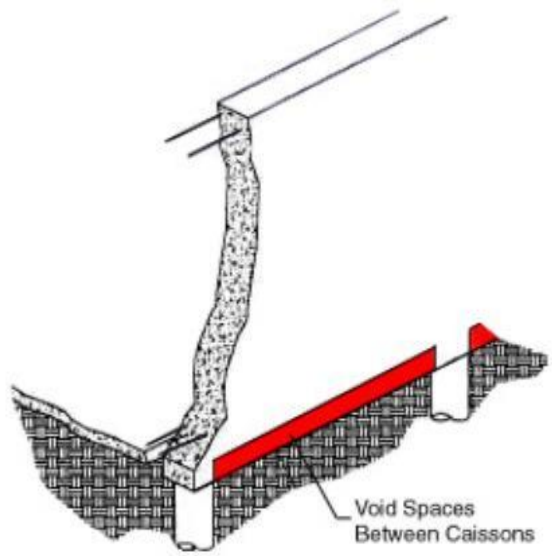
- Caisson foundations are similar in form to pile foundations, but are installed using a different method. Caissons (also sometimes called “piers”) are created by auguring a deep hole into the ground, and then filling it with concrete. Steel reinforcement is sometimes utilized for a portion of the length of the caisson. Caissons are drilled either to bedrock (called “rock caissons”) or deep into the underlying soil strata if a geotechnical engineer finds the soil suitable to carry the building load. When caissons rest on soil, they are generally “belled” at the bottom to spread the load over a wider area. Special drilling bits are used to remove the soil for these “belled caissons”.
- Drilled shafts (also called caissons, drilled piers or bored piles) have proven to be a cost effective, excellent performing, deep foundation system, that is utilized world-wide. Typically they are used for bridges and large structures, where large loads and lateral resistance are major factors.
- Caisson foundations are used when soil of adequate bearing strength is found below surface layers of weak materials such as fill or peat. A caisson foundation consists of concrete columns constructed in cylindrical shafts excavated under the proposed structural column locations. The caisson foundations carry the building loads at their lower ends, which are often bell-shaped

CONCRETE CAISSONS

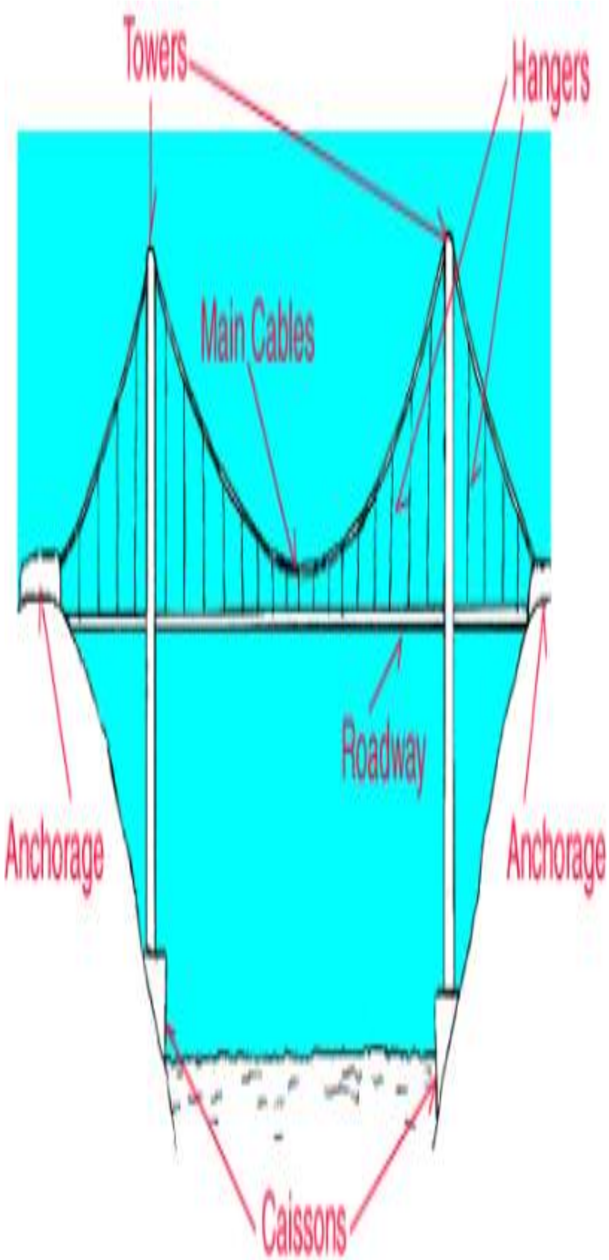
- A 10″ or 12″ diameter holes are drilled into the earth and embedded into bedrock 3 to 4 feet. Usually used for the structural support for a type of foundation wall, porch, patio, monopost, or other structure. Two or more “sticks” of reinforcing bars (rebar) are inserted into and run the full length of the hole and then concrete is poured into the caisson hole. A caisson is designed to rest on an underlying stratum of rock or satisfactory soil and is used when unsatisfactory soil exists.

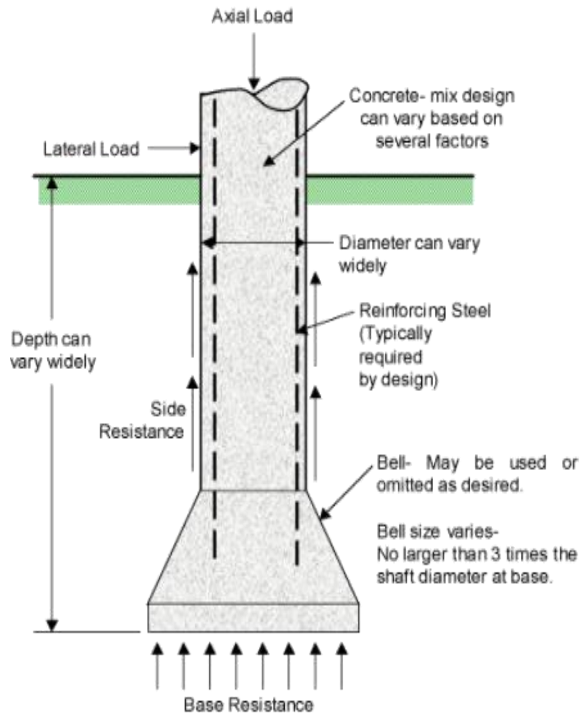
THE PROCESS: BUILDING A CAISSON

- After some initial form work and concrete pours, the **cutting edge** is floated to the breakwater by towboat and fastened to the **caisson** guide. Concrete is placed (poured) into steel forms built up along the perimeter of the box. With every concrete placement, the box becomes heavier and sinks into the water along the caisson guide.
- Forms are also built inside the box around the **air domes** and concrete is placed in between. The resulting open tubes above the air domes are called **dredge wells**



- When the caisson finally touches the river bottom, the air domes are removed and earth is excavated through the long dredge well tubes, as shown in the animation below. The caisson sinks into the river bottom. Excavation continues until the caisson sinks to its predetermined depth
- As a final step, concrete is placed (poured) into the bottom 30 feet of the hollow dredge wells and the tops are sealed.

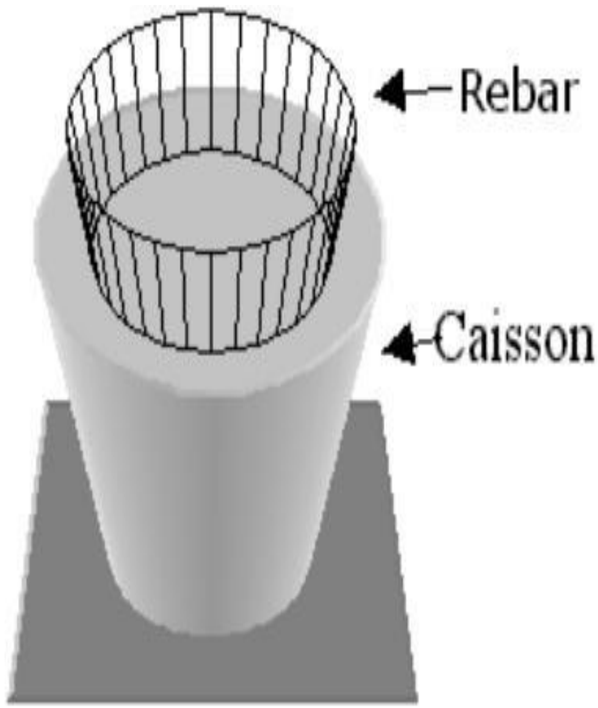




Straight Shaft Drilled Piers (Caissons)

- Used in moderate to high swelling soils. **(This is one of the most effective foundation designs for use in sites that contain expansive soils.)**
- Purpose is to attain required penetration into zone where there is little or no seasonal moisture variation. Current standard of care in the area is a minimum penetration of 6 feet into bedrock and minimum length of 16 feet. Dead loads should be as high as practical. This design requires relatively long spans between piers and more reinforcing in grade beam.
- Caissons into bedrock
- Friction Piers into stiff clays
- End Bearing Belled Piers
- **Appropriate Voiding –**

Should be constructed with void material (**Wall Void™**) of appropriate strength and thickness series of 1.2-metre thick diaphragm wall panels were joined to form a 24-metre diameter caisson shaft. Four of these caissons were built to provide a sound base for the foundation of the main structure of the building tower. The photo shows the excavation work using typical excavating machines inside one of the caisson shafts.



Box Caisson Foundation

This is circular or even rectangular cylindrical box with open top.

This is used when excavation is not required. It is simply sunk at spot filled with concrete or masonry and well cap is laid.

For the same above mentioned reason this to not suitable and avoided.

Open Caisson Foundation

This is an RCC or steel cylinder open fully or partly both at top and bottom. The caisson is floated to and sunk at spot. Then soil is excavated by a suitable means from within inside like a well foundation and the caisson is allowed sink.

One innovation involves in Caisson wells is pumping water at high pressure to inside which then mixes with soil and escapes from bottom to the sides. While this soil flow lets the caisson settle, the upward flow on outside surface of caisson helps reduce friction to sinking.

These wells could be of different shapes in cross section. The shape of the well is obviously linked to the size and shape of superstructure involved, ease and cost of fabrication and sinking, susceptibility to tilt and shift while sinking and the stability of the foundation.

Circular shaped well have less perimeter to area ratio, hence easy to drive in and they are less susceptible to tilt. However, its obstruction to water way is more. Used for small and short span bridges. For long span bridges, wells shaped like double-D or twin circular wells are used. They confirm to the shape of bridge pier cross section as well. In these types, usually two circular or D-shaped wells are sunk close and only well cap is laid in common. Rectangular, Rectangle with D ends, Dumb-bell, Twin hexagon and Twin octagonal wells are other types in use.

The flow is not clear and the pumping expectations with soil may fail and hence avoided.

Pneumatic Caisson Foundation

By far, it is the most widely used type of bridge foundation in deep river beds laid in perennial water. It is made of twin shelled stiffened box with concrete filling in between the shells. Its bottom has a cutting edge and top is closed with three openings. Of them two were provided with airlocks and used to movement of workmen and lifting excavated earth.

The other one is used to pour concrete. The caisson is fabricated on bank, sunk at desired spot, water from inside is pumped out and compressed air is pumped into well to keep water or soil from flowing in. after exposing the soil at well bottom, soil is excavated as usual with suitable means and the caisson well is allowed to sink slowly and uniformly. When it reached required depth or hard rock, the well bottom is sealed with a thick layer of concrete, to with stand water pressure. After the concrete sets enough, the air pressure is relieved and remaining space is also filled with concrete from opened top. Well cap and pier construction is done over it.

The depth and size of the wells required do not allow the air chambers and hence avoided.

The open well with manual excavation was resorted to considering the site restrictions on approaches and head way available. Fast excavations mat disturb the cast iron screw piling and the failures are nor affordable and would mean heavy repercussions and avoided.

FLOATING CAISSONS

In certain cities and even countries the demand for land and space is rapidly exceeding the supply. Maritime centers do not escape from this reality, the continuous expansion of commercial maritime traffic and activities in seaports due to increases in international trading has generated an increased demand for an effective use of ports and harbors. Construction activities have been oriented to the expansion of existing facilities.

Port and harbour facilities form the infrastructure that makes marine traffic possible, facilitating the construction of vessels, its protection against wave action and, its loading and unloading activities. In other words, they play an important role in facilitating international commerce.

The different **marine** works and harbor constructions in which these caissons can be used include:

- Ports
- Breakwaters
- Wharves
- Berthing Facilities and Docks
- Dry Docks and Slipways
- Fishing Ports and Marinas

Floating Caissons Fabrication

- Each caisson is built in an ascending sequence starting with the slab. The slab reinforcement cage is assembled in an auxiliary floating platform, then the cage is moved to the floating dock. A sliding form is placed and the slab is poured as a monolithic element

- After the slab is ready, the construction of the upper part of the caissons begins, ascending in increments of one meter using the sliding form Each of these increments includes: placing the reinforcement, sliding the forms, and pouring and vibrating the concrete. This sequence is repeated until the total height of the caisson is reached
- Once the caisson fabrication is completed, a special set of supporting and locking bars are removed to allow the release of the caisson from the floating dock. The caisson floats by itself and is guided with the help of cables from the shore and tow-boats, to its final location (Fig. 4). When the caisson reaches the final position the cylinder cavities begin to be filled with granular material. This operation is performed by auxiliary floating platforms that carry both the material and a special crane to transfer the material. Tractors, dozers, loaders and trucks help finish the filling operation on top of the caisson In the floating dock, operations begin for the fabrication of the next caisson

Step 1 – Construction of Hand-dug Caisson

- Hand-dug Caisson is one of piling methods in the past, however, it is almost banned in Hong Kong. Therefore, we should know something about it and there are some notes about it.

Installation of Hand-dug Caisson

- a. Set out caisson position and size.
- b. Excavate one meter into ground.
- c. Erect caisson lining steel form.
- d. Concrete caisson lining.
- e. Erect excavation platform on top of caisson centre.
- f. Dismantle caisson lining steel form on the next day.
- g. Repeat step 2 to 6 (excluding step 5) until bedrock.
- h. Excavate bellout into bedrock until required level.
- i. Fix caisson reinforcement.
- j. Clean caisson bottom.
- k. Install concreting chute.
- l. Concrete caisson heart until required level.

Step 2 – Preventive Measures

- In order to stabilise any unstable layers of subsoil which may be encountered during caisson excavation and prevent excessive settlement to the adjoining building/pavement due to the effect of dewatering, grouting is one of methods can be adopted as a preventive measure before caisson excavation.

Step 3 — Monitoring

- In order to ensure no adverse effects imposed on the adjoining structures during caisson construction, the following precautionary measures and limiting criteria, to be monitored throughout the construction period, are recommended
- a. The level of checkpoints should be monitored regularly and the measured settlement of building structures and road pavement must not exceed 10 mm and 25 mm respectively.
- b. Standpipes piezometer shall be installed before the caisson excavation for monitoring of the ground water table drawdown which shall in no case exceeding the specific value.
- c. During the construction period, duplicate copies of all monitoring results shall be submitted to the Consultants on a regularly basis and be kept available on site for inspection at all times.

Caisson Sinking

There are two methods for lining a well with caissons. The first method involves digging an unlined well and then lowering the caissons into place. This is very similar to the in-situ method of lining. It has all the same problems of safety but without of the benefits of a tight grout seal along the edge of the well. Problems also arise if the hole is not strait and uniform. The additional backfilling that is required also makes the method undesirable.

Digging a Caisson Lined Well

A starter hole can be dug first, or the bottom caisson can be placed directly on the ground. It is important to make sure the first few segments start out strait and level. As the hole progresses the upper sections will keep the lining strait, but it must be strait to begin with.

If separate pre-cast sections are used it is important that the sections are secured together. If the sections are simply stacked, the bottom caisson, or the cutting ring, can fall out of place while the stack progresses. This is potentially dangerous and it can be impossible to recover from.

Finishing a Caisson Lined Well

When the bottom of the caisson reaches the required depth, the bottom should be filled with 7-10 cm of gravel to allow good flow up to the well but prevent fines from moving up. To increase the inflow, the bottom sections are often perforated.

Some sources recommend always using a perforated or porous material on the sections under the water table. Others say that joints between the sections will allow substantial flow and modifying the bottom sections with holes or porous concrete is not worth the decrease in strength.

Well Foundation

This work consists of construction of well foundation, taking it down to the founding level through all kinds of sub-strata, plugging the bottom, filling the inside of the well, plugging the top and providing a well cap in accordance with the details shown on the drawing. Well may have a circular, rectangular, or D-shape in plan and may consist of one, two or more compartments in plan.

Well Components & their Function

In brief the function of various elements is as follows:

1. Cutting edge

The mild steel cutting edge shall be made from structural steel sections. The cutting edge shall weigh not less than 40 kg per metre length and be properly anchored into the well curb, as shown in the drawing.

When there are two or more compartments in a well, the bottom end of the cutting edge of the inner walls of such wells shall be kept at about 300 mm above that of outer walls.

2. Curb

The well curb may be precast or cast-in-situ. Steel formwork for well curb shall be fabricated strictly in conformity with the drawing. The outer face of the curb shall be vertical. Steel reinforcements shall be assembled as shown on the drawings. The bottom ends of vertical bond rods of steining shall be fixed securely to the cutting edge with check nuts or by welds.

The formwork on outer face of curb may be removed within 24 hours after concreting. The formwork on inner face shall be removed after 72 hoursIt is made up of reinforced concrete using controlled concrete of grade M-35.

3. Steining

The dimensions, shape, concrete strength and reinforcements of the well shall strictly conform to those shown on the drawings. The formwork shall preferably be of M.S. sheets shaped and stiffened suitably. In case timber forms are used, they shall be lined with plywood or M.S. sheets. The steining of the well shall be built in one straight line from bottom to top such that if the well is tilted, the next lift of steining will be aligned in the direction of the tilt. After reaching the founding level, the well steining shall be inspected to check for any damage or cracks

4. Bottom plug

Its main function is to transfer load from the steining to the soil below. For bottom plug, the concrete mix shall be design (in dry condition) to attain the concrete strength as mentioned on the drawing and shall contained 10 per cent more cement than that required for the same mix placed dry.

5. Sand filling

Sand filling shall commence after a period of 3 days of laying of bottom plug. Also, the height of the bottom plug shall be verified before starting sand filling. Sand shall be clean and free from earth, clay clods, roots, boulders, shingles, etc. and shall be compacted as directed. Sand filling shall be carried out upto the level shown on the drawing or as directed by the Engineer.

6. Intermediate plug

The function of the plug is to keep the sand filling sandwiched & undisturbed. It also act as a base for the water fill, which is filled over it up to the bottom of the well cap.

7. Top plug

After filling sand up to the required level a plug of concrete shall be provided over it as shown on the drawing, It at least serves as a shuttering for laying well cap.

8. Reinforcement

It provides requisite strength to the structure during sinking and service.

9. Well cap

It is needed to transfer the loads and moments from the pier to the well or wells below. A reinforced cement concrete well cap will be provided over the top of the steining in accordance with the drawing. Formwork will be prepared conforming to the shape of well cap. Concreting shall be carried out in dry condition. A properly designed false steining may be provided where possible to ensure that the well cap is laid in dry conditions.

After water filling, pre-cast RCC slabs shall be placed over the RCC beams as per the drawings, as non-recoverable bottom shuttering for well cap. Initially built false wall shall act as outer shuttering for well cap casting. In case, there is no false wall, then steel shuttering is to be put from outer side.

For well Steining and well cap shuttering, permissible tolerances are as follows: -

- Variation in dimension : +50 mm to –10mm
- Misplacement from specified Position in Plan : 15mm
- Variation of levels at the top : +/- 25mm

Depth of Well Foundation

As per I.R.C. bridge code , the depth of well foundation is to be decided on the following considerations:

- The minimum depth of foundation below H.F.L should be 1.33D, where D is the anticipated max. Depth of scour below H.F.L depth should provide proper grip according to some rational formula.
- The max. Bearing pressure on the subsoil under the foundation resulting from any combination of the loads and forces except wind and seismic forces should not exceed the safe bearing capacity of the subsoil, after taking into account the effect of scour. With wind and seismic forces in addition, the max. bearing pressure should not exceed the safe bearing capacity of the subsoil by more than 25%
- While calculating max. Bearing pressure on the foundation bearing layer resulting from the worst combination of direct forces and overturning moments. The effect of passive resistance of the earth on the sides of the foundation structure may be taken into account below the max. Depth of the scour only.

The effect of skin friction may be allowed on the portions below the max. Depth of scour. Accordingly for deciding the depth of well foundation we require correct estimation of the following:

1. Max. Scour depth.
2. Safe bearing capacity.
3. Skin friction.

4. Lateral earth support-below max. scour level.

   It is always desirable to fix the level of a well foundation on a sandy strata bearing capacity. Whenever a thin stratum of clay occurring between two layers of sand is met with in that case well must be pierced through the clayey strata. If at all foundation has to be laid on a clayey layer it should be ensured that the clay is stiff.

Sinking of Wells

- In case of well sinking on dry grounds, an open excavation upto half a metre above subsoil water level is carried out and the well curb is laid. In case the wells are to be sunk in mid stream, a suitable cofferdam is constructed around the site of the well and islands are made.
- The islands in shallow water are formed by an edging of sand bags forming an enclosure filled with sand or clay. When the water depth is of the order of 3 to 5 m. the site is surrounded by sheet piling and the enclosure so formed is filled with clay or sand. The centre point of well is accurately marked on the island and the cutting edge is placed in a level plain. The wooden sleepers are inserted below the cutting edge at regular intervals so as to distribute the load and avoid setting of the cutting edge unevenly during concreting. The inside shuttering of the curb is generally made of brick masonry and plastered. The outer shuttering is made of wood or steel.
- Initially the well steining should be built to a height of 2m. Only. Later steining should not allowed to be built more than 5m. at a time. For this bridge the subsequent lifts were of 2.5 m. each.
- The well is sunk by excavating material from inside under the curb. Great care should be taken during well sinking in the initial stages because the well is very unstable. Excavation of the soil inside the well can be done by sending down workers inside the wells. When the depth of the water inside the well becomes more than one meter, the excavation is then carried out by a Jham or a Dredger.

- The sump position at 8 equidistant locations along dredge hole sides & at well center are taken & recorded. The dredge water level is also recorded.
- Vertical reinforcement of steining shall be bent & tied properly to facilitate the grab movement during sinking operations.
- The position of the crane shall be such that the operation shall be able to see the signalman on the well top at all the times, & the muck is safely deposited away from the intermediate vicinity of the well.
- Grabbing process shall commence normally with the grabbing at the above designated sounding positions.
- If the well is not sinking after reasonable amount of grabbing is done, say after two rounds of grabbing, the sump position shall be checked accordingly, in combination with the tilt position, the grabbing pattern shall vary. The sump should not normally exceed 1.75m average. And thereafter, air jetting or water jetting shall be resorted to.
- The sinking operation shall be done in two shifts, day & night. In normal coarse, the sump and the dredge hole water levels shall be observed twice in each shift, and the cutting edge reduced level shall be checked by level at four positions at the end of the shift.
- As the well sinks deeper, the skin friction on the sides of the well progressively increases. To counteract the increased skin friction and the loss in weight of the well due to buoyancy, additional loading known as kentledge is applied on the well. The kentledge is comprised of iron rail, sand bags concrete blocks etc.

- Pumping out of water from the inside of the well is effective when the well has gone deep enough or has passed through a clayey stratum so that chances of tilts and shifts are minimized during this process. When the well has been sunk to about 10 m. depth, sinking thereafter should be done by grabbing, chiseling and applying kentledge. Only when these methods have failed dewatering may be allowed upto depressed water level of 5 m. and not more.

  In case of sandy strata frictional resistance developed on the outer periphery is reduced considerably by forcing jet of water on the outer face of the well all round.

Damage of well due to breakwater

FLAC3D is applied to study the influence of pore water flow in the seabed and foundation. As previously mentioned, Darcy flow is assumed, which may be unrealistic for a highly permeable foundation where the pore water flow becomes turbulent. Again the four combinations of P2 max/P1 max and t1 listed in Figure 4 are analyzed. The results of the non-porous FLAC3D model and the analytical solution are compared with the outcome of two porous FLAC3D models with the hydraulic conductivity k = 10–4 m/s or k = 10–6 m/s of the foundation material. The hydraulic pressures on the caisson are applied as total stresses in the direction normal to the surface. However, the pressure on the base of the caisson is not provided as an external load in phase with the pressure on the front of the caisson, which has been assumed in the previous computations. Instead, an additional pressure is applied on the free surface of the foundation and the seabed in front of the caisson. The value of the pressure is P1(t) at the caisson front. For simplicity the pressure decreases linearly to 0 Pa at a distance of 40 m away from the caisson corresponding to a quarter of a wavelength. The boundary condition is applied as a total stress as well as a pore pressure, such that the effective normal stress remains zero at the free boundary.

Figures 12 and 13 show the time histories of the pore pressures and plastic shear strains for the impulsive load defined by (t1 = 0.2 s; P2 max/P1 max = 8). Similar trends are observed for the loads with lesser magnitude and shorter duration. It should be noted that the caisson is modeled as a non-porous material in all the analyses. The pore pressures in the caisson are a reminiscence from the first part of the computation, in which a hydrostatic pore pressure has been defined in the entire model before application of gravity. Thus, the pore pressures in the caisson do not change during the dynamic analyses. On the other hand, the pore pressure within the foundation and the subsoil will be influenced by the boundary conditions as well as the deformation of the caisson and soil.

A clear difference is observed between the response of the model with the hydraulic conductivity k = 10–4 m/s (Figure 12) and the less pervious model with hydraulic conductivity k = 10–6 m/s (Figure 13). In the first of the models, the permeability of the foundation material is much higher than that of the sandy seabed, in which k = 10–4 m/s. Therefore, the pore water runs relatively unhindered through the quarry rock, and the conditions right beneath the caisson are close to being fully drained.

However, in the sand under the toe of the foundation, a significant negative pore pressure builds up when the impulsive wave load acts on the caisson top during (see

Figure 12). The excess pore pressure dissipates over time and at t = 1.0 s the influence of the impulsive load has diminished. At the same time, a stationary flow has evolved through the foundation.

Furthermore, Figure 12 indicates that large plastic deformations occur on the front side of the foundation during the wave impact. The increasing pore pressure on the free surface induces a rapid flux of water into the foundation. Instead of a flow deeper into the foundation and soil, the water is stored close to the surface, leading to local increase of the volume. This is a result of the relatively simplistic constitutive model applied for the soil and is not regarded as physically sound behavior. In the remaining part of the foundation, a large part of the plastic deformations occurs after the passage of the impulsive load. At t = 0.4 s, a slip plane has formed at the foundation–seabed interface. At this stage, the deformation mechanism closely matches that of the non-porous models, cf. Figure 10. However, over the next half second, plastic deformations continue to develop, and the main part of the response takes place in a 5 m wide zone in the bottom half of the foundation behind the caisson.

In the previous computations without the pore pressure model, this zone remains completely elastic. When k is reduced to 10–6 m/s, the response of the caisson and subsoil changes significantly. The pore pressure in the seabed is only weakly influenced by the impulsive load. However, a negative pore pressure arises under the toe of the caisson and a positive excess pore pressure occurs at the heel during the wave impact due to the undrained behavior. This is observed in Figure 13 at the time t = 0.1 s.

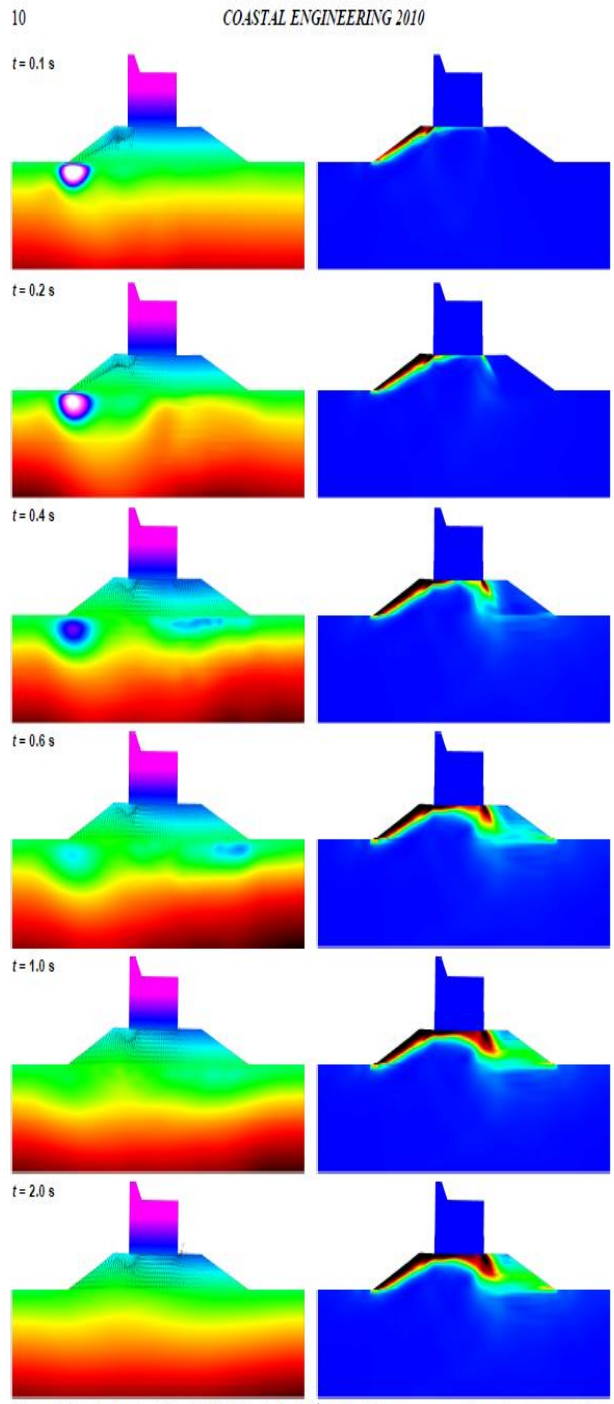


Figure 12. Results for $t_1 = 0.2$ s; $P_2^{max}/P_1^{max} = 8$; $k = 10^{-4}$ m/s Left: Pore pressures; the color range goes from magenta (p = 0 Pa) to dark red (p = 600 kPa); white shades indicate negative pressures and arrows show the flow. Right: Plastic strain magnitude (dark red shades indicate large deformation).

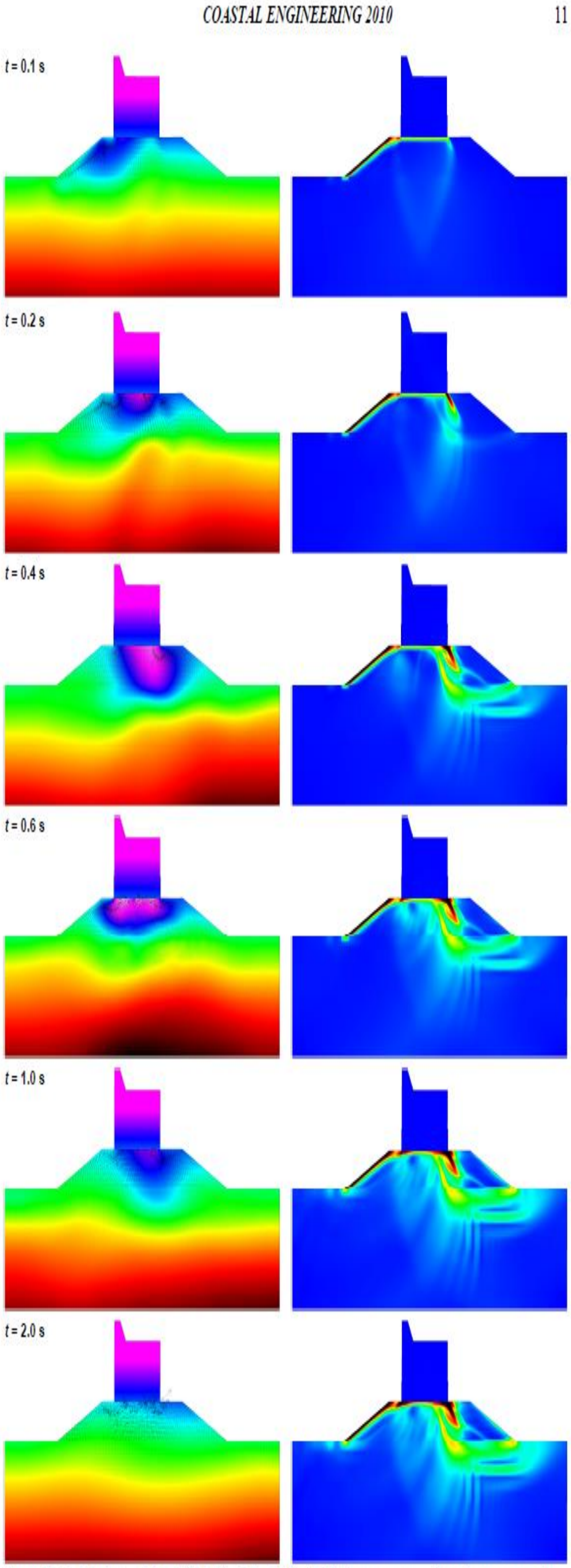


Figure 13. Results for $t_1 = 0.2$ s; $P_2^{max}/P_1^{max} = 8$; $k = 10^{-6}$ m/s. Left: Pore pressures; the color range goes from magenta (p = 0 Pa) to dark red (p = 600 kPa); white shades indicate negative pressures and arrows show the flow. Right: Plastic strain magnitude (dark red shades indicate large deformation).

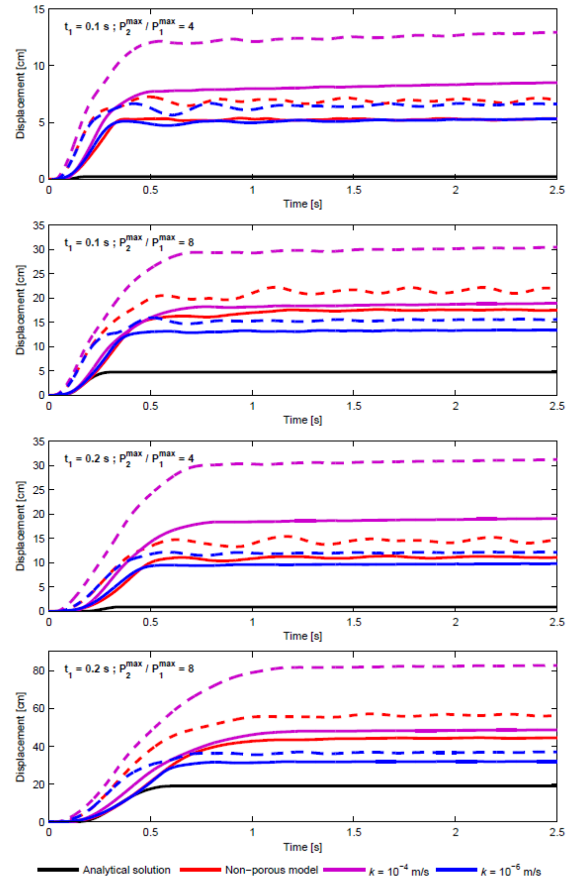


Figure 14. Displacements at the base (full drawn lines) and top (dashed lines) of the caisson front for the non-porous model and two models with different hydraulic conductivities of the rubble foundation.

GENERALIZED CONCLUSIONS

(1) With the developed interactive software one can analyze and design the details of a well foundation saving lot of monotonous efforts. Drafting of different cross-sections of the well with its dimension and reinforcement details is possible very easily.

(2) The maximum allowable lateral deflection for well by IRC method was 21.25 mm for the present study, whereas the lateral deflection obtained by the other methods were very less and generally in the order of 1 mm. However, depending on the end conditions specified the maximum deflection obtained by using Winkler model was of the order of 8 mm which was much lower than the allowable lateral deflection 21.25 mm specified by IRC.

(3) Design bending moment with IRC method was 62.62 % more than the maximum bending moment obtained by using Poulos and Davis approach.

(4) It was seen that the Winkler approach gave more (0.21 %) deflection as compared to the Poulos and Davis approach, the same for maximum bending moment was 2.56 %.

(5) Vertical deflection obtained by Poulos and Davis approach (1.19 mm) was more than the Winkler approach (0.337 mm). However, both the deflections could be considered negligible.

(6) It was seen form the analysis done by using IRC method, subgrade reaction approach and continuum approach, that the design done with IRC method was very conservative both in terms bending moment and lateral deflection. As such, IRC method could be used with confidence.

Well foundation is a popular foundation system in Indian subcontinent for bridges on rivers especially where the scour in river bed is a major concern. Many of these bridges are located in high seismic region. In the present study, seismic analysis of 2-D soil-well-pier system is performed for three possible cases of embedment length and for two earthquake motions in longitudinal direction considering structural and interface nonlinearity. The bridge is analysed assuming the piers and the well as linear structure and in the subsequent steps nonlinearity in piers and in well are added to see their effect on the response of the bridge. It is found that pier nonlinearity does not substantially reduce the response of well while nonlinearity in well reduces the rotational ductility demand in the piers. However, in such a situation, the well must possess adequate rotational ductility that in most situations may be impractical to ensure. Therefore, either one should increase the capacity of well or carry out more sophisticated analysis.

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MIVAN TECHNOLOGY

By – Mr. Chiranjit Samanta

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Abstract— **The aluminum formwork system was developed by Malaysian Company and that's why the aluminum formwork technology is named after it. Mivan is new construction technology upcoming for successful completion of mass housing project in India. In this project we have discussed about cost comparison of mivan technology with conventional construction technology. The Mivan technology is absolutely fine with cost, quality and time saving as compare to conventional. In this project we have taken a review from the people who are occupying the houses constructed by mivan technology to get the feedback from occupant on mivan technology. The project also include remedial measure for one of the defect in mivan technology i.e. segregation while placing the concrete resulting honeycombing in shear walls by using "MasterGlenium ACE 30JP" admixture.**

I. INTRODUCTION

The Mivan Technlogy System was developed by Mivan Company Ltd from Malaysia late 1990s as a system for constructing mass housing project in developing countries. The units were to be of cast-in-place concrete, with load bearing walls using a formwork of aluminum panels. To be erected by the hundreds, of a repetitive design, the system ensured a fast and economical method of construction. The concrete surface finish produced with the aluminum forms allows achievement of a high quality wall finish without the need for extensive plastering. This is one of the systems identified to be very much suitable for Indian conditions for mass construction, where quality and speed can be achieved at high level. The speed of construction by this system will surpass speed of most of the other construction methods/technologies

I. MIVAN FORMWORK

A. Requirement of Mivan Formwork

The Mivan formwork is made up of of an aluminium alloy. While Construction is in process , the formwork is supposed to bear, besides its own weight, the weight of wet concrete, the live load due to labor, and the impact due to pouring concrete and workmen on it. The vibration caused due to vibrators used to compact the concrete should also be taken care off. Thus, the design of the formwork considering its requirements is an essential part during the construction of the building. The Mivan Formwork should be able to take a live load including the impact about 370kg/m². It is however, usual to work with a small factor of safety in the design of formwork. The surfaces of formwork should be dressed in such a manner that after deflection due to weight of concrete and reinforcement, the surface remains horizontal, or as desired by the designer. The sheathing with full live load of 370 kg/m² should not deflect more than 0.25 cm and the joists with 200kg/m² of live load should not deflect more than 0.25cm.Maintaining the Integrity of the specifications. The modular nature of the mivan formwork should allow easy fixing and removal of formwork and the construction can proceed speedily with very little deviation in dimensional tolerances. Further, it should is quite flexible and can be easily adapted for any variations in the layout.

B. Genenral specification of Mivan Formwork

The basic element of the Mivan Formwork is the panel, which is an extruded aluminum rail section, welded to an aluminum sheet. This produces a lightweight panel with an excellent stiffness to weight ratio, yielding minimal deflection under concrete loading. Panels are manufactured in the size and shape to suit the requirements of specific projects. The panels are made from high strength aluminium alloy with a 4 mm thick skin plate and 6mm thick ribbing behind to stiffen the panels. Earlier the panels were used to manufacture only in factories in Europe and South East Asia but in recent the formwork componamts are started manufacturing in india as well e.g. COSMOS Construction Machineries And Equipments Pvt. Ltd . Once they are assembled they are subjected to a trial erection in order to eliminate any dimensional or on site problems. The formwork components are durable they can be used repetitively up to 200 times. It is light weighted so heavy lifting is eliminated, the heaviest components is of 25 kg, a labor can easily lift it

II. COST COMPARISON

By adopting Mivan technology in the project not only it gives the better quality of construction and but also increases the speed of construction and reduces the cost since some of the construction activities are completely eliminated and others are reduced to a extent . This project includes the cost comparison of conventional construction with Mivan Technology of construction. The following comparison is from the data acquired at Paranjpe schemes's "Blue Ridge" a 138 acre integrated township Hinjewadi , Pune.

A. Detailes about the structure

It is a part of Paranjpe schemes's "Blue Ridge" a 138 acre integrated township Hinjewadi , Pune. We have acquired the data of Tower HA-1 (25 Floors).

1.	Grade of Concrete	M 35
2.	Slump	180 – 200 mm
3.	Wall Thickness	External : 200 mm Internal : 100 mm
4.	Steel	Partition wall: 10 mm dia. Structural: 12 mm dia. Shear Wall : 16 mm dia.
5.	Slab Thickness	Hall : 175 mm Bedroom : 150 mm

		Kitchen : 125 mm
6.	Finishing	External : Texture Paint Internal : Paint over gypsum
7.	No. of Floors	25 floors
8.	Area	3 BHK : 184 sq. meter 2 BHK 125.4 sq. meter

Table 1: cost comparison between construction by conventional and mivan technology and mivan technology

Sr. No.	Parameter	Cost By Conventional Technology	Cost by Mivan Technology	Cost Saving
1.	Shuttering after repetitions	Wooden Materials =Rs. 88.50 /sq.m M. S. Material = Rs. 100.00/sq m	Rs. 83.8/ sq.m	Rs. 104.63/sq. m
2.	Concreting	Rs. 1400 / sq.m	Rs. 1505/sq.m	Rs. -105/sq.m
3.	Reinforcement	1,480.00	2,115.20	-635.2/sq.m
4	Brickwork	484.00	0.00	480/sq.m
5	Plaster	700.00	0.00	700/sq.m
6			Total cost saving	Rs. 548.43/sq.

III. FEEDBACK

It's now well known that the Mivan Technology reduces the cost of construction from above analysis, hence the technology is useful to the construction company and builder . However what about the end user i.e the people who are going to occupy the houses built by mivan technology. In India the occupants of houses built by mivan technology must have experience of living in a house constructed by conventional technology as mivan technology has recently came in india.

- Concrete using admixture: 1:1.2:3.2

C. Slump cone test result

Sr. No.	Description	Slump
1.	Conventional concrete	100 mm from top
2.	Concrete using admixture	220 mm from top

D. Compressive strength testing result

Sr. No.	Day	Load (KN)		Strength (N/mm^2)	
		Conventional	Admixture concrete	Conventional	Admixtured
1.	7	414	453	18.40	20.1
2.	14	577	610	25.67	27.1
3.	28	798	902	35.48	40.1

By using admixture the workabilty of concrete is incresed by 120 % whereas the strength of concrete is incresed by 13 %

Based on this fact, in this project we have taken a survey of people who are occupied in houses built by mivan technology. We have prepared questionnaire and took the feedback from occupant. The result analysis of selective question out of questionnaire of this survey is as follow

IV. HONEYCOMBING AND CRACKS IN SHEAR WALL

The mivan technology follows monolithic construction i.e. all the structural member viz. beam, shear wall, slab are casted at same time. In conventional construction the concrete is placed from height of 0.6 to 1 meter, and that is what recommended height to place the concrete. In Mivan Technology of construction the concrete is placed from height of 3 meter in shear wall and compacted using vibrator, now as height of placing concrete is more there are chances of segregation in concrete resulting in honeycombing and cracks in wall. In mivan construction it is generally happened that after removing formwork there is honeycombing in shear wall, in this project we had tried to fix the problem of honeycombing in shear wall. We had gone to BASF The chemical company pertaining this problem; they suggested us to use the MasterGlenium ACE 30JP as admixture to concrete so as to increase the workability of concrete to reduce honeycombing and increase the strength of concrete. One of the measures to check the workability of concrete is its slump and to check the strength is compressive strength. In this project we have compared the slump and strength of concrete using admixture and no admixture by slump cone test and compressive testing machine respectively. Following are the details .

A. Specification of MasterGlenium ACE 30JP

- Appearance : Brownish Liquid
- Specific Gravity: 1.00-1.02 g/cm3
- PH Value : 6-9

B. Concrete mix design

- Grade Designation: M35
- Type of Cement: OPC 53 Grades
- Reduced water content for admixture: 20%
- Mix design
- Conventional concrete: 1:1.4:2.2

V. CONCLUSION

The task of housing due to the rising population of the country is becoming increasingly monumental. In terms of technical capabilities to face this challenge, the potential is enormous; it only needs to be judiciously exploited by innovative construction methods. Traditionally, construction firms all over the world have been refraining to adopt the innovation and changes. It is the need of time to analyze the depth of the problem and find effective solutions. mivan serves as a cost effective and efficient tool to solve the problems of the mega housing project all over the world. MIVAN aims to maximize the use of modern construction techniques and equipments on its entire project.

We have tried to cover new aspects related to mivan technology viz. cost comparison based on case study, feedback from the people and remedial measure to solve the one of the major defect in mivan technology. We thus infer that mivan technology is able to provide high quality construction at unbelievable speed and at reasonable cost. This technology has great potential for application in India to provide affordable housing to its rising population.

Thus it can be concluded that quality and speed must be given due consideration with regards to economy. Good quality construction will never deter to projects speed nor will it be uneconomical. In fact time consuming repairs and modification due to poor quality work generally delay the job and cause additional financial impact on the project. Some experts feel that housing alternatives with low maintenance requirements may be preferred even if at the slightly may preferred even if at the higher initial cost.

From the survey and cost comparison we can come to the conclusion that mivan technology is win-win situation for the builder who is going to construct and consumer who is going to occupied the house. Hence mivan technology is the need of time to solve the problems of mega housing projects in India.

VI. FUTURE SCOPE

This thesis work is restricted to some aspects of Mivan technology . The future researchers can continue by working over the aspects of mivan construction such as Climatic effect on structure as whole structure is constructed in concrete only and Modernization in electrification work in mivan formwork. Furthermore interviews of different people from construction industry can be taken based on questionnaire prepared and analysis can be done.

ACKNOWLEDGEMENT

In regards we are extremely fortunate in having Asst. Prof. Mahesh V. Tatikonda (Department Of Civil Engineering) as our project guide. It had been not possible without his incredible help coupled with valuable suggestions, relentless effort and constructive ideas, more over his optimistic attitude, guidance and understanding making us believes all that accomplished was our effort for which we will ever remain indebted to him.. We would like to express our gratitude to Prof. Smita V. Pataskar, H.O.D. of the Department Of Civil Engineering for her escorting role in meeting our objectives. At this moment, we cannot forget to pay sincere regards to our Parents who are a big source of inspiration and blessings.

BUILDING WITH WASTE

_ By Subarna Das (T.A)

Waste is now a global problem, and one that must be addressed in order to solve the world's resource and energy challenges. Every year, human settlements produce 1.3billion tonnes worth of solid waste products. . So , some international companies currently under confidential agreement, to ultimately build commercial scale plants, capable of processing large scale plants which are capable of grinding the heterogeneous solid waste products from recycling facilities and feed the material through a complex system of hoppers, heaters and augers to produce good and eco-friendly building materials which are very cheap, durable in nature. So that in future, we could end up re-using pretty much everything, to build the cleanest and safest facilities in the world.

"The future city makes no distinction between waste and supply."

Can this quote be kept reserved for our future generation??

I think, the answer will be YES……… after reading these article, where new building materials are made from nasty waste based materials.

Building materials made entirely from waste products

1. Newspaper Wood

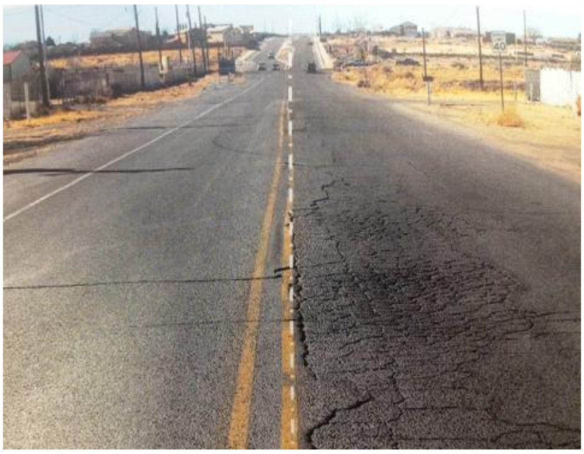
This design comes from Norway, where over 1m tonnes of paper and cardboard are recycled every year. The wood is created by rolling up paper and solvent-free glue to create something not dissimilar to a log, then chopping it into usable planks. The wood can then be sealed so it's waterproof and flame-retardant, and used to build anything you would normally build with wood.

2. Blood Brick

This idea rests on the assumption that animal blood counts as a waste product. This, we realize, is a potentially offensive idea – but while carnivores are still munching away, they're still wasting loads of animal blood, especially in societies without industrialized food production systems. And, as it turns out, blood is one of the strongest bio-adhesives out there, as it contains high levels of protein.

British architecture student Jack Munro proposes using freeze-dried blood (which comes as a powder), mixed with sand to form a paste; this can then be cast as bricks. This could be especially useful in remote communities, where blood from animal slaughter is plentiful, but strong construction materials are thin on the ground.

3. Plasphalt



Plasphalt is made up of grains of plastic produced from unsorted plastic waste, which replaces the sand and gravel traditionally used in asphalt production. In testing, it was found that plasphalt roads were far less vulnerable to wear and tear than traditional asphalt, because the asphalt emulsion bonded better with the plastic than with gravel or sand.

4. Recycled blocks

These colorful bricks are made from old plastic bags, which are notoriously difficult to recycle in any other way. Recycled bags or plastic packaging are placed in a heat mold, and forced together to form the blocks. They're too lightweight to act as load-bearing walls, but can be used to divide up rooms or outdoor areas. These recycled blocks can be used as a decorative partition wall.

5. Nappy roofing

Good news: something can be salvaged from all those nappies and sanitary products we throw away, even though they're, well, really gross. Special recycling plants separate out the polymers from the organic waste, and these polymers can then be used to create fiber-based construction materials like the tiles in the image above.

6. Mushroom walls

Here, designers figured out a way to grow wall insulator and packing materials using mycelium, a bacteria found in rotting organisms like tree trunks and agricultural byproducts. If placed in a mold, these organic matters grow to the desired shape within a couple of days, and can then be stopped using a hot oven. This is particularly useful because traditional insulating and packing materials tend to be non-biodegradable or, in the case of asbestos, poisonous.

7. Bottle bricks

This proposal is a little different, as it relies on producing a consumer good specifically so it can later be used as a building material. Lots of companies now make bottles in cuboids or other tesselative shapes, to make them easier to transport.

But the practice of doing so to create construction materials actually started with beer company Heineken in the 1960s – Alfred Henry Heineken, owner of the brewery, visited a Caribbean island and was dismayed at both lack of shelter, and the number of discarded Heineken bottles scattered everywhere. So the company landed on a new, brick-shaped design for the bottle, shown in the images above. The bottleneck slots into the base of the next bottle, forming an interlocking line.

Conclusions

This work effectively converts waste plastic and other hazardous non-biodegradable waste products into useful building materials like building bricks and floor interlocks which can effectively reduce the environmental pollution and further decreases the problem of waste plastics in the society. Rather than the waste plastics or other non-biodegradable waste going into the landfill or incinerators it can be used as construction materials at a much lower cost after undergoing certain specific processing. It also reduces the construction cost by eliminating the use of mortar during construction by using recyclable plastic/composite bricks and floor interlocks.

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WATER TREATMENT PLANT, Paddapukur

By Sayantika Saha
(Lecturer, Civil Engineering department)

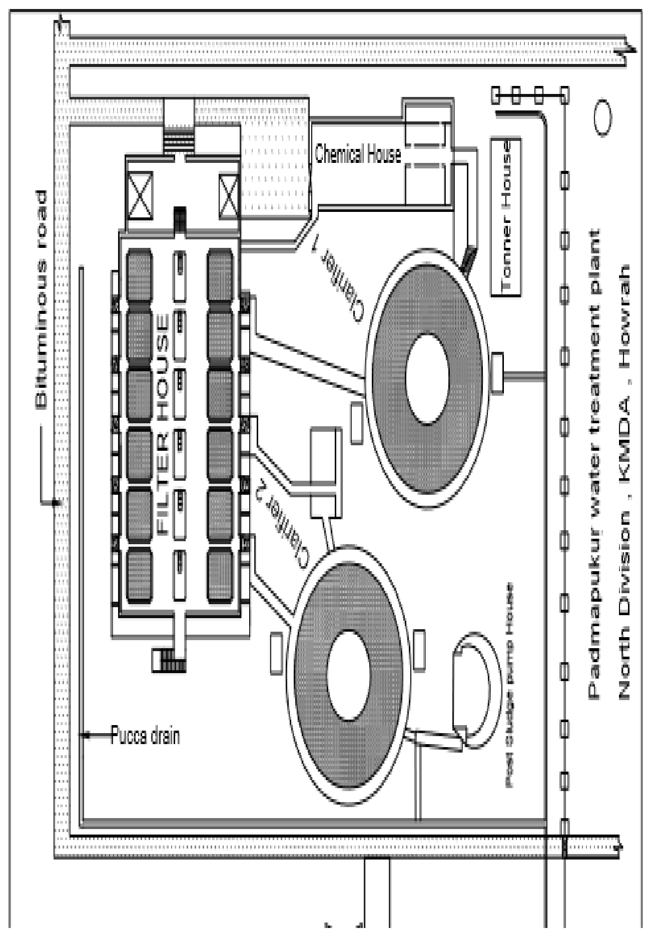
Abstract

The main aim of the field visit to surface Water Treatment Plant at Padmapukur , was to enhance the theoretical knowledge obtained in the college by observing the different procedures of the treatment plant. Different parts of the treatment plant were observed and the process was understood. The processes involved in removing the contaminants include physical processes such as sedimentation with coagulation , flocculation, clarification, filtration, disinfection.

Introduction:

Until 1984, the Howrah city used to get 18 MLD of treated surface water from Serampore water treatment plant. Padmapukur WTP with its headworks within Botanical Garden was set up in 1984 for supplying treated surface water. Padmapukur Water Treatment Plant, will be able to meet the water demand of the entire HMC area, upto end of Financial Year 2040, with complete reliability.80% water demand is met through the Padmapukur Water Treatment Plant. The rest of the demand is met through ground water sources.Duration of Water Supply is for eight hours per day as per the following schedule, 6 AM – 8 AM: 2 hours, 12 PM – 1:30 PM: 1.5 hours, 6 PM – 8 PM: 2 hours, 8:30PM-11:0.The following information which is collected from the field visit is given below:

Raw Water Pump House at Botanical Garden presently operates three pump units 1, 2, and 3 each of capacity 4251 m3/hr with a head of 16.80 mwc for Paddapukur Water Treatment Plant (PWTP) operated by Howrah Municipality Corporation (HMC). Normally two units operate at a time. However, sometimes one unit is operated at a time. The other four units 4, 5, 6, and 7 each of capacity 3412 m3/hr with a head of 23.0 mwc have operated by Kolkata Metropolitan Development Authority (KMDA). Normally two units operate at a continuous period of 3-4 days. However, most of the cases three units (one from HMC and two from KMDA) are operated at a time .



Modes of Treatment

There is various types of treatment unit is available.

Raw Water:

Raw Water Pump House at Botanical Garden presently operates three pump units 1, 2, and 3 each of capacity 4251 m3/hr with a head of 16.80 mwc for Paddapukur Water Treatment Plant (PWTP) operated by Howrah Municipality Corporation (HMC). Normally two units operate at a time. However, sometimes one unitis operated at a time. The other four units 4, 5, 6, and 7 each of capacity 3412 m3/hr with a head of 23.0 mwc have operated by Kolkata Metropolitan Development Authority (KMDA). Normally two units operate at a continuous period of 3-4 days. However, most of the cases three units (one from HMC and two from KMDA) are operated at a time .

Pre- Chlorination:

Chlorination is the application of chlorine to the water for the purpose of disinfection. During pre –chlorination, chlorine is usually added to raw water after screening and before flash mixing. It improves the coagulation and reduces load on filters.

Coagulation with alum

In water treatment, coagulation is a process that occurs when a coagulant is added to water to "destabilize" colloidal suspensions. Conversely, flocculation involves the addition of polymers that clump the small, destabilized particles together into larger aggregates so that they can be more easily separated from the water. Coagulation is a chemical process that involves neutralization of charge whereas flocculation is a physical process and does not involve neutralization of charge

Coagulant, when added to water forms spongy gelatinous precipitation, which absorbs fine size particles in water and bind them together. Jar test is used to determine the optimum dosage of coagulant .Alum is acidic in water and can reduce pH.

Flocculation and clarification

After flash mixing, flocculation begins through a slower, gentler mixing that brings the fine particles produced during the coagulation step into contact with each other. The flocculation phase usually goes on for 30-45 minutes in a flocculation basin that may have multiple compartments. Each compartment has a different mixing speed, and these speeds randomly decrease as water flows from the top of the basin to its bottom. This approach allows increasingly large clumps of matter to form without being broken apart by the mixing blades.

Clarifiers consist of tanks or basins which hold water or wastewater for a period sufficient to allow the floc and other suspended materials to settle to the bottom. The clarification process makes the water clear by removing all kinds of particles, sediments, oil, natural organic matter and color.

Clariflocculator is a combination of flocculation and clarification in a single tank.Inner tank serves as a flocculation basin and outer tank serves as a clarifier. Two no of clariflocculator is present

Filtration:

Filtration is a purely physical drinking water purification method. Rapid sand filters (RSF) provide rapid and efficient removal of relatively large suspended particles. Two types of RSF are typically used: rapid gravity and rapid pressure sand filters. For the provision of safe drinking water, RSFs require adequate pre-treatment (usually coagulation-flocculation) and post-treatment (usually disinfection with chlorine). Both construction and operation is cost-intensive. It is a relatively sophisticated process usually requiring power-operated pumps, regular backwashing or cleaning, and flow control of the filter outlet. Rapid sand filtration is common in developed countries for the treatment of large quantities of water where land is a strongly limiting factor, and where material, skilled labour, and continuous energy supply are available.

Size of Rapid Sand Filter 12x 2.5MGD Before filtration. Turbidity of clariflocculator (1) & clariflocculator(2) is 8.02 NTU & 6.40NTU respectively. Turbidity is reduced to 0.61 NTU after filtration process. Backwashing of RSF occurs daily.To meet the water demand there is no filter available for standby condition, all of these are in under working condition.

Post- Chlorination:

Post-chlorination is the final step in the treatment of water. It is a method of adding a minimum level of chlorine into the water mainstream down to distribution. The purpose of it is to remove pathogenic microorganisms, which can cause a variety of diseases, and also to prevent the growth of harmful

microorganisms by maintaining low levels of chlorine in pipelines.

Post-chlorination is the final step in the treatment of water. Residual chlorine of filtered water is 0.2mg/lit. Chlorine is then added at the rate of 3kg/hr.

Under Ground Reservoir:

12 m 600 m dia pumping main to feed treated water to 6 UGRs. 5 UGR is situated at Kona Expressway, Belgachia, Salkia, Bally, Liluah

FLOORING

By- Arghya Das (Lecture, Civil Engineering Department)

The flooring of a room sets the very 'mood' of a room. Care should be taken to see that the flooring matches the interiordesign and furnishing and accentuates the overall aesthetic look and feel of the room. The choice of the flooring material should be according to the purpose of the room and on the frequency of use, for example, materials like granite are appropriate for rooms like the living room and the veranda, which are frequently used.

A wide variety of flooring materials are available in the market today. In addition to conventional materials like marble and granite, manufactured flooring materials like ceramic tiles, vitrified tiles and terrazzo are widely used nowadays. The type of flooring used depends on two main factors-functionality and aesthetic appeal.

The different kinds of flooring available in the market are:

Granite:-
Granite flooring is not at all slippery.
Granite comes in over 45 shades. The most commonly used shades are black, tan brown and red. Quality-wise, all granites are same. Just the look is different. The rates of granites vary on colours.

Granites are maintenance free and are long lasting. They are scratch resistant, durable and very easy to clean.

A variety of colours to choose from and easy on your budget, granite is undoubtedly the best and the strongest material available today when compared to others like marble or mosaic.

Granites come in two forms. One is in Tile form and the other in Slab. In the tile form, the thickness is around 10 mm. This type of granite can be used for flooring and cladding. The price for tile ranges between Rs.25 per square feet to Rs.75 per square feet depending on the colour and size.

The thickness of the slab is between 17 mm and 18 mm. It is used for counters, kitchenplatforms,tabletops, pillars and walls. This is priced at Rs.50 per square feet to Rs.150 per square feet depending on the colour and size.

Marble:
Indian marble is definitely stronger, less porous and economical. But today there is more demand for the beautiful shades of Italian marble, that is softer and more porous. Despite this, the sheer pleasure of seeing a beautifully polished Italian marble surface makes people go in for it, compared to the Indian ones. Not all Indian marbles achieve the same glossy, glitzy look of the Italian variety. There are many marbles coming from all over the world such as Nepali, Yemeni, and Iranian that look very similar to the Italian marble. Another stone, which is largely available but rarely used, is onyx. Onyx is generally used in combination with other Italian marbles, as an inlay element, border or special design feature. It is also used as tabletops, bedside tables etc.

Marble flooring is a descendent of limestone and is hard. It comes in a number of shades and hues, ranging from off-white to brown, grey or pink. Stone with different coloured streaks is termed statuary marble. Although hard marble is softer than granite and is therefore more prone to stains and scratches. This can easily be repaired and corrected by polishing.

Finishes of Marble:-
Marble flooring can have one of two finishes, polished or honed.

<u>Polished:</u> This is a highly reflective finish and brings out the stone's natural shades and marking. This type of finish will not last in areas that have a flow of traffic.

<u>Honed:</u> This is a matte finish (non-reflective) and is better

suited in areas where there is a greater flow of traffic. It is also difficult to see scratches on this type of surface finish. The material itself comes in the form of tiles that are usually found in two dimensions: 12" by 12"and 12" by 18" and the thickness usually averages around an inch.

There are many other Indian stones available in different colours. There is a range of brown stones, such as Mandana, Agra stones and the Andhra variety. Another favourite of the architects is Jaisalmer stone, which is used alone or in combination with green Baroda marble.

Rectified tiles:
These tiles (made with a mix of marble and granite powder under high pressure) have high gloss porcelain coating. Since they come with sharp edges, the gap between two tiles is minimal and this gives a better look. Though stronger than ceramic tiles, rectified tiles get scratches easily. They cost Rs. 35 to Rs. 45 a sq. ft.

Ceramic tiles:-
Ceramic tiles are essentially clay tiles with a ceramic coatings of 80 micron thickness. Over a period of time, the coating withers away. But ceramic tiles are easy to maintain. A simple mopping is enough to keep them in good condition. They are best used in toilets. Laying them with proper packing is vital. Any gaps or vacuum underneath can have them breaking apart. And beware of "the seconds" in ceramic. About 40 per cent of the material are said to be seconds. Check the batch numbers and ensure that all the tiles you buy belong to the same batch. If batches differ, their sizes differ and there will be difficulty in laying them. These tiles cost about Rs. 20 to Rs. 40 a sq. ft. The seconds cost Rs. 16 a sq. ft.
For bathrooms, you get anti-skid ceramic tiles which come in sizes 8"x8" which cost Rs. 28 a sq ft. or more and 12"x12" which cost Rs. 26 or more.

Vitrified tiles:
These tiles compressed under 5,000 tonne pressure are the strongest among the manmade tiles and are scratch-resistant. They also have glossy finish. They are not porous and absorption is only 0.05 per cent or less. This means that coffee or lemon spills hardly cause any harm to them.
Semi-vitrified tiles have this quality only on the surface while fully-vitrified varieties have it across the section. The latter can be polished again and again to retain the original look. Vitrified tiles are easy to lay.
Semi-vitrified tiles cost Rs. 65 to Rs. 110 while fully-vitrified tiles cost Rs. 90 to Rs. 150 a sq. ft.

Hardwood floor:
Hardwood tiles are quite fashionable these days as they give an elegant look to your floor. But their maintenance is not that easy. Though they come with polyurethane coat, it hardly stays long and wears out fast. Termites can damage them and are inflammable. Cost ranges from Rs. 150 to Rs. 300 a sq. ft. depending on the type of wood chosen.

Andhra Marble:
Also known as BethamCharla tiles, they come in an amazing variety of textures and hundreds of colours. Skilled workers from Chittoor district alone can lay these tiles which are highly uneven in the bottom. The artisans can create almost any pattern one wishes to have. The Andhra Marble does not suffer wear and tear even after years of use. Instead it gets a better look. The tiles come in 10"x"10" size. Cost is Rs. 20 to Rs. 25 a tile.

Stones:
Kota stones from Rajasthan which are supposed to be the best alternative for marble (widely used in St. John's Hospital) come in 22"x22" slabs. The more you use it the more beautiful it looks, according to an architect. It costs Rs. 30 to 35 a sq. ft. Cuddapah and Shahabad stones which are other options. Very few prefer them because of their sedimentary nature and the polish does not remain for long. Cuddapah stones cost Rs. 12 to Rs. 15 a sq. ft. while Shahabad stones cost Rs. 15 to Rs. 20 a sq. ft.

Cement:
Even cement flooring is an option with a coating of red/black/green oxide. But as people do not prefer it these days, you do not get skilled labourers who can do it well. The citric acids cause stains on these floors. This flooring costs about Rs. 12 to Rs. 13 a sq. ft.

False Flooring:

It's a floor installed above the structural floor. It's used to conceal things like electrical cables, heat and air vents- etc. Much of it is made so these components can be accessed for maintenance or additions.